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CANADA - NEWFOUNDLAND

FLOOD DAMAGE REDUCTION PROGRAM

ENVIRONMENT CANADA | DEPARTMENT OF ENVIRONMENT

RUSHOON FLOOD STUDY REPORT

VOLUME 1 of 2

MAIN TEXT

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Report # SMR-22-85

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Dr. Wasi Ullah
Technical Committee
Canada-Newfoundland Flood Damage Reduction Program
c/o Newfoundland Department of Environment
Elizabeth Towers
St. John's, Nfld.
A1B 1R9

Dear Dr. Ullah:

We are pleased to submit Volumes 1 and 2 of our "Rushoon Flood Study Report".

We trust the findings of this study will provide useful advice on dealing with the flood problems in the Community of Rushoon, Newfoundland.

We have appreciated the opportunity of working on this technically challenging assignment and wish to acknowledge the assistance provided by members of the Technical Committee, Frank Murphy, Mayor of Rushoon, members of the Council and citizens of Rushoon.

Yours very truly,

ADPeach

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

PCH/bck
Encls:

EXECUTIVE SUMMARY

The principle objectives of this study were:

- identification of the mechanics, physical processes and factors responsible for producing floods in the Community of Rushoon.
- estimation of the 1:20 year and 1:100 year recurrence interval flood risk contours, and
- evaluation of suitable remedial and preventative measures to alleviate the flood problem.

To meet these objectives a program of analytical and field studies was undertaken. Analytical studies focused on two related areas:

- hydrology of Rushoon Brook, and
- hydraulic studies, including modelling of open water and ice conditions in Rushoon Brook.

Field studies, which included topograhic surveying, flow measurments and an ice observation program, were carried out in support of the analytical studies.

Methods for estimating flood peaks and for identifying ice and open water seasons on Rushoon Brook were developed in the hydrologic studies. These permitted identification of sets of annual maxima for "ice season" and open water conditions for the period 1966 - 1985. Frequency analyses were then performed on these data sets to provide conditional probability distributions for use in determining flood risk contours.

In the hydraulic studies, a computer model was used for simulation of open water and ice conditions on Rushoon Brook. This model incorporated river bed geometry obtained by a topograhic survey done during the field studies program. For open water conditions the model was calibrated and verified using flow profiles obtained in the field program. Calibration for ice modelling used water level observations reported for the 1983 flood with verification against water level data from the 1973 flood. Simulation studies were then carried out for a wide range of flows (5 m^3/s to 60 m^3/s) to investigate open water and ice behavior in Rushoon Brook.

These simulations generally confirmed observed behavior in Rushoon Brook and verified that the main ice jam site would be in the vicinity of Salmon Hole Point. This jam was shown to be stable for flows up to 40 m^3/s. Stage discharge curves were then derived from the results of the simulation runs for each cross-section for use in determining flood risk contours. Finally the ice model was used to investigate the performance of ice weirs and to test the effectiveness of channel improvements.

The 1 in 20 year and 1 in 100 year flood risk contours were delineated by applying the conditioned probability relationships, obtained in the hydrologic studies, to the stage - discharge curves produced in the ice studies. The resulting flood risk contours are shown in Figure 5.1 (in envelope at end of the report). The annual flood damage value was also assessed during this stage of the study utilizing synthetic flood damage data and stage-probability relationships at each section. The annual flood damage value was found to be about $3,000.

The following eight remedial measures were investigated:

Alternative #1	-	Channel Improvements
#2	-	Ice Storage Weirs
#3 (a)	-	Perimeter Dyke
(b)	-	Raise Fender Wall
#4	-	Flood Control Dam

Alternative	#5	-	Flood Proofing
	#6	-	Flood Warning
	#7	-	Control of Development
	#8	-	Placement of Boulders in Tidal Pool

If a "real interest rate*", i = 5%, is taken as the criteria for assessing feasibility then flood proofing of five houses is considered feasible (Alternative 5). All other alternatives were found to be uneconomic. If the guidelines on economic assessment were strictly followed only flood proofing of the basement of House #3 would be economically justified.

Sudden inflows of ice onto the flood plain at Rushoon, resulting from breakup surges, is a major hazard to life and limb. This problem can be eliminated, at minimum cost, by raising the fender wall as proposed in Alternative 3(b). The elimination of this hazard will be an important intangible benefit and therefore implementation of Alternative 3(b) is recommended.

Finally, a zoning plan is proposed to control new developments in flood prone areas.

Recommendations

The following list summarizes recommended actions for dealing with the flood problem in the Community of Rushoon:

(i) Alternative 3(b) - Raise Fender Wall, as indicated in Figure 6.4. The estimated capital cost of this work is $50,000.

(ii) Alternative 5 - Floodproofing, of five houses as in the following table.

* This interest rate corresponds to the current (bank) interest rate discounted for current inflation. Some authors prefer the terminology "effective interest rate", in this context.

(ii) Alternative 5 (Cont'd)

HOUSE	OWNER	SCOPE OF WORK
#3	Mr. Joe Hayden	Floodproof Basement
#13	Mr. Gary Lake	Raise house to sill elev. 5.40m (lift 1.4m)
#11	Mrs. S. Cheeseman	Raise house to sill elev. 5.70m (lift 1.2m)
#15	Mr. Roy Barrow	Raise House to sill elev. 5.05m (lift 1.1m)
#4	Mrs. M. Hayden (Store)	Raise house to sill elev. 5.90m (lift 1.1m)

The estimated cost of this work = $23,600. For these house locations see Figure 6.5. It is also suggested that Government consider extending aid to the owners of Houses #5 and #12, as well, since benefit cost ratios for raising these dwellings were only marginally less than unity.

(iii) Alternative 7 - Control Development

Modify the Municipal Plan for the Community of Rushoon - to incorporate the zoning requirements shown in Figure 6.6.

RUSHOON FLOOD STUDY REPORT

VOLUME 1 of 2

MAIN TEXT

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1. INTRODUCTION

1.1 Background

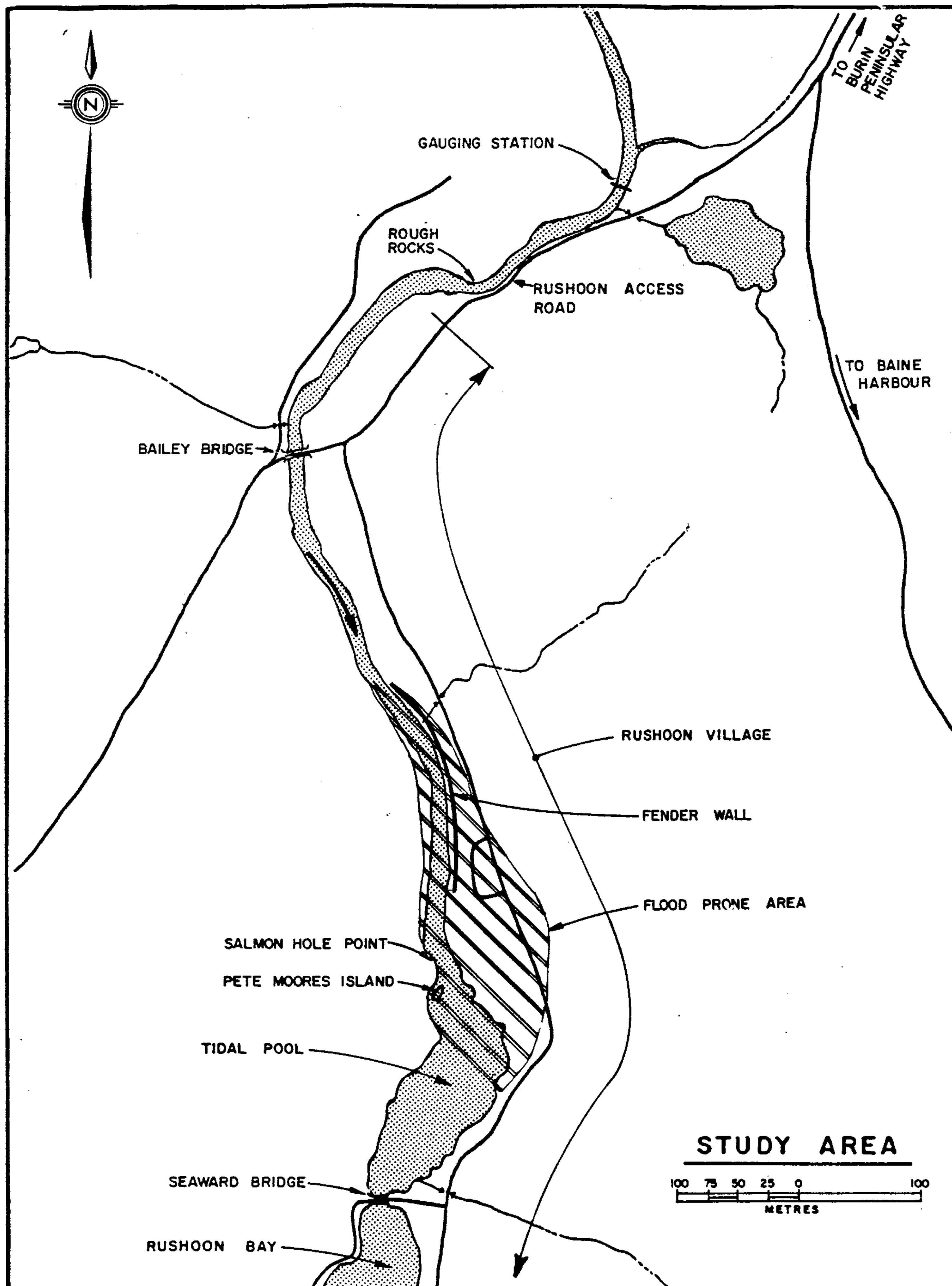
The Community of Rushoon is located on the Placentia Bay coast of the Burin Peninsula on the Island of Newfoundland. The original settlers built their homes around the shore of Rushoon Bay, where it was convenient for mooring their boats. More recent settlers have preferred to build their homes inland, on a zone of level ground bordering the lower portion of Rushoon Brook, as shown in Figure 1.1.

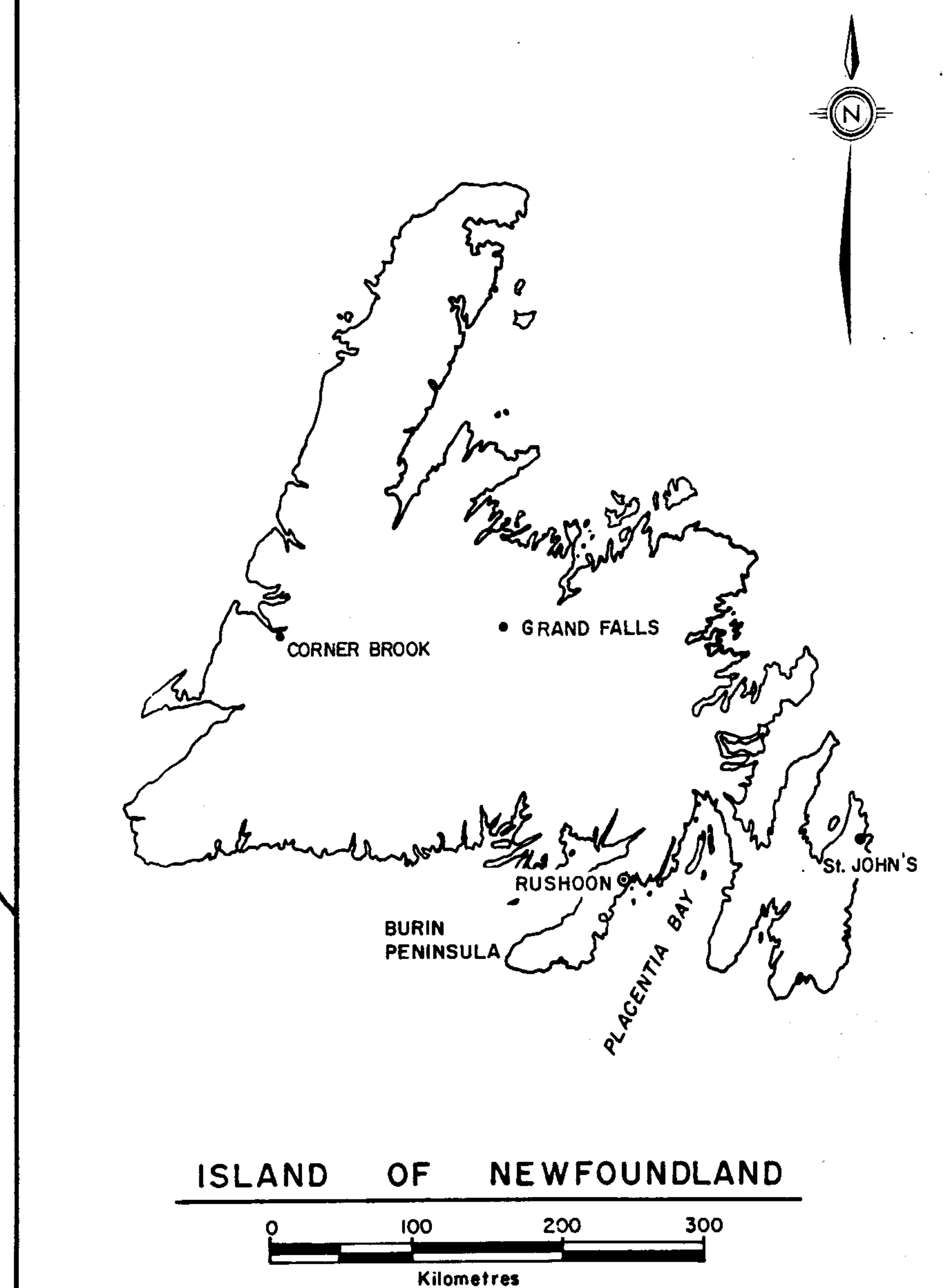
Unfortunately, this otherwise attractive riverside zone, encroaches upon the flood plain of Rushoon Brook and consequently homes in this area have been subjected to periodic flooding, including severe floods in 1973 and 1983 in which several homes were damaged.

In response to this problem the Governments of Canada and Newfoundland decided to commission a thorough study into the flood problem at Rushoon under the auspices of the Canada - Newfoundland Flood Damage Reduction Program.

1.2 Authorization

In the fall of 1984 a request for proposals was published giving the terms of reference for the Rushoon Hydrotechnical Study. The proposal submitted by ShawMont Newfoundland Limited in conjunction with LaSalle Hydraulic Laboratory Limited was selected. The study contract was subsequently awarded to ShawMont Newfoundland Limited by letter from the Newfoundland Department of Environment, signed by the Minister, the Hon. Hal Andrews, and dated March 5, 1985.





RUSHOON HYDROTECHNICAL STUDY

LOCATION MAP

FIGURE 1.1

2. METHODOLOGY

2.1 Objectives of the Study

The principal objectives of this study, as outlined in the Terms of Reference, were:

-- identification of the mechanics, physical processes and factors responsible for producing floods in the Rushoon area,

-- estimation of the 1:20 and 1:100 year recurrence interval flood levels and the extent of flooding associated with each, and

-- evaluation of suitable remedial and preventive measures to alleviate the flood damage problem in the study area.

The study area comprises the portion of the Community of Rushoon bordering the lower portion of Rushoon Brook, generally downstream of the section known as Rough Rocks to its outlet in Rushoon Bay (see Figure 2.1).

A detailed description of the Terms of Reference can be found in Reference(1)*

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* Numbers in parenthesis refer to the Reference list at end of the Report.

2.2 Rushoon Brook Climate and Topography

Rushoon Brook has a small drainage area of about 58 km^2. The upper portion of Rushoon Brook, comprising about 80% of the total drainage area lies on the plateau of the Burin Peninsula at a mean elevation of about 100 m above MSL. This area is predominantly open-rolling-barrens with significant tree cover limited to river valley zones. There are many small lakes up to a maximum of 0.75 km^2 in area. The lower portion of Rushoon Brook comprises a series of "gullies"* separated by rapids and waterfalls as the river descends from plateau to sea level. The watershed has not been signficantly altered by human activities, flow is unregulated and no dams exist in the watershed.

The climate on the Burin Peninsula is significantly modified by proximity to the ocean and hence may be described as being a temperate-maritime climate. This type of climate is typified by cool summers and mild winters and does not display the extremes of temperature typical of temperate-continental climates. Prolonged periods of sub-freezing weather (in excess of 30 days), such as observed in continental locations at the same latitude, are infrequent - the normal pattern indicates several periods of above freezing weather can be expected during the course of a typical winter. Another feature of

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* Wide slow flowing reaches in a river are called gullies in Newfoundland parlance.

the climate on the Burin Peninsula is the regularity of precipitation throughout all months of the year. The following table summarizes pertinent climate statistics for locations at sea level on the Burin Peninsula. Points inland at higher elevations will tend to have somewhat lower mean temperatures and higher precipitation due to orographic effects:

TABLE 2.1 SUMMARY OF CLIMATE STATISTICS FOR THE BURIN PENINSULA NEAR RUSHOON*

Annual Mean Daily Temperature	=	$5^{o}C$
Mean Daily Temperature, February	=	$-4^{o}C$
Mean Daily Temperature, August	=	$15^{o}C$
Annual Precipitation	=	1250 mm
Percentage of Annual Ppt. as snow	=	15%
Precipitation in November (wettest mo.)	=	135 mm
Precipitation in July (driest month)	=	75 mm

2.3 Description of Past Flooding

All major floods that have been observed on the lower portion of Rushoon Brook were ice jam floods with the most severe events generally occurring in mid-winter following lengthy periods of sub-freezing weather.

* These statistics were estimated from the published Climatic normals for the Come-by-Chance, Grand Bank and St. Lawrence climatological stations on the Burin Peninsula.

Although occurrences of ice jam floods in this study area are reported to be common, documentation on past flooding is unfortunately scarce and is limited mainly to the floods which occurred in 1973 and 1983.* Observations on Rushoon Brook indicate the following sequence of ice build-up:

(i) at the beginning of winter an ice cover initially forms on the more tranquil sections of Rushoon Brook - the section downstream of Rough Rocks including the Tidal Pool. The three gullies lying between Rough Rocks and the Burin Peninsula Highway also freeze over at this time (see Figure 2.1 for locations).

(ii) freeze-up occurs at a later date in areas of more rapid flow, notably Rough Rocks and at the Water Fall, although a few leads normally remain open throughout winter in localized areas of extremely rapid flow. Substantial quantities of frazil ice are produced in these areas.

The sequence of breakup is as follows:

(i) the initial release of ice is caused by rising water levels in the portion of the brook immediately downstream of the Water Fall

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* Detailed accounts of these floods, prepared by the Newfoundland Dept. of Environment are given in Volume 2, Appendix I.

2.3 Description of Past Flooding (Cont'd.)

(ii) breakup proceeds sequentially in a downstream direction and, barring ice jams, continues via Rough Rocks, through the lower section of the Brook, the Tidal Pool to the ocean clearing these sectors of ice.

(iii) ice from the three gullies lying between the Water Fall and the Burin Peninsula Highway, normally comes down 10 - 12 hours after breakup on the lower portions of the Brook, according to observations taken by Mr. F. Murphy - Mayor of Rushoon.

(iv) ice in the ponds upstream of the Burin Peninsula Highway Bridge decays in place and does not contribute to ice jams in the study area.

Ice jams are reported to form in three lodgement areas:

- at Salmon Hole Point,
- just upstream of the Bailey Bridge, and
- at Rough Rocks.

Features at Salmon Hole Point which contribute to ice jam formation include:

- a natural rock dyke encroaching upon the flow section from the right bank,
- shallow water adjacent to Pete Moores Island, and

2.3 Description of Past Flooding (Cont'd.)

- the presence of a static ice cover on Tidal Pool (particularly when this cover is grounded at low tide).

It is reported that water levels in the Tidal Pool significantly affect the tendency of ice jams to form at Salmon Hole Point: that ice jams are unlikely to form if breakup occurs when the tide is high, but likely to form if breakup coincides with a low tide when the ice cover in Tidal Pool may be grounded.

On some occasions in the past, ice brought down by the Brook on breakup has been deflected by Pete Moores Island and rafted up in the area now occupied by the ball field. In order to eliminate this problem Pete Moores Island has been levelled by several bulldozer operations during the period 1969-1973. However, it appears that this area may be filling in with gravel from upstream erosion.

The severe floods which occurred in 1973 and 1983 were reported to be the results of ice jams which formed in the vicinity of Salmon Hole Point.

During 1973 an ice jam was also reported just upstream of the Bailey Bridge. This jam was apparently keyed between the abutments of the old concrete bridge which used to span Rushoon Brook at this location*. It was also responsible for flooding one home. Following the 1973 flood the old abutments have been removed to eliminate this obstruction.

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* The bridge was destroyed in 1971 when hit by a heavy truck.

2.3 Description of Past Flooding (Cont'd.)

Ice jams also have been reported to occur in the Rough Rocks sector of Rushoon Brook. This is the narrowest section of the river and was further restricted when the Rushoon Access Road was widened in 1963. A jam at this site in 1973 resulted in an ice pile-up and flooding of the road which lasted for 2 to 3 days.

Dramatic breakups have been observed on Rushoon Brook when surges of ice and water cascade through the lower section of the river. The rate of travel of such a surge was estimated by an eye witness to be between 20 - 30 km/h for the breakup preceding the March 1983 flood. Under these circumstances large slabs of ice were pushed into the town causing some damage to property but fortunately no injuries to residents*, see Figure 2.2.

Following the 1973 flood a fender wall was constructed alongside the river in the affected area. So far it seems to be an effective remedy and successfully prevented ice slabs from entering the community during the 1983 flood.

An appreciation of the 1973 and 1983 floods can be gained from the photo collections in Figures 2.2, 2.3 and 2.4.

House locations are shown on Figure 2.5.

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* In 1973 a little girl was reported to have been saved just in the nick of time as she was being swept away by the flood waters.

2.3 Description of Past Flooding (Cont'd.)

During these floods, Rushoon Brook invaded much of the area lying between the main road through the community and Rushoon Brook, as shown in Figure 2.1. Some 6-8 homes were affected by flooding with water depths of 0.3 - 0.6 m being experienced on the grounds adjacent to these homes, sufficient to flood basements (one home) and approaching or exceeding ground floor elevations in other homes. Such shallow water depths would not pose a serious safety hazard; however, a far greater hazard results from the sudden inrush of ice and water produced by break-up surges from upstream. This problem was particularly evident during the 1973 flood as witnessed by photos shown in Figure 2.2 and noted earlier.

2.4 Study Approach

Descriptions of past floods indicate that flooding on the lower portion of Rushoon Brook is associated with jams of loose river ice occurring at breakup, triggered by a period of thawing temperatures, rainfall and consequent increased flows. Accordingly, this study focused on the factors associated with the occurrence of breakup ice jams. The severity of floods related to freeze-up jams and open water conditions were also examined to confirm that these conditions were not critical.

The methodology which has been applied in this study assumes, in essence, that an ice jam flood would be produced by any flood flow occurring during the "ice season". For the purposes of this study, the "ice season" is defined as the period(s) during which sufficient ice would be available in Rushoon Brook to form an ice jam if breakup occurred.

2.5 Study Program

The study program was organized around four major activities, as explained below:

(i) Hydrologic Studies - the purposes of these studies were to delineate past ice seasons and to select open water and ice season peak flows for each year of the nineteen year study period (1967 - 1985).

(ii) Hydraulic Studies - the purposes of these studies were (a) to investigate the factors which produce ice jams in Rushoon Brook and to develop stage - flow relationships for ice jam floods, and (b) to model open water flows and produce open water stage-flow relationships.

(iii) Engineering and Economic Studies, to identify and evaluate remedial measures.

(iv) Field Studies - to provide topographic data and information on ice conditions during the winter of 1984/85 in support of the above analytical studies.

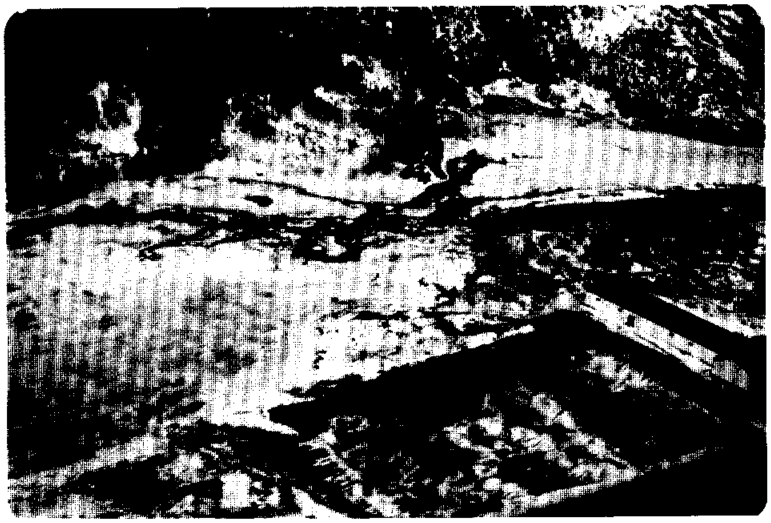
Details on the hydrologic, hydraulic studies and engineering-economic phases of the study program are provided in the following sections of this report; while, detailed results of the field studies are described in separate reports (2) and (3). Inter-relationships between the various elements of the study plan are shown in Figure 2.6.

FIGURES

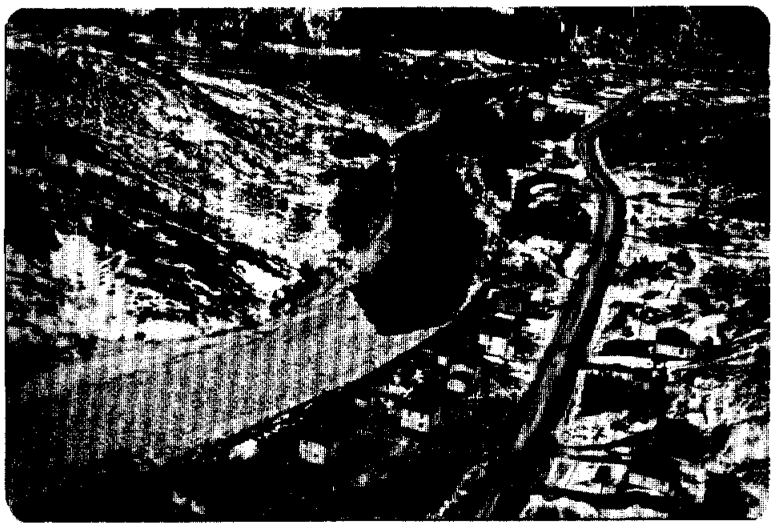
Photo Credits:

Photos in Figure 2.2, Courtesy - Mr. Joe Hayden, Rushoon

Photos in Figures 2.3 and 2.4 Courtesy - Water Resources Division, Dept. of Environment (Nfld.)



Rushoon River - March 5, 1983, Salmon Hole Point in centre of Photo.



Rushoon River upstream limit of ice jam, March 5, 1983..

Rushoon River Ice Jam, March 5, 1983.

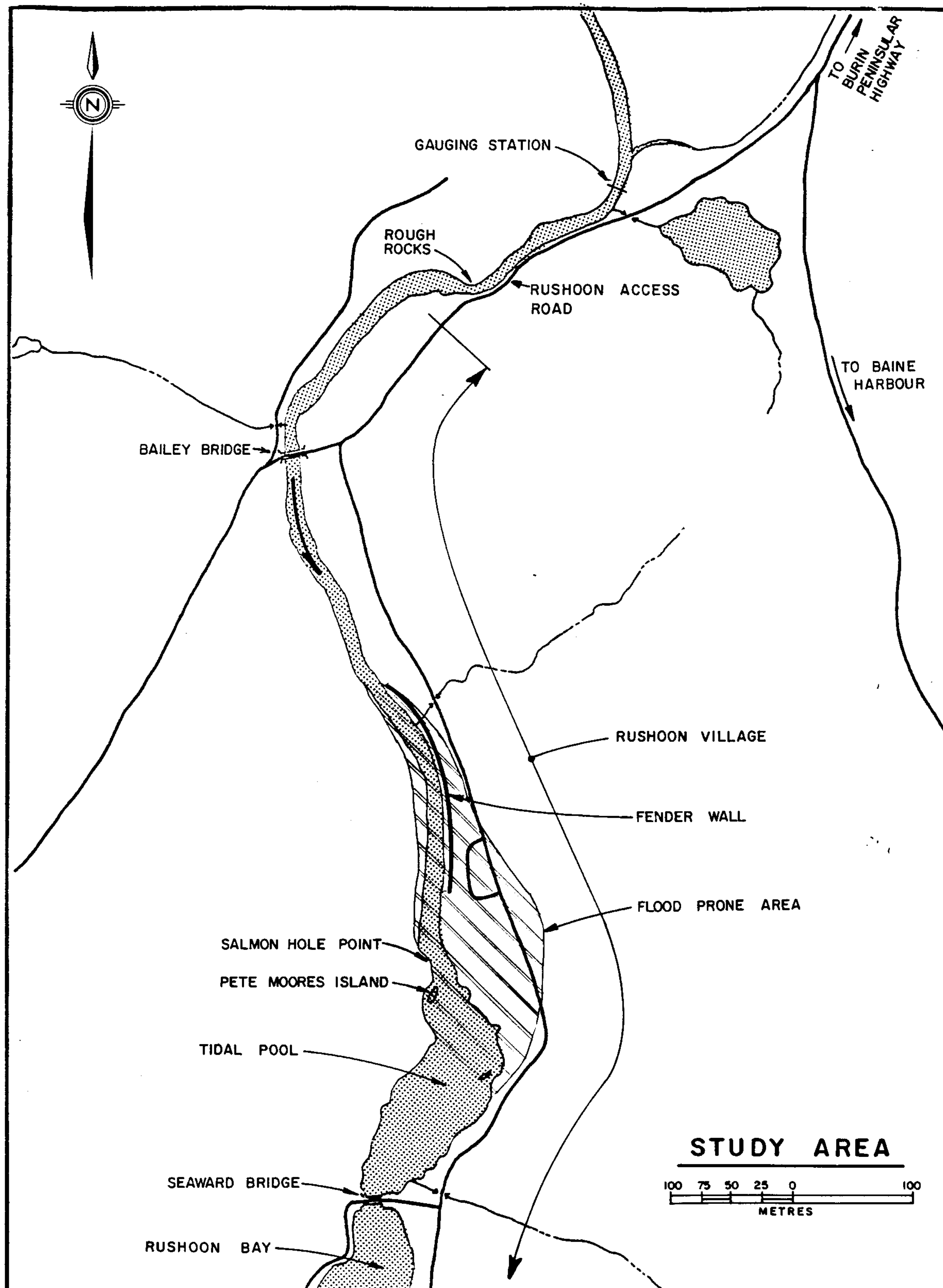
FIGURE 2.3 MARCH 1983 FLOOD - AERIAL VIEWS

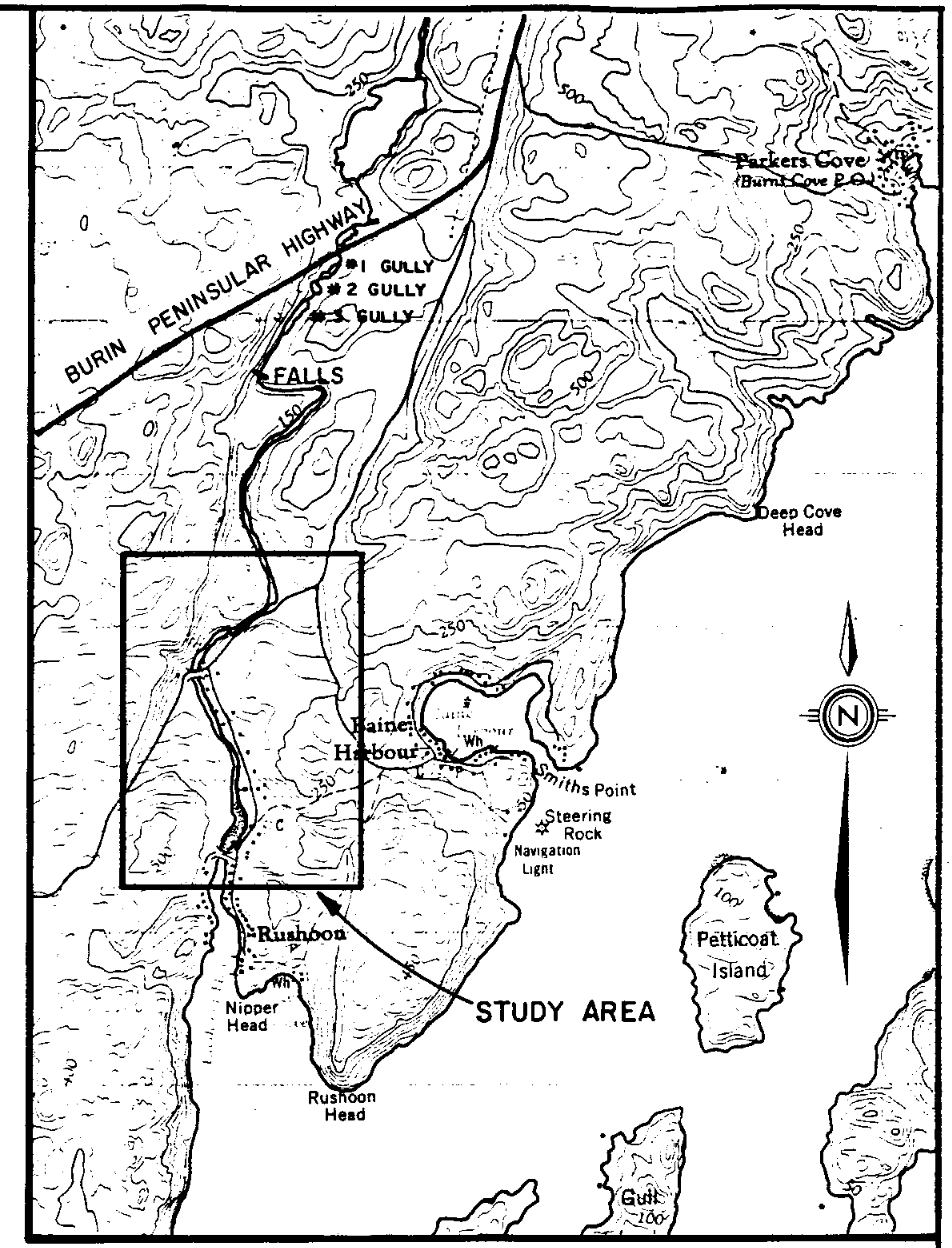
Flooding along Main Road, river is to right of houses.

Start of Fender Wall. Note flow under and through wall.

Upstream of Fender Wall, looking towards Harbour.

FIGURE 2.4 MARCH 1983 FLOOD - CLOSE UP VIEWS





LOCATION MAP
1: 50,000

RUSHOON HYDROTECHNICAL STUDY

MAP OF STUDY AREA

FIGURE 2.1

Looking upstream towards house #4. Main Road in background.

Looking towards house #13. Main Road in background.

House #3 at centre of photo. Note build-up of ice where Timber Crib Fender Wall now stands.

Looking upstream towards house #3. Main Road to the right.

Looking upstream towards houses #5, #4 and #3. River is to the right.

View taken from Main Road looking toward house #3. River in the background.

FIGURE 2.2 FEBRUARY 1973 FLOOD

3. HYDROLOGIC STUDIES

3.1 Purpose

The main objectives of this phase of the study program were:

(i) to develop means for estimating peak flows on Rushoon Brook, and

(ii) to develop criteria for delineating "ice season" periods on Rushoon Brook.

(iii) to carry out frequency analyses for "ice season" and open water season floods.

3.2 Estimation of Peak Flows

No flow data is available for Rushoon Brook, however, flow data is available from an adjacent watershed - Rattle Brook for which daily flow data have been collected since January 1, 1981. As shown in Table 3.1 both basins are of similar size and have similar characteristics; hence it was decided to estimate Rushoon Brook peak flows by prorating flows measured on Rattle Brook.

TABLE 3.1: Comparison of Basin Characteristics Rushoon Brook versus Rattle Brook

Parameter:	Rattle Brook	Rushoon Brook
Drainage Area (km_2)	42.7	55.5
Mean Annual Runoff (mm)	1200	1200
Area Controlled by Lakes and Swamps (%)	83.2	86.5
Shape Factor	1.39	1.62
Mean Annual Flood Peak Q_2 (instantaneous), m^3/s	31.1	28.9

3.2 Estimation of Peak Flows (Cont'd.)

The proration factor, K, was taken as the ratio:

$$K = \frac{Q_2 \text{ Rushoon Bk.}}{Q_2 \text{ Rattle Bk.}}$$

$$\therefore \quad K = \frac{28.9}{31.1} \doteq 0.93$$

where Q_2 = the mean annual instantaneous flood peak. Values of Q_2 for Rushoon and Rattle Brooks were computed using a formula from a recent regional flood frequency study for the Island of Newfoundland (4).

For estimating flows prior to 1981 it was necessary to develop multiple correlation equations relating peak flows on Rattle Brook with corresponding peak flows on neighbouring gauged rivers. The rivers used in this analysis were Come-by-Chance, Piper's Hole and Garnish Rivers. All available data up to the end of 1984 was examined and eighteen comparable flood peaks on each river were extracted from the period of overlapping records for inclusion in multiple regression analyses. Care was taken to ensure that individual flood peaks were mutually exclusive events and not influenced by persistence from preceding floods.

Various groupings of data were tested by regression analysis, in both linear and transformed modes to select the relationship having the best statistical fit. The selected formulae were then used for estimation of peak flood flows on Rushoon Brook. These formulae are summarized in Table 3.2.

3.2 Estimation of Peak Flows (Cont'd.)

TABLE 3.2: Formulae for Estimating Peak Flows on Rushoon Brook

Period	Formulae	Data Points	(R)
1981-1985	$Q_R = 0.93Q_{Rattle}$ (Equation 3.1)	N/A	N/A
Prior to 1981	$Q_R = 0.93(0.0217Q_p + 0.5066Q_G - 0.1685)$ (Equation 3.2)	18	0.92
	$Q_R = 0.93(0.0368Q'_p + 0.358Q'_G + 0.616)$ (Equation 3.3)	18	0.92

Where Q_R = Instantaneous peak flow on Rushoon Brook
Q_G = Mean daily flow on Garnish River
Q_p = Mean daily flow on Piper's Hole River
R = Coefficient of Regression
Q'_G = Instantaneous peak flow on Garnish River
Q'_p = Instantaneous peak flow on Piper's Hole River

Both correlations were found to be significant at the 1% level with standard errors of estimate of 3.79 m^3/s and 3.78 m^3/s for equations 3.2 and 3.3, respectively. For hydrologic studies this level of significance is considered to be good and the correlation equations to be a reliable means of estimating peak flows.

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* (Details of the multiple regression analyses are given in Volume 2, Appendix II).

3.2 Estimation of Peak Flows (Cont'd.)

During the field studies phase of the program a flow gauging station was installed on Rushoon Brook and a stage - discharge relationship developed for the gauged section (2).

The peak spring flow occurred on May 5, 1985 and was estimated to be 24.1 m^3/s. The comparable peak flow, derived by proration from the Rattle Brook gauge was 19.8 m^3/s. It had been hoped that these results would agree within $\pm$ 10% ($\pm$ 2 m^3/s). Notwithstanding this outcome, a single data point is clearly insufficient to confirm the accuracy of the proration factor. The significance of errors in flow estimation are further discussed in Section 5.4.

3.3 Criteria for Delineating Ice Seasons

As noted previously, Rushoon Brook will only be susceptible to the occurrence of breakup ice jams during periods when sufficient quantities of river ice are available to form a breakup jam. These periods have been designated as the "ice season" in this study. Delineation of the ice season requires development of a set of criteria to indicate the occurrence of the following events in the process of ice formation/growth/decay and/or breakup:

(i) date of initial formation of the ice cover, and hence magnitude of the freeze up flow - Q_f

(ii) date when the quantity of ice in the brook reaches the critical volume needed to produce a breakup jam. This marks the beginning of the ice season.

3.3 Criteria for Delineating Ice Seasons (Cont'd.)

(iii) date on which breakup would be produced by hydraulic conditions: that is, when flow would increase to a magnitude at which the ice cover would no longer be stable.

(iv) date on which the river would be cleared of ice by thermal decay of the ice cover.

The earliest of the dates determined in (iii) or (iv) above would mark <u>the end of the ice season.</u>

Detailed site specific information for Rushoon Brook on these processes is limited to a single winter of detailed observation - January - April 1985. Related but less detailed information, can be deduced from the reports and photographs of the 1973, 1975 and 1983 floods.

Complementary data is also available for part of 1984 from the Newfoundland Light & Power Company, from a study on North Harbour River by Environment Canada Fisheries and Marine Service (5) and from backwater indications on the flow records published by the Water Survey of Canada.

A substantial amount of information on the processes of ice growth and decay is also available in the literature on ice engineering; but unfortunately, most of this literature has limited applicability since it concerns relatively large rivers in nordic regions; whereas Rushoon Brook is a relatively small river in a maritime climatic region - References 6, 7 and 8.

The following paragraphs give summary descriptions of these criteria and their derivation:

3.3 Criteria for Delineating Ice Seasons (Cont'd.)

3.3.1 Criteria for Identifying Start of the Ice Season

(i) first ice forms (at start of winter) after a freezing degree day total of $40^{o}C$. days has been reached (calculated as $\Sigma -T^{o}C$.days for $T \leqslant 0^{o}C$). This estimate of initial cooling, which accounts for cooling the source water bodies to $0^{o}C$, was based on data for the winter of 1983/84, from the West Pond Power Canal, near Marystown provided by the Newfoundland Light & Power Company. Similar values are cited in ice engineering literature (6).

(ii) refreezing after a mid-winter breakup - occurs on the first day on which daily average temperature falls to $-5^{o}C$ or below (8). This conforms with the observation that river ice growth requires lower temperatures than for static ice growth as on lakes or ponds.

The "freeze-up" flow, Qf, occurring when first ice forms or upon refreezing should be noted, since knowledge of Qf is required for predicting the breakup flow Q_b.

(iii) The ice season start is assumed to occur when the ice thickness t, reaches 15 cms (15 cm of ice on the reach below the falls will produce about 12,600 m^3 of ice, the amount required to produce a significant jam*).

The ice thickness (t) is predicted from the following equation:

$$t = 4.5\sqrt{S} - 4.8\ D\ ... \qquad \text{Equation 3.4}$$

---

* This amount corresponds to the volume of ice in the grounded portion of the 1983 ice jam as estimated from photographs and reports of this event.

3.3.1 Criteria for Identifying Start of the Ice Season (Cont'd.)

Where

t = ice thickness in cm

S = degree days of freezing calculated as $\Sigma - T\ ^{\circ}C.$ days for $T \leq 0^{\circ}C$

D = degree days of thawing calculated as $\Sigma - T\ ^{\circ}C.$ days for $T > 0^{\circ}C$

This equation is a unified ice thickness equation taking into account both processes - thermal ice growth and decay. These processes are fundamentally different:

- ice growth results in thickening of the ice sheet from the bottom surface as a function of heat loss from the water via the ice sheet itself. As the ice sheet thickens its insulating capacity is increased and the rate of thickening slows down. The process of ice growth is thus non-linear and is approximately proportional to $\sqrt{S}$ (6)

- ice decay occurs by conversion of ice (assumed to be at $0^{\circ}C$) to water at a rate proportionate to the supply of thermal energy (mainly from the atmosphere). This process takes place on the upper surface of the ice sheet and is independent of the thickness of the sheet, hence decay is proportional to D (8).

3.3 Criteria for Delineating Ice Seasons (Cont'd.)

3.3.1 Criteria for Identifying Start of the Ice Season (Cont'd.)

The coefficients 4.5 and 4.8 in the unified ice thickness equation have been chosen to fit ice thickness observations taken on Rushoon Brook during the winter 1984/1985. In view of the approximate nature of this equation, it should be regarded mainly as an indicator of ice conditions rather than taken as an exact means of computing ice thickness.

Derivation of Equation 3.4 and test applications of these procedures are demonstrated in Volume 2, Appendix III.

3.3.2 Criteria for Identifying End of The Ice Season

(i) Hydraulic breakup will occur on the first day that the breakup flow, Qb exceeds the freezeup flow Qf.

$$Qb > Qf$$

Results of other studies (9) indicate that breakup flow can be related to freezeup flow on a river.

(ii) Thermal decay, ice disappears on date when t (as calculated by equation 3.4) is reduced to zero.

End of the ice season is determined by the earliest date as determined in (i) or (ii).

3.3.2 Criteria for Identifying End of The Ice Season (Cont'd.)

A rigorous verification of these criteria is not possible due to scarcity of data; however, the criteria were found to fit available observations on Rushoon Brook and are consistent with indications of ice conditions reported by the Water Survey of Canada at stream flow stations on the Burin Peninsula - see Figure 3.1.

3.4 Frequency Analysis

Peak ice season and open season flows have been estimated on Rushoon Brook by transposition from Rattle Brook as explained in Sub-Section 3.2. By these means, sets of nineteen (19) annual ice season peak flows and eighteen (18) annual open season peak flows have been generated for the period 1967-1985, as summarized in Table 3.3. Frequency analyses of these data sets were carried out using the FDRPFFA computer program (10). The relationships shown in Figures 3.2 and 3.3 were obtained from these analyses.

These relationships will be applied to stage-discharge data produced by the hydraulic model, in order to delineate flood risk contours, as explained in Section 5.

Table 3.3

ICE SEASON DELINEATIONS AND IDENTIFICATION OF MAXIMUM FLOWS

Year	Period of Ice Cover on Rushoon River					Duration days	Flow (m3/s) (1)	(2)	(3)	(4)
1966 /67	Jan 20-Feb 9	Feb 13-Mar 3	Mar 6-Mar 19			51	5.4	5.4	5.4	23.7
1967 /68	Jan 12-Jan 22	Jan 29-Feb 4	Feb 16-Mar 4	Mar 9-Mar 15		39	21.1	21.1	21.1	23.5
1968 /69	Jan 19-Jan 27	Feb 7-Feb 12	Feb 27-Mar 6	Mar 20-Mar 27		27	5.4	8.7 *	9.7	33.6
1969 /70	Jan 12-Feb 4	Feb 22-Feb 25				26	6.4 *	6.4 *	7.9 *	33.8
1970 /71	Jan 9-Jan 28	Feb 3-Feb 14				30	34.1	34.1	34.1	27.1
1971 /72	Dec 20-Dec 25	Dec 28-Jan 12	Jan 17-Jan 24	Jan 27-Mar 6	Mar 10-Mar 18	74	16.3	16.3	16.3	16.8
1972 /73	Dec 15-Dec 23	Dec 26-Jan 2	Jan 5-Feb 4	Feb 7-Apr 4		102	24.6	24.6	24.6	19.5
1973 /74	Jan 7-Feb 22	Mar 2-Mar 6	Mar 10-Mar 27			67	7.0	11.0	7.0	30.2
1974 /75	Jan 7-Mar 23					75	16.3	16.3	16.3	37.7
1975 /76	Dec 15-Dec 22	Dec 26-Jan 10	Jan 12-Jan 19	Feb 6-Feb 24	Mar 3-Mar 21	65	14.6	14.6	14.6	23.5
1976 /77	Dec 13-Dec 14	Jan 3-Jan 7	Jan 14-Jan 28	Feb 1-Mar 24		70	12.8	12.8	12.8	24.1
1977 /78	Dec 22-Dec 23	Jan 7-Jan 10	Jan 13-Jan 14	Jan 22-Jan 25	Feb 4-Mar 24	56	10.5	10.5	10.5	26.9
1978 /79	Jan 6-Jan 8	Jan 13-Jan 24	Feb 11-Mar 6			36	9.5	9.5	6.1 *	25.1
1979 /80	Dec 20-Dec 26	Jan 6-Jan 20	Jan 25-Mar 10	Mar 14-Mar 23		74	12.7	12.7	12.7	27.2
1980 /81	Dec 23-Dec 25	Jan 6-Jan 10	Jan 17-Feb 8			28	3.8	3.8	10.2	24.6
1981 /82	Jan 13-Jan 14	Jan 20-Mar 9				49	6.7	6.7	6.7	16.1
1982 /83	Jan 9-Jan 10	Jan 21-Feb 5	Feb 11-Mar 3			36	17.5	17.5	17.5	35.3
1983 /84	Jan 2-Jan 6	Jan 13-Feb 5	Feb 10-Feb 25	Mar 6-Mar 17		53	20.0	20.0	20.0	30.4
1984 /85	Jan 9-Apr 27					109	11.9 *	11.9 *	11.9 *	--

(1) Study Criteria Breakup Flows
(2) Breakup Flow for Qb >= 3Qf
(3) Breakup Flow from Garnish Indication (maximum during breakup)
(4) Non-Ice Season Peak Flow (open water)
(5) Instantaneous maximum flows estimated from stream flow data measured on Garnish River, Pipers Hole River or Rattle Brook by applying correlation equations, 3.1, 3.2 or 3.3 . Method for identifing break up flows given in Volume 2, Section III.
* Maximum Ice Season Flow, No Breakup

3.5 Observations

In most of the years examined breakup was due to hydraulic conditions where large flow increases were observed in flow records at both Garnish and Piper's Hole Rivers (or Rattle Brook after 1980). However, in a few years the analysis showed that thermal conditions would have governed and that the ice cover would have disappeared without the occurrence of an ice jam flood such as was observed in 1985. In about three years out of the sample period it was less clear whether thermal decay did or did not proceed flow peaks. In these situations it was assumed that breakup was due to hydraulic conditions.

As noted in Sub-section 3.3.2, the study criteria were found to agree closely with backwater indications shown on stream flow records for neighbouring gauging stations on the Burin Peninsula and with recorded flood events.

As a further check on the reliability of the procedures used to identify ice season periods and to select breakup flow peaks, two alternative selection procedures were examined:

(i) assuming that breakup would be indicated when:

$$Q_B > 3\ Q_f\ *$$

(ii) basing ice season indications soley on reports of backwater conditions recorded in the streamflow records for Garnish River.

---

* Based on inspection of flow records from Garnish River.

3.5 Observations (Cont'd.)

When frequency analyses were carried out on the data sets selected by these alternative procedures,(as tabulated in Table 3.3), the resulting predictions of Q_{20} and Q_{100} were not found to differ significantly from predictions based on the study criteria, as shown in Table 3.4.

TABLE 3.4: Comparison of Alternative Design Flood Determinations

Delineation Procedure	Ice Season Flood Peaks	
	Q_{20}	Q_{100}
Study Criteria:	30.3 m^3/s	45.3 m^3/s
Study Criteria with $Q_B > 3Q_f$:	30.0 m^3/s	43.6 m^3/s
Garnish River ice indications:	28.3 m^3/s	39.9 m^3/s

As a matter of interest a comparison has also been made between the 1 in 20 years and 1 in 100 years return period floods on Rushoon Brook as determined by:

(i) frequency analysis of the annual maximum floods tabulated in Table 3.3, and

(ii) RFFA formulae (5).

This comparison is given in Table 3.5.

3.5 Observations (Cont'd.)

TABLE 3.5 COMPARISON OF ANNUAL FLOOD PEAKS

Return Periods	Annual Flood Peaks	
	Study Estimate	RFFA Formulae
1 in 2 years ($\leq$ 50%)	26.3 m^3/s	28.9 $\pm$ 6 m^3/s
1 in 20 years($\leq$ 5%)	39.7 m^3/s	50.0 $\pm$ 10 m^3/s
1 in 100 years($\leq$ 1%)	48.6 m^3/s	63.8 $\pm$ 14 m^3/s

The estimates obtained by the RFFA formulae were generally higher than the study estimate by 10%-25% (this difference is approximately equal to the standard error of estimate for the RFFA formulae). However, this comparison does not constitute an independent check on the study determinations because the proration factor used to transpose Rattle Brook flows to Rushoon Brook was itself determined using a RFFA formula.

The study procedures have, for the most part, preserved the pecularities of Garnish River flows, which form the predominant parameter in the correlation equations 3.2 and 3.3 and from which fourteen of the eighteen data points used in the study were derived.

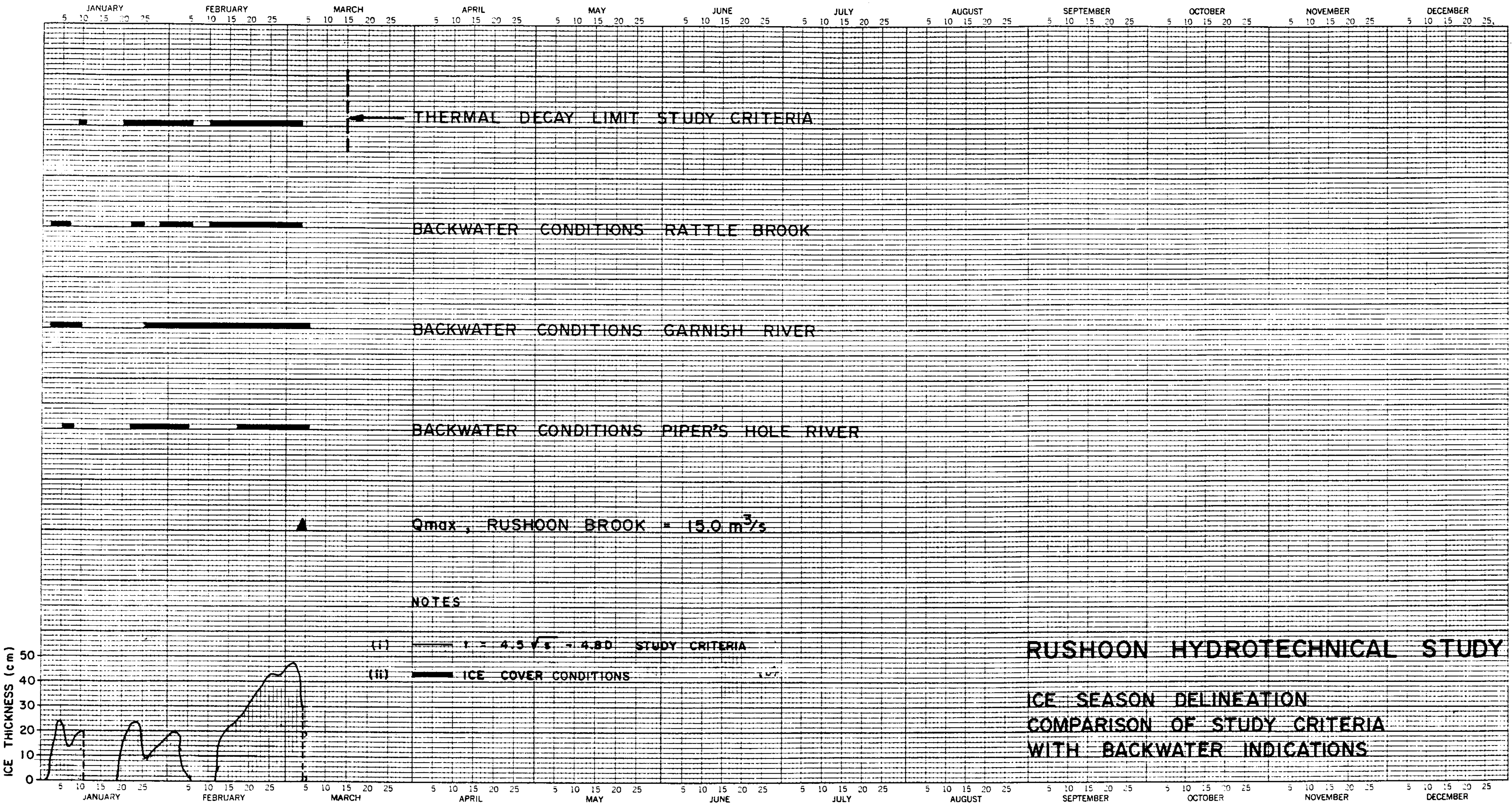
3.6 Conclusions

(i) Ice season frequency analyses were found to be relatively insensitive to the methods used for delineating the ice season periods and selecting ice season flow maxima (see Table 3.4).

(ii) Additional flow measurements on Rushoon Brook are required to verify the proration factor used for transposing flows from Rattle Brook to Rushoon Brook.

FIGURES

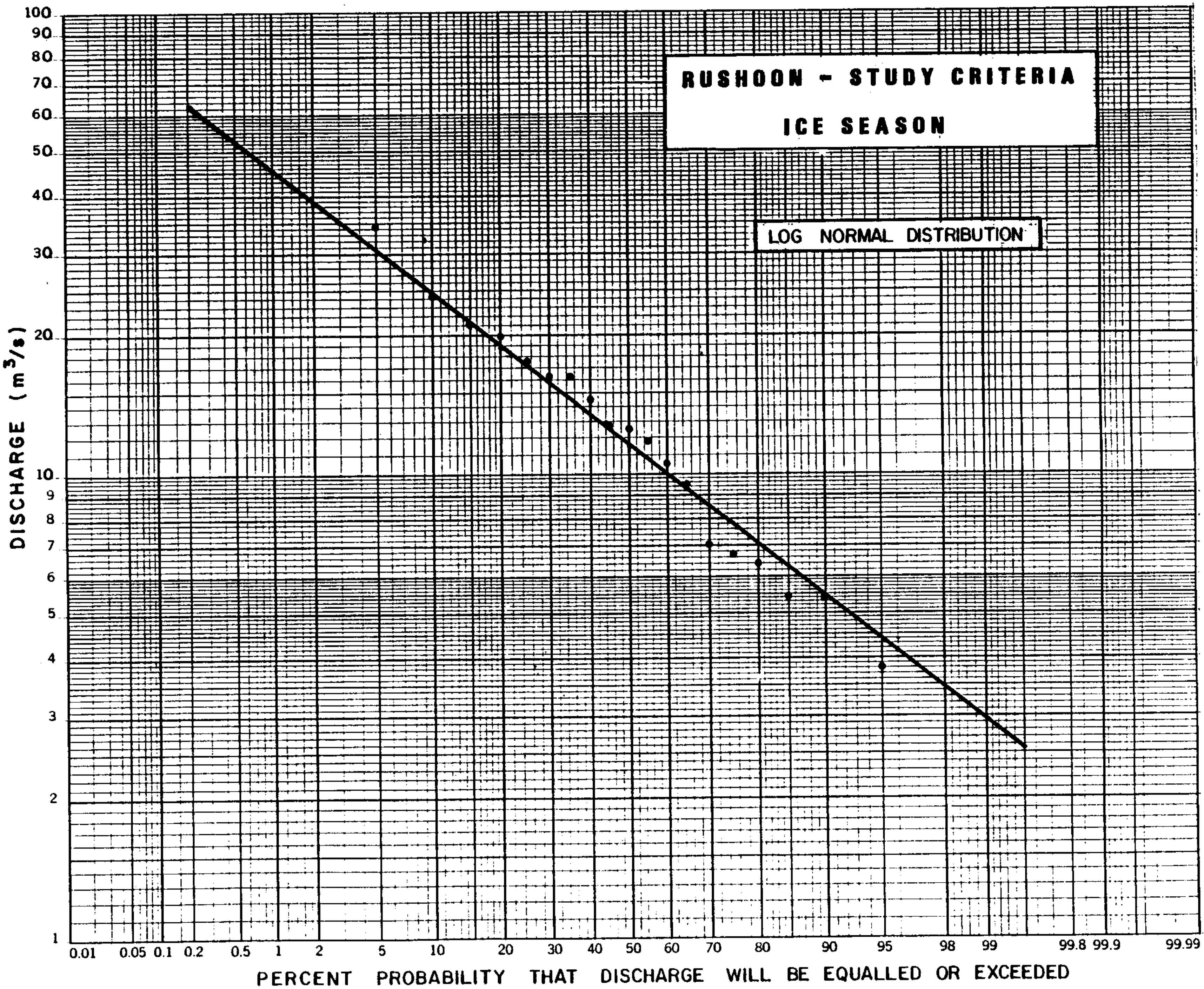
FIGURE 3 - I



RUSHOON HYDROTECHNICAL STUDY

ICE SEASON DELINEATION
COMPARISON OF STUDY CRITERIA
WITH BACKWATER INDICATIONS

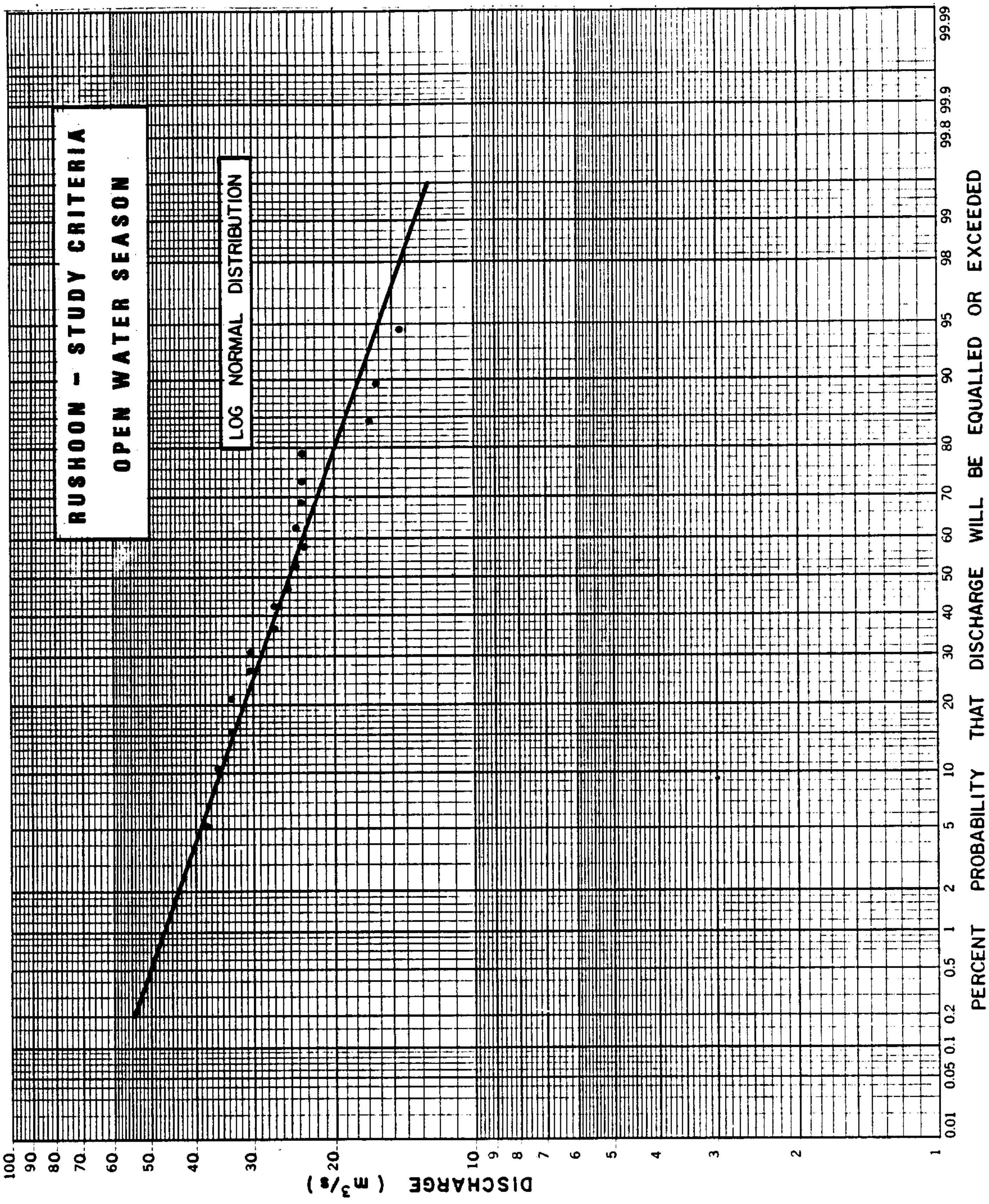
1 9 8 3



RUSHOON - STUDY CRITERIA
ICE SEASON
LOG NORMAL DISTRIBUTION
DISCHARGE (m3/s)
1
2
3
4
5
6
7
8
9
10
20
30
40
50
60
70
80
90
100
0.01
0.05
0.1
0.2
0.5
1
2
5
10
20
30
40
50
60
70
80
90
95
98
99
99.8
99.9
99.99
PERCENT PROBABILITY THAT DISCHARGE WILL BE EQUALLED OR EXCEEDED

FIGURE 3.2

FIGURE 3.3


RUSHOON - STUDY CRITERIA
OPEN WATER SEASON
LOG NORMAL DISTRIBUTION
DISCHARGE (m^3/s)
PERCENT PROBABILITY THAT DISCHARGE WILL BE EQUALLED OR EXCEEDED
100 90 80 70 60 50 40 30 20 10 9 8 7 6 5 4 3 2 1
0.01 0.05 0.1 0.2 0.5 1 2 5 10 20 30 40 50 60 70 80 90 95 98 99 99.8 99.9 99.99

4.0 HYDRAULIC STUDIES

4.1 Purpose

All the major floods that have been observed on the lower portion of Rushoon Brook appear to have been the result of water backing up behind ice jams.

The following section of this report deals with the hydraulic characteristics of Rushoon Brook, for open water and ice season conditions with particular emphasis on floods associated with ice jamming.

4.2 Features of Rushoon Brook

Features of Lower Rushoon Brook which govern its hydraulic behaviour are described below:

(i) from Seaward Bridge in Rushoon Harbour, a tidal pool approximately 100 to 150 m wide and 1.0 m average depth extends upstream to Pete Moores Island. Mean tidal fluctuations in the Pool range between approximately $\pm$ 0.8 m (G.S.C.) with extreme high and low spring and neap tides attaining $\pm$ 1.1 m (G.S.C.).

(ii) just above Pete Moores Island near the tidal limit at Salmon Hole Point, a rock outcrop causes an abrupt channel constriction to 30 m. The narrowing of the brook at this point and the shallow water adjacent to Pete Moores Island have been reported to be the key elements associated with ice jam formation.

(iii) from Salmon Hole Point to Rough Rocks, the upstream limit of the study area, the brook has a bed gradient of 6.7×10^{-3} m/m and an average channel width of 20 m. Flow in this reach is supercritical, i.e. Froude No. $<$ 1.0.

In order to investigate the hydraulic behaviour of Rushoon Brook, both open water and ice season conditions were simulated, over a range of flows, using a computer model developed by Lasalle Hydraulic Laboratory. For open water conditions only the backwater routines of this model were required. For ice season conditions, on the other hand, it was necessary to simulate, freeze-up, mid-winter thermal ice thickening and breakup. The model was set up utilizing topographic and hydrologic field data and ice observations taken during the winter of 1984/85.

4.3 Setting-up Model

4.3.1 Description of Model

The computer model (henceforth, ice model), used for the Rushoon Study was developed at Lasalle Hydraulic Laboratory based on studies done by Pariset and Hausser in the late 50's for the old Quebec Hydro Commission.

The results of these studies were the subject of two papers by Pariset and Hausser and Pariset, Hausser and Gagnon (References 11 and 12).

The calculation procedures developed by Lasalle were based almost entirely on the hydraulic/mechanical phenomena which take place as individual non-cohesive ice particles come down a river, meet an obstruction, and build up a cover. Implicit in this approach is the assumption that an adequate supply of ice is available to satisfy the ice cover equilibrium criteria for whatever discharge is selected. In fact, this corresponds to calculating the highest level at each section; if more ice arrives, either the cover will progress in the up-stream direction or the ice will be entrained under the cover downstream. The level at the given section can be exceeded only if ice transport downstream creates a higher cover at another section further downstream whose backwater drowns the original section.

During the freeze-up, a thin accumulation cover builds up as a mass of non-cohesive ice pieces, much of it in the form of frazil slush rather than as discrete, hard ice. As winter progresses, cold from the surface penetrates the cover, gradually freezing the non-cohesive particles into the mass of the thermal cover down to the thickness determined by the degree-days of frost. In the spring, this whole cover is lifted and broken up by the rising discharge, furnishing a large supply of ice debris, broken pieces of solid ice, that become available to form ice jams.

A mathematical model developed at LaSalle by Pariset and Hausser was used to simulate this pattern of ice formation, thermal thickening and break-up and has been used with success on many projects in the past. Thus, a computer program utilizing these proven techniques was used to simulate the ice regime in Rushoon Brook.

4.3.1 Description of Model (Cont'd)

As stated previously, ice formation in rivers is generally caused by the progressive build-up of an ice cover composed of individual ice floes accumulating on the river surface upstream from an obstacle or lodgement site. The thickness of the cover is dependent on the velocity of the flow, low velocities producing thin covers, high velocities thick ones. Where the velocity is close to zero or zero such as in a lake, the ice thickness will be equal to the thermal thickness.

Ice studies carried out at Lasalle Hydraulic Laboratory revealed that rivers could be categorized as "wide" or "narrow" depending on the forces acting.

For both categories, the leading edge of the cover progresses due to the accumulation of incoming floating ice with a thickness (t) is obtained by solving equation 4.1.

$$\frac{V}{\sqrt{2gH}} = \sqrt{\frac{\rho-\rho^1}{\rho}\frac{t}{H}}\left(1-\frac{t}{H}\right) \qquad (4.1)$$

where: V = is the mean flow velocity upstream of the leading edge m/s

H = the mean depth, m. Defined by H = A/B where A is the cross-section area and B the surface width

p = the specific gravity of the water;

p'= the specific gravity of the ice;

g = acceleration due to gravity m/s/s;

t = thickness of ice m.

A schematic representation of these parameters is given in Figure 4.1.

4.3 Setting-up Model (Cont'd)

4.3.1 Description of Model (Cont'd)

As the cover lengthens, the increasing hydrodynamic load forces on the cover must be balanced by the internal resistance created by the buoyancy of the accummulated ice and the river bank reactions.

For "narrow rivers" the "leading edge thickness", as obtained from equation 4.1, is sufficient to resist the applied external hydrodynamic loads and no further thickening of the ice cover occurs.

For "wide rivers", external forces increase beyond the internal resistance governed by the "leading edge thickness", with the result that the edge shoves and the cover thickens until its resistance increases sufficiently for the cover to become stable. The "wide river" category is normally found to govern for real rivers.

Experiments, confirmed by field observations, have shown that an ice cover will remain stable if certain criteria are maintained. This condition is shown in the bell shaped curve in Figure 4.2. The ratio of ice thickness, t, divided by the depth, H, is shown as a function of:

$$\frac{Q^2}{C^2 B H^4} \tag{4.2}$$

where:

Q =	discharge	m^3/s
C =	Chezy roughness coefficient	$m^{1/2}/s$
B =	surface width	m
H =	depth of flow	m

4.3.1 Description of Model (Cont'd)

For points which fall within the bell shaped curve the cover is stable; hence some latitude in ice thickness, depth or discharge is possible for a stable cover. However, when the edge of the bell is reached, an ice shove, increase or decrease in water level will occur to compensate for the new condition (12), see Figure 4.2.

These calculation methods utilize the characteristic dimensions of each individual cross-section and the distances in between them as primary input. A backwater curve is subsequently established to link the cross-sections in the ice calculations, providing a strong degree of control on the results.

After a stable accumulation cover has formed, it starts to thicken thermally at a rate proportional to the freezing degree days and as it thickens and solidifies the ice porosity is reduced from approximately 0.6 to 1.0. Over the winter period, water levels fall and the cover sags causing hinge cracks to form along the edges. These cracks and others, formed by thermal contraction of the ice sheet, fill with water which subsequently freezes. By the end of the winter period, the combination of these factors will produce an ice cover with cohesive strength. It has been found by experiment and field observations that this cohesive strength is a function of the thermal ice thickness.

In the mathematical model, the thermal ice thickness can be entered either as a measured value or calculated as a function of the freezing degree-days from the date the accummulation ice cover has formed.

4.3 Setting-up Model (Cont'd)

4.3.1 Description of Model (Cont'd)

Ice cover break-up is generally caused by a combination of increased discharge, and consequently stage, thermal ice weakening and melting although either effect may be enough to remove an ice sheet from a river.

During spring, a combination of warming air temperatures and solar radiation produce melting of the snow pack that cause river discharges to rise, signalling the start of break-up. The ensuing run-off causes water levels to increase, the ice cover to lift and shore leads to form.

As the discharge increases, a point is reached when the cover becomes hydro-mechanically unstable and ruptures.

A break-up model was developed at Lasalle Hydraulic Laboratory incorporating the above logic. The rupture flow for the ice cover during the spring break-up at each cross-section can be calculated by equation (4.3) using results obtained from the ice cover freeze-up/mid-winter program, together with the given cross-sections and hydraulic parameters for each section.

$$\frac{Q^2}{B_{TOT}C^2} = \frac{\left[K_{coh}\, H_1^2 + 0.094 H_1^4 \left(\frac{t_1}{H_1}\right)^2\right]\left(1 - 0.92\,\frac{t_1}{H_1}\right)^3}{1 + 0.92\,\frac{t_1}{H_1}} \qquad (4.3)$$

Where:
- Q = mean daily discharge (m^3/s);
- B_{TOT} = total surface width of section under the ice cover (m);
- C = Chezy roughness coefficient ($m^{1/2}/s$);
- H_1 = mean depth (m);
- t_1 = thermal ice thickness (m);
- K_{coh} = cohesive shear strength factor (m^2).

4.3.1 Description of Model (Cont'd)

If the right hand side of the equation is larger than the left, the cover is stable. If not, instability has occurred.

The cohesive shear strength factor (Kcoh) used in the preceding equation was derived from the original work done on the St. Lawrence River, where for a break-up thermal ice thickness of 0.6 m, an effective ice shear strength per metre of 1.32 kN/m was observed. Subsequent studies have shown that this value appears to vary linearly with respect to the thermal ice thickness giving the following expression:

$$Kcoh = \frac{2\tau t_{th}}{\rho g} \qquad (4.4)$$

where: Kcoh = cohesive shear strength factor (m^2);
τ = ice shear stress of 2.2 kPa;
t_{th} = thermal ice thickness at break-up (m);
g = acceleration due to gravity, m/s/s;
ρ = density of water (kg/m^3).

Equation 4.4 has been developed empirically based on observations on other rivers in Canada, and ideally should be calibrated for each river on which it is applied. However, since adequate quantitative ice observations are not available on Rushoon Brook, the use of a τ value derived from other locations means that results of the rupture calculations should be interpreted with care.

4.3.1 Description of Model (Cont'd)

Maximum break-up water levels are determined at each cross-section as a function of the maximum discharge that occurs at rupture. Broken ice debris from upstream pile against the upstream edge of the unbroken ice cover and if sufficient ice is available a stable accummulation cover will develop upstream, the thickness being dependent on the river discharge.

The flow charts showing the calculation procedures are given in Volume 2, Appendix IV.

4.3.2 Model Set-up and Calibration

Model set-up, calibration and verification was done in successive steps as follows:

(a) input topographic data and initial estimates of bed roughness.
(b) calibrate and verify against open water data by varying assumed values for bed roughness.
(c) calibrate and verify against ice jam data by varying assumed value for composite ice-bed roughness.

Cross-sections of the Rushoon Brook were selected and surveyed at twelve points between Seaward Bridge and Rough Rocks. Care was taken to choose sections that were representative and at suspected ice keying locations, Figure 4.3.

An initial estimate of the Manning "n" value of bed roughness to be about 0.04 was made by F. Parkinson of Lasalle Hydraulic Laboratory from an inspection of photographs and observations on site.

4.3.2 Model Set-up and Calibration (Cont'd)

Mean sea level, 0.0 m (G.S.C.) in Rushoon Bay was assumed in all simulations except for one series in which effects of tidal variations were studied (Section 4.5).

Two observed open water surface profiles were available with which to calibrate the model for bed roughness. One profile was taken with a discharge of 0.21 m^3/s while the other was taken with a "bank full" flow of 16.5 m^3/s.

First runs on the computer made using the two above discharges and the Manning "n" of 0.04 gave water levels that were too low. The Manning "n" value was gradually increased until a satisfactory profile was obtained with n = 0.05, see calibration and verification comparisons, Figure 4.4. The use of a single "n" value gave good agreement with observed levels in the portion of the river of concern, between Cross-Section 5.1 (ch. 1+040) and Cross-Section 9 (ch. 0+660). Less satisfactory agreement was noted in the vicinity of ch. 0+500 and ch. 0+300. Since these locations were outside the flood prone area, and hence of little practical interest to the purposes of the study, further refinements to obtain a better fit were not deemed to be worthwhile.

4.3.3 Open Water Profiles

With mean sea level (0.03 m, G.S.C.) as the start point at Seaward Bridge, open water surface profiles were then calculated for discharges of 5, 10, 15, 20, 25, 30, 40, 50 and 60 m^3/s and stage discharge curves derived at each cross-section (Figures 4.5 and 4.6).

4.3.3 Open Water Profiles (Cont'd)

Backwater calculations follow the procedures shown in the flow chart "Remous en eau libre", ("Backwater for Open Water Conditions"), Volume 2, Appendix IV.

4.3.4 Freeze-up Levels

Inspection of published Water Survey of Canada (W.S.C.) discharge data for Rattle Brook, an adjacent catchment area of similar size to Rushoon, showed that at freeze-up in December, discharges vary from 2 to 4 m^3/s. From this data, an average freeze-up discharge of 3 m^3/s was chosen for Rushoon Brook.

Backwater profiles with a thin accummulation ice cover were then calculated from the Seaward Bridge upstream, following the procedures shown in the flow chart "Formation d'un convert mince", ("Formation of a Thin Ice Cover"), Volume 2, Appendix IV. At Cross-Section 6, (see Figure 4.3) and above, however, it was found to be impossible to get an ice cover to form regardless of the tide level downstream. The supercritical velocities encountered were found to be too high. On site observations tend to confirm that in fact, the ice cover is not uniform but develops through a complicated process of water overtopping, snow accumulation, border and anchor ice growth. By these means the ice cover is able to progress beyond Cross-Section 6 during extended periods of cold weather, as was noted during the winter of 1984/85 (3).

4.3 Setting-up Model (Cont'd)

4.3.5 Mid-Winter Conditions

As previously mentioned, during the winter, the ice cover consolidates and strengthens due to thermal penetration. Thus, mid-winter simulations must take into account thermal ice growth and the development of cohesive strength. This was handled in the ice model by imposing a typical mid-winter ice thickness of 0.4 m,* at sections where the thin ice accummulation was less than this value, and by introducing a cohesive shear strength factor (Kcoh) computed from equation 4.4. Then the backwater profile and stability of each section was recalculated following the procedures shown in the flow chart "Debacle avec cohesion" ("Breakup with Cohesion"), Volume 2, Appendix IV. Once again, regardless of tide level, it was found that an ice cover would not form upstream of Cross-Section 6.

---

* Ice thickness at breakup, as determined in Volume 2, Appendix III, gave an average value of t 0.4 m, while a Chezy "C" value of 35 m /s was estimated for roughness with an ice cover.

4.3.6 Break-up

The breakup process was simulated by taking the mid-winter freeze-up profile and section parameters and then gradually increasing the discharge until rupture occurred, see flow chart "Debacle avec cohesion", ("Breakup with Cohesion"), Volume 2, Appendix IV. From the results of this simulation, the computer selects either the first upstream section ruptured or open water section in the profile and then tries to calculate a full depth, stable accummulation cover.

(Computer printouts for open water backwater calculations, ice rupture break-up profiles and ice accummulation cover break-up profiles are shown in Volume 2, Appendix V).

4.3.7 Model Calibration

Inspection of the computer printouts show that for all discharges and tide levels, Cross-Sections 6 to 12 always ruptured together with Cross-Section 4 or both 4 and 5. To obtain a better definition of conditions at Salmon Hole Point, a duplicate Cross-Section 5 was fabricated and added 20 m downstream of the original Cross-Section 5. The original Cross-Section 5 was retained and renumbered Cross-Section 5.1 (see Figure 4.3). Cross-Section 5.1 at Salmon Hole Point held firm until a discharge of between 40 to 45 m^3/s was reached.

Normally, when an accummulation cover forms at the end of a fast flowing brook like Rushoon, the broken ice pieces not only build up upstream of the leading edge but also

4.3.7 Model Calibration (Cont'd)

tend to pack under the existing thermal cover downstream. The first calculations based on the above premise showed that this could not have happened at Rushoon. More ice than was available from upstream would have been needed to satisfy the cover characteristics. To meet the requirements of the ice jam that formed in 1983, nearly all the available ice would have been held upstream of Salmon Hole Point. This would require almost a complete blockage of ice transport; not only would ice from upstream be retained by the unbroken cover at Cross-Section 5.1, but entrainment underneath the cover would be prevented by a "grounded" cover resulting from a falling tide or because the cover was frozen solid to the bottom. Reports from "on site" observers generally confirm this hypothesis, which is also supported by photographs taken of the 1983 ice jam. At the head of this jam, water was described as passing under, through and around the ice fender wall constructed along the river indicating a flow detour. The photographs showed little or no evidence of level build-up downstream of Salmon Hole Point due to ice accummulations.

Final calibration of the model was achieved by simulating the ice jam conditions observed in the 1983 ice jam flood. This involved a series of simulation runs with a flow of 15 m^3/s (flow on Rushoon Brook during this event as estimated from Rattle Brook flow data) in which model Chezy "C" values were varied until a good fit between calculated and observed data was obtained. The Chezy "C" value giving the best fit was found to be 35 $m^{1/2}/s$. The

4.3.7 Model Calibration (Cont'd)

results from this run are given in Figure 4.7. As can be seen in this figure, the water level profile between Cross-Sections 5.1 and 9 matches almost exactly the observed profile except for a small deviation upstream of Cross-Section 8.

Additionally, the calculated ice volume of 20,500 m^3 can be supplied from available ice upstream of the jam - a volume of some 36,000 m^3 between Salmon Hole Point and the natural ice bridge at "The Falls", see Field Report - Table III-7 (2).

4.3.8 Model Verification

Supplementary data for model verification is limited to two reports of flood levels from the February 1973 flood. These points plot slightly below the calculated profile based on the estimated daily mean flow (21.4 m^3/s) for this event - see Figure 4.8. Given the uncertainties associated with these water levels as recalled by residents, and estimation of the flood flow, these results provide satisfactory verification of the accuracy of the ice model.

However, when the composite Manning "n" is estimated from the computer simulation results, based on Chezy "C" value of 35, an "n" value of 0.022 is found. This value is obviously too low considering that the open water bed roughness is 0.05 and should be even higher with an ice cover. This leads to the premise that the actual discharge under the jam is probably very small with the majority of the flow passing through or overland around

4.3.8 Model Verification (Cont'd)

the jam. This in fact was observed in the 1983 flood which probably accounts for the difference between the water level elevations observed and calculated at Cross-Section 8. Here, nearly all the flow would be retained within banks and the low roughness value assumed would result in levels slightly lower than observed. The complexity of this type of flow distribution is beyond the scope of the ice model and therefore the reduced Chezy "C" of 35 with the total average flow values was retained in the calculation procedures.

In spite of these theoretical shortcomings, model predictions have been found to be satisfactory for similar conditions on other rivers; moreover, both calibration and verification simulations, Figures 4.7 and 4.8, compare well with field observations thus confirming the utility of the model.

4.4 Stage Discharge Curves

In order to develop a set of stage discharge curves the same complete blockage characteristics for ice at Salmon Hole Point were assumed and ice jam simulation runs were made for discharges of 10, 15, 20, 25, 30, 35, 40, 45 and 50 m^3/s. The results have been plotted in Figures 4.5 and 4.6 which also include stage discharge curves for thin ice and open water conditions.

Figures 4.5 and 4.6 show that water levels at Cross-Sections 5.1, 6, 7, 8 and 9 progressively increase

4.4 Stage Discharge Curves (Cont'd)

until the cover at Salmon Hole Point releases at a discharge of 40 m^3/s and the ice jam moves downstream to Cross-Section 3. Under this condition the water level rises at Cross-Sections 3 and 4 but falls at Cross-Sections 5, 6, 7, 8 and 9.

For each discharge condition simulated, the volume of ice upstream of Section 5.1 was calculated and plotted. The results, given in Figure 4.9, show that as the discharge increases, the ice jam thickens and increases in volume, causing the water level to rise as previously mentioned. A limit is reached when the ice jam has a volume equal to the volume of ice available from upstream.

4.5 Significance of Tidal Effects

Residents in the Community of Rushoon have reported that ice jams are more likely to form when the tide is low. To test this hypothesis the ice model was run assuming a high tide level of 1.1 m at Cross-Section 1 and a floating ice cover in the Tidal Pool.

It was found that under these conditions ice would build up under the cover downstream of Seaward Bridge (assuming the Bay was frozen over) and that more than 97,000 m^3 of ice would be required to produce the water profiles observed in the 1983 flood, see Vol. 2, Appendix V. This contrasts with the conditions on a falling tide. For this case, ice model simulations showed ice jams forming initially against the leading edge of the unbroken Tidal Pool cover; however, entrainment of ice pieces under the cover was prevented by grounding of the unstable portion of the ice cover between Pete Moores Island and Salmon Hole Point.

4.5 Significance of Tidal Effects (Cont'd)

Model simulations thus indicate that the ice jam at Salmon Hole Point is sensitive to tide level as suggested by observers familiar with the area.

4.6 Flood Surges

Dramatic flood surges of ice and water have been observed on Rushoon Brook during breakup. These surges are produced by collapse of ice jams formed upstream of the Study Area, notably the jam which forms in the vicinity of Rough Rocks.

It has been suggested that maximum surge levels might in fact exceed the levels produced by ice jams. This phenomenon cannot, at this time, be simulated by the ice model. However, Henderson and Gerard (13) suggest that ice jam surges can be analyzed by the usual equations governing open water surges ignoring the presence of ice. Although an investigation of the significance of ice surges lies outside the Terms of Reference of this Study, an approximate analysis has been attempted using a method described by Hagen (14). This method gives an approximate solution to the dam break problem. In applying this method to Rushoon Brook, it was assumed that a flood surge would be produced by the sudden collapse of an ice jam at Rough Rocks retaining a head of 2 metres and a volume of 3000 m^3 of water.

The estimated surge flow from such an ice jam collapse was found to be 50 m^3/s at Section 8, the point at which a flood surge might enter the Community. The corresponding water level at this point was estimated to be 4.0 m, which would be just sufficient to drive water and ice over the original river bank and in among the houses, if the fender wall was not there. This compares with water levels of 4.4 m and 4.9 m observed in this area in the 1983 and 1973 floods, and levels of 5.2 m and 5.6 m for the 1 in 20 year and 1 in 100 year floods respectively. Hence, it appears unlikely that the levels resulting from flood surges would exceed the levels produced by significant ice jams.*

It should be noted, however, that flood surges of this type containing large chunks of ice can be very destructive and thus any new structures (water intakes, bridge piers, etc), built in the river channel should be designed with this in mind.

4.7 Sensitivity of Water Level Predictions

The sensitivity of the model results with an ice jam was then checked by varying the Chezy "C" roughness coefficient. Computer runs were made with Chezy "C" values of 25, 30, 33, 35 (used for calculations), 38 and 40 and a flow of 15 m^3/s. The results are given in Table 4.1.

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* These results have been further confirmed by calculations carried out by Dr. S. Beltaos of the National Water Research Institute, Burlington, Ontario.

TABLE 4.1 SUMMARY OF SENSITIVITY RUNS SHOWING VARIATIONS IN W.L. AND ICE VOLUMES FOR A RANGE OF CHEZY "C" VALUES & Q = 15 m^3/s

Cross-Section	Water Levels in m (GSC)					
	C=25	C=30	C=33	C=35	C=38	C=40
5.1	3.38	3.25	3.19	3.16	3.08	2.94
6.0	3.79	3.65	3.60	3.56	3.47	3.29
7.0	4.29	4.09	4.03	4.01	3.92	3.61
8.0	4.77	4.41	4.38	4.37	4.21	4.01
9.0	5.13	5.00	4.93	4.89	4.84	4.81
Ice Volume m^3	24,317	21,337	20,796	20,500	17,140	14,935

As shown, over the range of the roughness coefficients selected (Chezy 25 to 40), water level changes vary from between a maximum of 0.76 m at Cross-Section 8 to a minimum of 0.32 m at Cross-Section 9. Ice volumes in the jam varied between 24,300 and 14,900 m^3.

In the mathematical model, C=35 was originally chosen from experience gained with other studies. This value was found to give good results when used in the simulations of the 1973 and 1983 ice jam events so was retained for the other ice cover discharge runs.

Sensitivity analyses of the results of this study are discussed at greater length in Section 5.

4.8 Summary of Results

After initial setting up of the ice model it was calibrated and verified for open water condition by varying the value of Mannings "n". When this calibration was completed a set of backwater computations was carried out over a flow range of 0 to 50 m^3/s. The results of these runs give water level flow relationships at each cross-section, as summarized in Figures 4.5 and 4.6.

Following the investigation of open water conditions the ice model was then used to simulate ice season conditions. These simulations provided an accurate description of the 1983 ice jam event, reproducing the water level profile, ice jam thickness and length as observed "on site" for a discharge calculated to be 15 m^3/s and also checked closely with data on the 1973 flood, flow = 21.4 m^3/s, used for model verification. The results show that the ice cover at Salmon Hole Point is very resistant and forms an obstruction to the floating ice debris that arrives from upstream. A jam forms at this point when this ice cannot be transported further downstream under the Tidal Pool ice cover which grounds at low tide. At high tide, when this ice cover floats, sufficient room beneath the cover is provided to allow broken ice to be transported under the cover, which explains why ice jams are more likely to occur at low or falling tides.

Computer runs were then made to determine flood levels that would occur at higher discharges with an ice jam at Salmon Hole Point. The results showed that the ice cover here would remain stable until a discharge of 40 m^3/s is reached. Above this value, the ice would rupture and the complete ice mass move downstream of Pete Moores Island to Section 3 (see Figure 4.5).

Stage discharge curves at each section were prepared from the results of these runs for thin ice cover and thick accummulation (ice jam) cover conditions and spanning a flow range of 5 to 50 m^3/s, as shown in Figures 4.5 and 4.6.

A series of runs were then carried out to test the sensitivity of water level prediction to changes in model parameters. These results showed that minimal changes in water level prediction would result over the range of normal Chezy "C" values observed in other river studies.

The model was also used to test the effectiveness of two suggested remedial measures:

- channel improvements, and
- ice storage weirs

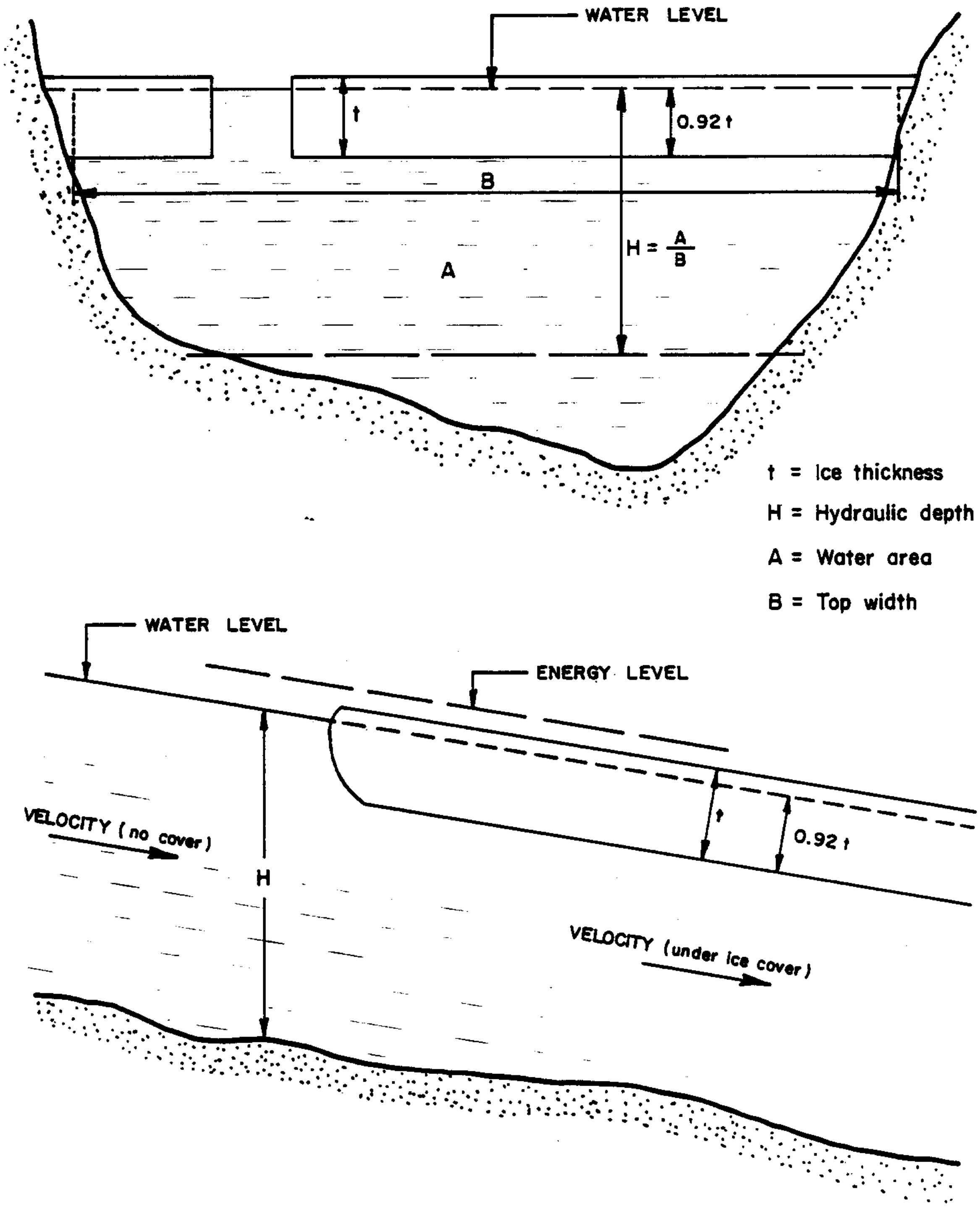
as explained in Section 6.

4.8 Summary of Results (Cont'd)

Finally, the significance of flood surges as a cause of flooding in the Study Area was assessed by an approximate method. It was concluded that it would be unlikely that flood levels sufficient to cause serious flooding in the Community would result from flood surges.

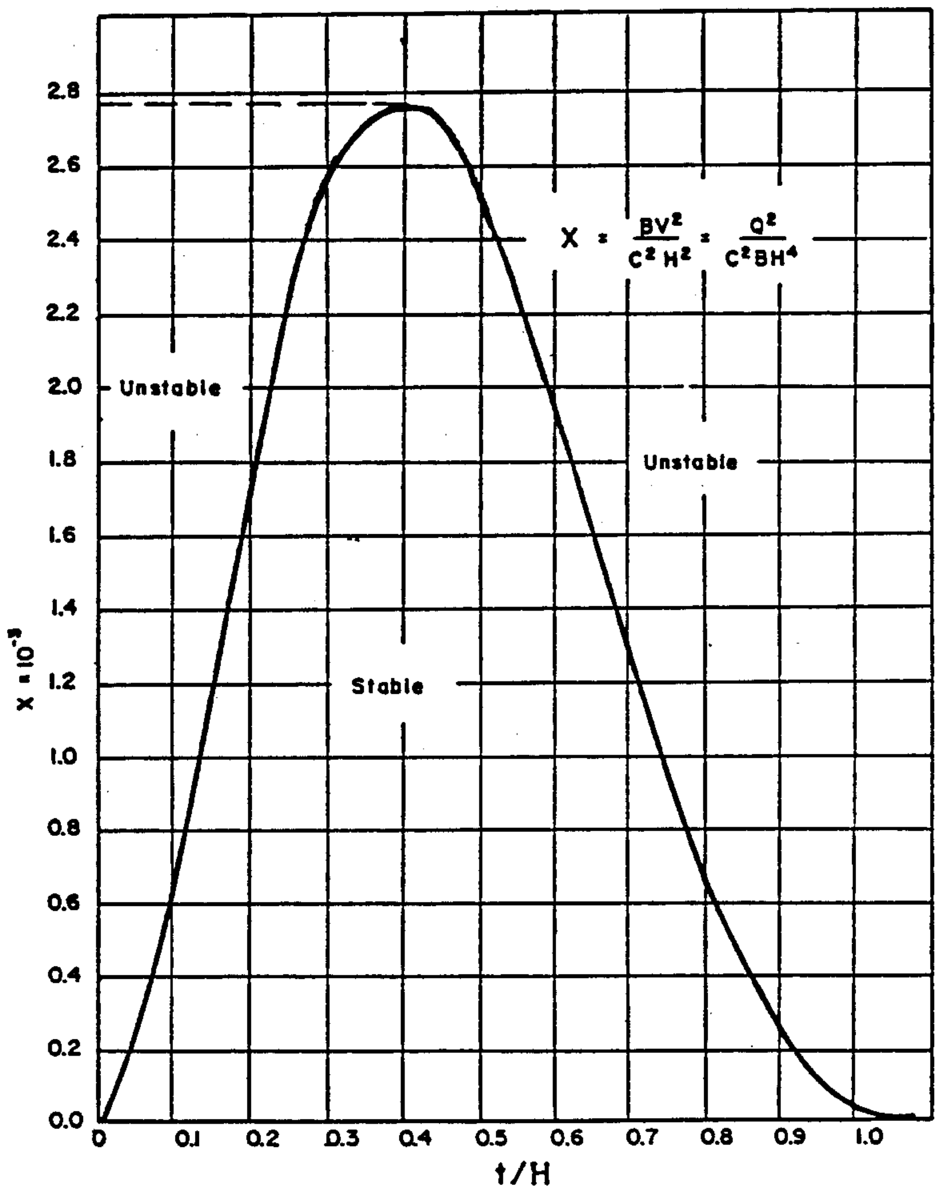
LIST OF FIGURES

FIGURE 4.1



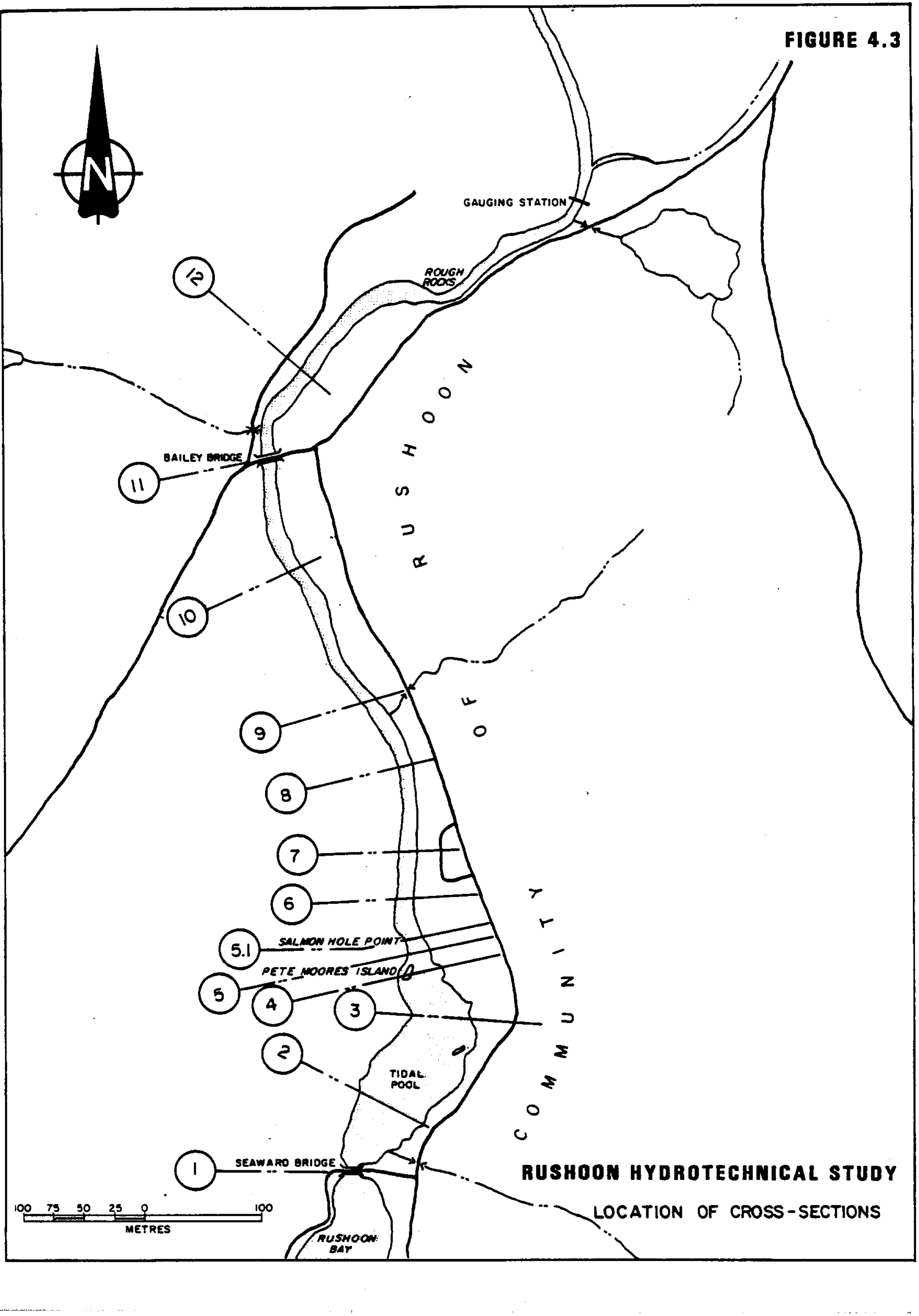
RUSHOON HYDROTECHNICAL STUDY

HYDRAULIC AND SECTION PARAMETERS



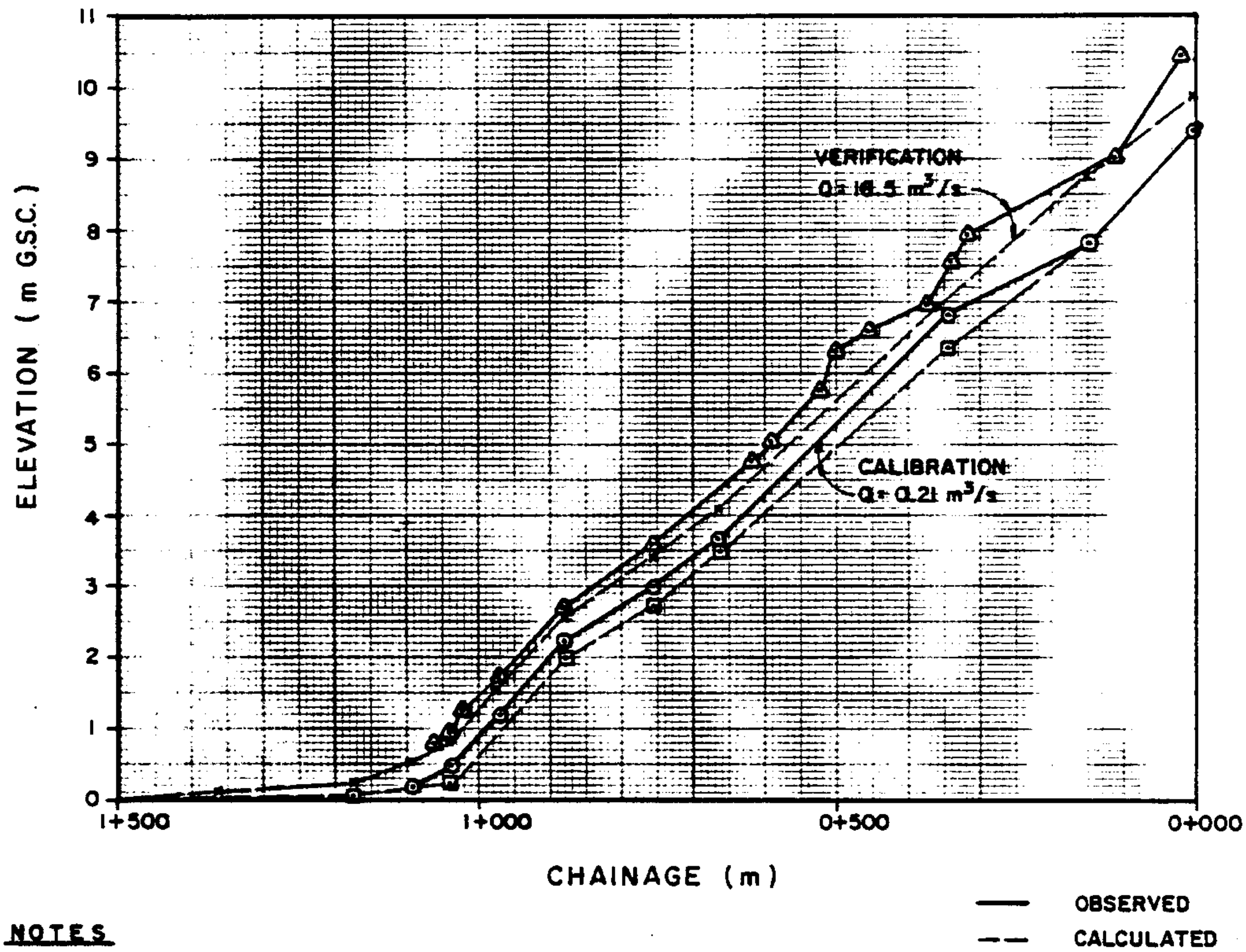
RUSHOON HYDROTECHNICAL STUDY

NON-COHESIVE ICE COVER
STABILITY CURVE



RUSHOON HYDROTECHNICAL STUDY

LOCATION OF CROSS-SECTIONS



NOTES

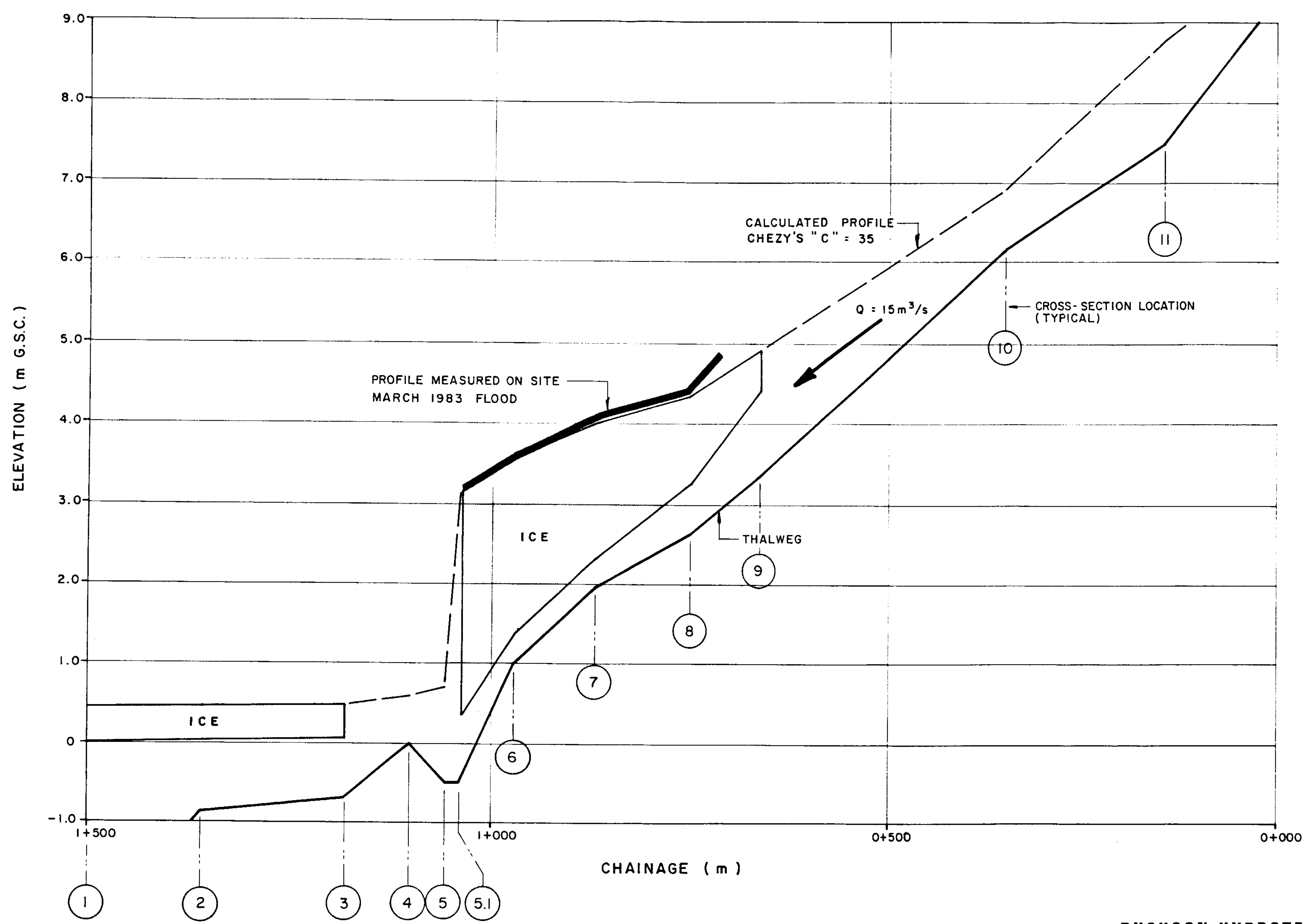
(i) —— OBSERVED DATA PLOTTED FROM TABLES III-3 AND III-4, IN FIELD REPORT (2).

(ii) --- CALCULATED DATA PLOTTED FROM COMPUTER PRINT OUTS FROM VOLUME 2, APPENDIX V(b).

RUSHOON HYDROTECHNICAL STUDY

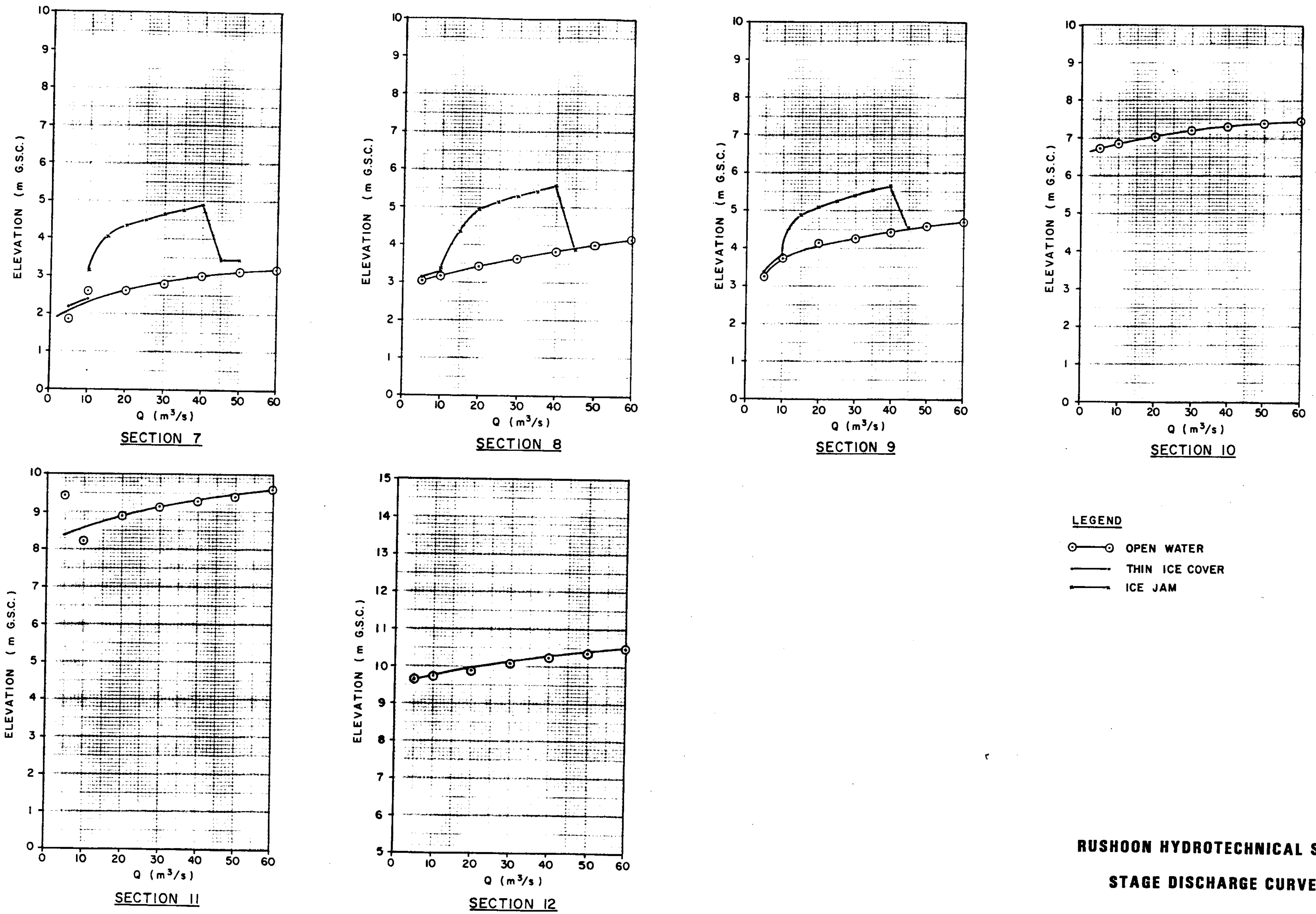
CALIBRATION & VERIFICATION COMPARISONS, OPEN WATER CONDITIONS

FIGURE 4.7



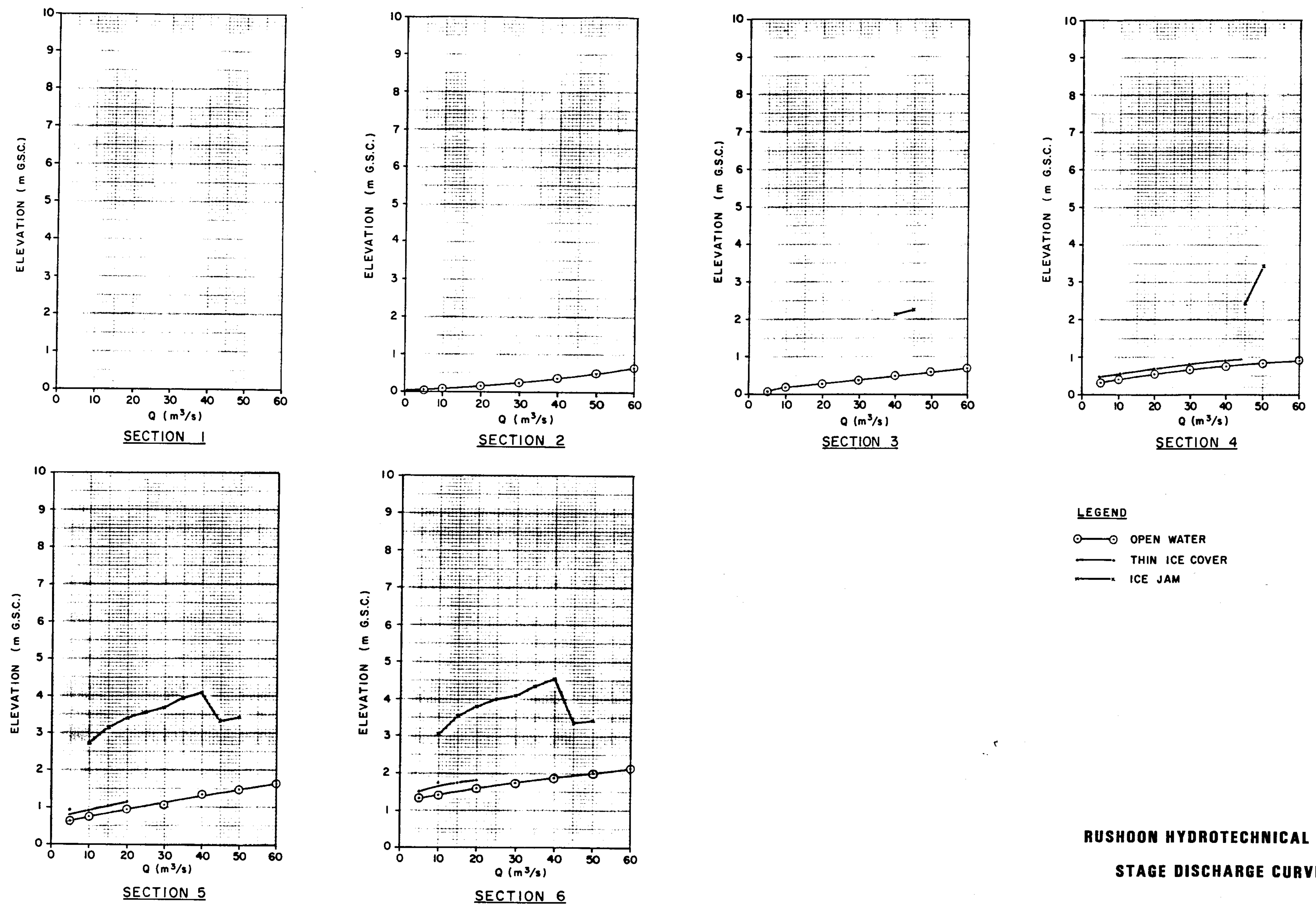
RUSHOON HYDROTECHNICAL STUDY

CALIBRATION RUN
ICE COVER PROFILE



RUSHOON HYDROTECHNICAL STUDY

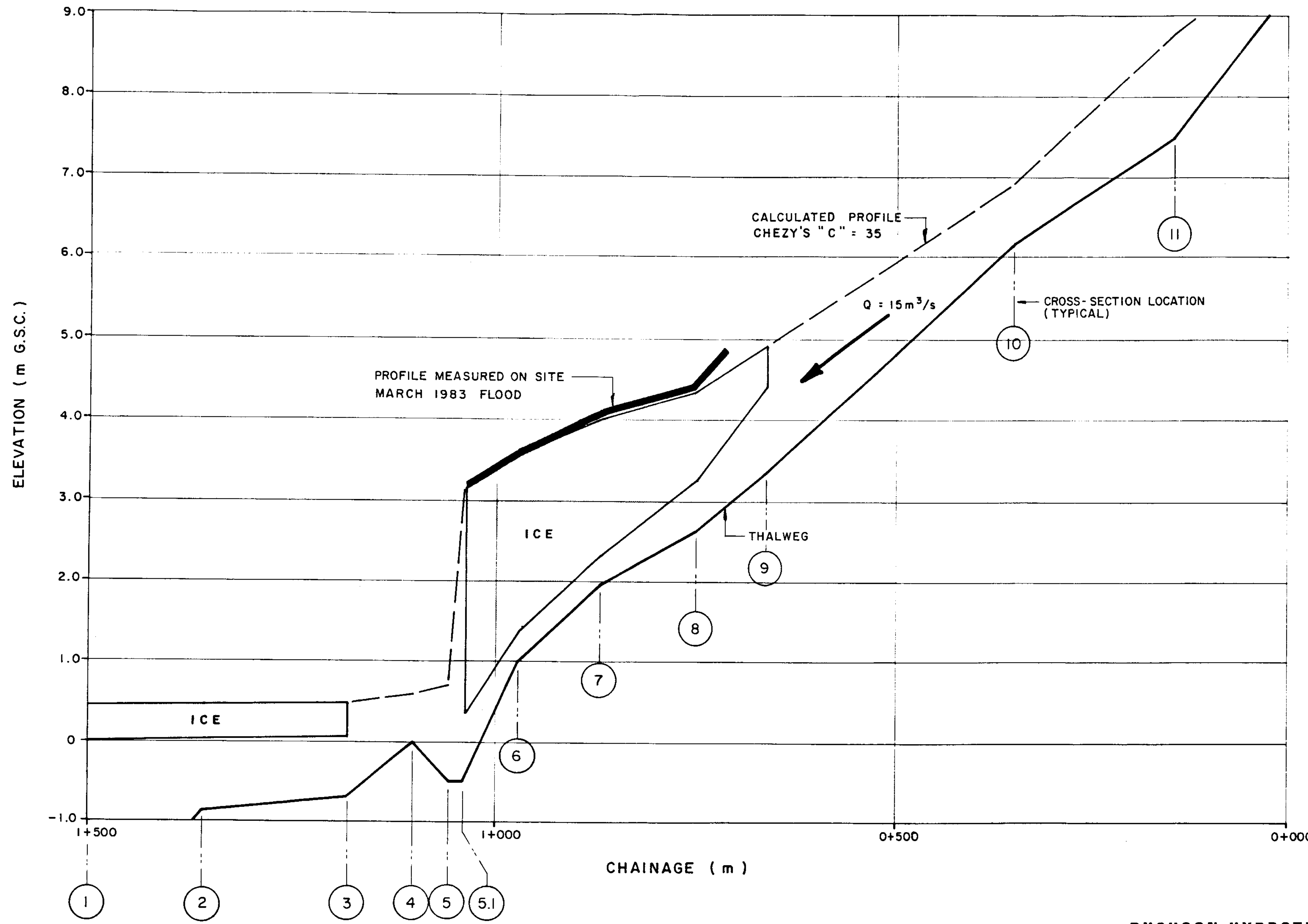
STAGE DISCHARGE CURVES



RUSHOON HYDROTECHNICAL STUDY

STAGE DISCHARGE CURVES

FIGURE 4.7

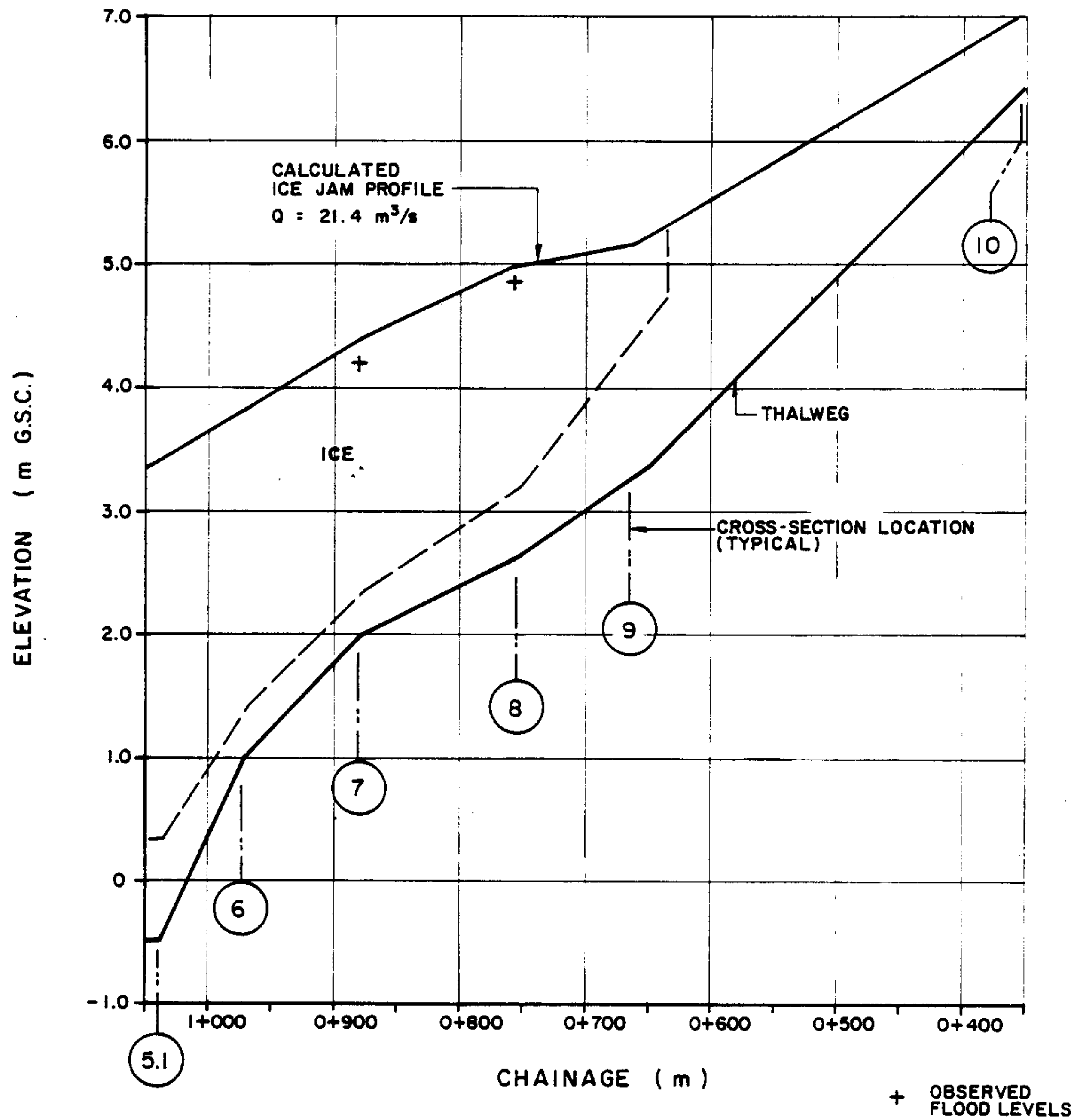


RUSHOON HYDROTECHNICAL STUDY

CALIBRATION RUN

ICE COVER PROFILE

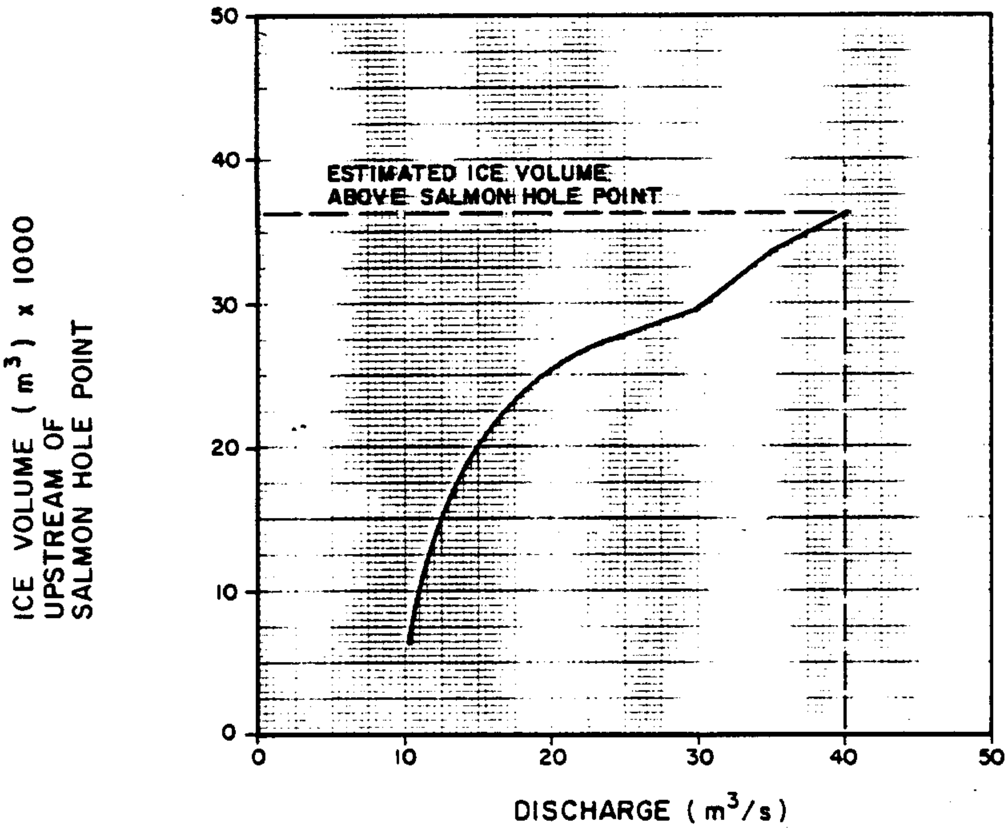
FIGURE 4.8



RUSHOON HYDROTECHNICAL STUDY

VERIFICATION OF CALCULATED PROFILE
WITH OBSERVED FLOOD LEVELS
FOR FEBRUARY '73 FLOOD

FIGURE 4.9



NOTE :

PLOTTED FROM COMPUTER RESULTS
GIVEN IN VOLUME 2, APPENDIX V (d).

RUSHOON HYDROTECHNICAL STUDY

ICE JAM VOLUMES AS A FUNCTION OF DISCHARGE

5. DELINEATION OF FLOOD RISK CONTOURS AND EVALUATION OF AVERAGE ANNUAL FLOOD DAMAGES

5.1 Background

The objectives of this section were:

(i) to delineate the 1 in 20 year and 1 in 100 year flood risk contours, and,

(ii) to evaluate average annual flood damages in the Study Area.

Flow probabilities of exceedence and stage-discharge relationships determined from the hydrologic and ice model studies, described in the preceding sections, provide the basis for these tasks.

5.2 Delineation of Flood Risk Contours

The starting point for determination of flood risk contours is a stage-probability relationship for the cross-section concerned.

For rivers subject to flooding from a unique cause the stage-probability curve can easily be determined from data provided on the flow-probability and stage-discharge curves. However, for a river such as Rushoon Brook, where the annual maximum stage could be caused by either ice season or open water floods, the calculation of the stage-probability relationship must include a means for weighing the probability associated with each condition. The method adopted assumes that the probability of the annual maximum stage occurring during the open water season (P_o) can be estimated from the observed behaviour of the river, such that:

$$P_o = \frac{\text{No.of yrs. annual max. stage was in open water season}}{\text{Total No. of years of record.}}$$

and likewise, for ice season maxima:

$$Pi = \frac{\text{No.of yrs. annual max. stage was in ice season}}{\text{Total No. of years of record.}}$$

Conditional stage-probability relationships for open and ice season conditions are then weighed using Po and Pi as shown in equation 5.1 below.

$$P(h) = [P(h) \mid open] \times Po + [P(h) \mid ice] \times Pi \qquad (5.1)$$

Where:

$P(h)$	=	the probability that a given annual max. stage (h) will be equalled or exceeded in any year.
$[P(h) \mid open]$	=	the conditional probability that a given annual max. stage (h) will be equalled or exceeded during the open water season of any year,

[P(h) | ice] = the conditional probability that a given annual max. stage (h) will be equalled or exceeded during the ice season of any year,

Po = the probability that an annual maximum stage is caused by an open water flood,

Pi = the probability that an annual maximum stage is caused by an ice season flood.

This procedure is applied as follows. At a given cross-section and selected water level, h:

- Obtain flows Q_{open} and Q_{ice} corresponding to water levels (h) from Figures 4.5 and 4.6.
- Then obtain the probabilities of exceedence for these flows from Figures 3.3 and 3.2, which correspond to [P(h)|open] and [P(h)|ice].
- Estimate Po and Pi as explained on page 5-2.
- P(h) is then determined from equation 5.1
- Repeat for a series of values of h.

Further details on this method are given in a paper by Mrazik (15).

Since Po and Pi varied from section to section depending on hydraulic conditions, it was necessary to determine maximum open and ice season stages, at each Section for each year of record, using flow data from Table 3.3 and the appropriate stage-discharge curves.

Using this technique, stage-probability curves were determined for each cross-section in the Study Area, and the 1 in 20 year and 1 in 100 year contours identified for each section. The resulting flood risk levels are summarized in Table 5.1 below:

TABLE 5.1 RUSHOON BROOK - FLOOD RISK LEVELS

Section	Flood Risk Levels in m (GSC) 1 in 20 Years	Flood Risk Levels in m (GSC) 1 in 100 Years	Remarks
1	1.1	1.1	Sections 1-4 are in a tidal zone where maximum levels correspond to large tides
2	1.1	1.2	
3	1.2	1.2	
4	1.2	1.3	
5	3.7	4.2	Sections 5-9, annual maxima may be caused either by ice jams or open water conditions
6	4.1	4.6	
7	4.6	5.0	
8	5.2	5.6	
9	5.3	5.7	
10	7.3	7.3	Sections 10-12, no ice cover forms in this area annual stage maxima produced by largest annual flow regardless of season
11	9.3	9.4	
12	10.2	10.3	

5.2 Delineation of Flood Risk Contours (Cont'd)

These levels have been plotted on the Flood Risk Map to delineate the areas subject to flooding, see Figure 5.1.

The results of the stage probability computations have been summarized in convenient form in Figure 5.2 which shows flood risk profiles in the main area of interest between Sections 5 and 9.

5.3 Evaluation of Average Annual Flood Damages

The average annual value of flood damages is an estimate, in present day dollars, of the average of all annual flood damages that would have occurred in the Study Area over a very long period of time. In the absence of historic samples of adequate length, average annual flood damages are normally determined by weighing estimated flood damages values by their probability of occurrence for floods of all severities. In this process the average annual flood damages, ΔE resulting from flood levels occurring between stages h_i and h_{i+1} is obtained by averaging the damage, Di, caused by such floods over their return period Ti (in years):

Thus, $\Delta E = \frac{Di}{Ti}$

recalling that $\frac{1}{Ti} = \Delta P\ (h)$

gives $\Delta E = Di\ \Delta P\ (h)$

The total average annual flood damages are then obtained by summing incremental values, ΔE, over the entire range of flood stages, thus:

$$E = \sum_{i=1}^{i=n} D_i \Delta P(h) \qquad (5.2)$$

Where: E = average annual flood damages

Di = the mean damages produced by flood stages between h_i and h_{i+1}

P(h)= the probability of occurrence of flood stages between h_i and h_{i+1}

i=1 corresponds to zero damage stage

i=n limit of flood plain.

Further information on this determination may be found in Reference 16 or in most texts on water resources.

To determine average annual flood risks by this method requires input on probability of flooding together with data on flood damage to homes, businesses, etc.

In the absence of detailed surveys on house designs and contents, flood damages were estimated using synthetic flood damage curves from Acres (17).

Acres' curves cover a variety of house classifications and give flood damage values as a function of depth of flood water above floor level. These curves were developed from a detailed analysis of housing types taking into account actual flood damage experience in the Town of Galt, Ontario. Floods occurring in the Galt area are normally of short duration and usually occur during the spring season, flow velocities are normally low and silt content minimal. These flooding conditions are similar to conditions in Rushoon since the fender wall, constructed in 1973, now prevents most ice from entering the community. Accordingly, Acres' synthetic cost curves may also be used for estimating flood damages in Rushoon.

Acres' flood damage estimates account for direct damages and indirect damages in monetary terms, while no monetary estimates are assigned to intangible damages.*

For assessment of indirect damages, Acres' report affirms the approach of the U.S. Department of Agriculture, which recommends that indirect damages be estimated as a percentage of direct damages. U.S. Dept. of Agriculture suggests that indirect damages be estimated at 10% - 15% of direct damages while Acres' sample calculations indicated values between 3% - 8%. Based on engineering judgment, a value of 10% of direct damages has been used for assessing indirect damages in this study.

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* (a) Direct Damages - "the actual physical losses that can be evaluated on the basis of restoring an object to its pre-flood condition either by replacement or repair" (Relates to damages to structure or contents of house).

(b) Indirect Damages - "are losses brought about by interruption of normal activities for which compensation cannot be obtained in other areas at a later date". Examples include loss of income, cleanup costs, etc.

(c) Intangible Costs - "costs that cannot readily be evaluated in direct monetary terms, such as loss of life, and suffering of adverse effects on health, social and economic security".

The main cause of exterior damages to fences, sheds, material and equipment (cars, skidoos etc) has been from impacts with ice debris. Risk of ice damage is now limited to flood stages higher than the fender wall, which corresponds approximately to floods having return periods of 10 years or greater (i.e. more severe than the 1983 flood). Order of magnitude damages from ice have been assessed at $400 per house, covering damage to fences, exterior walls, fences and cars (body damage only), allowing that some houses and cars will be sheltered and escape damage. Evidence from the 1983 flood indicated that exterior damage from water alone was negligible.

Altogether eleven buildings (nine houses, the Town Hall and the Parish Hall) were found to be at risk for the 1 in 100 year flood. All of these buildings are of wood frame construction and all were classified to Category CW* for the purpose of computing flood damages.

The Acres' study was completed in 1968 and all flood damage data were stated in terms of 1968 dollars. Therefore, for use in this study, Acres' data has been updated to 1985 cost levels by prorating 1968 costs by the ratios of the 1985/1968 Statistics Canada-Building Construction Indices. The data are summarized in Table 5.2.

---

* Category CW: An economy home of wood frame double wall construction, generally on post foundations.

TABLE 5.2

DIRECT DAMAGES FROM FLOODING OF A TYPICAL BUILDING

W.L. Above First Floor	Mean Damages (1968 Dollars)			Mean Damages (1985 Dollars) (Di)
	Contents	Structural	Total	
0.0 - 0.3 m	155	200	355	800
0.3 - 0.6 m	400	600	1000	2260
> 0.6 m	500	780	1280	2900
In Basement	30	300	330	750

Notes:

(i) Classification - all buildings belong to Category CW.

(ii) 1968 damage values are from Figure 1, 4 and 20, Reference 17.

(iii) Ratio of 1985/1968 building costs = 2.26

(iv) Losses to utilities, effects on transportation system, etc. are negligible.

Probabilities of occurrence $\Delta P(h)$ for 0.3 m stage increments were estimated from flood probability profiles given in Figure 5.2. Annual average flood damages were then computed, following equation 5.2, on a house by house basis. This resulted in an average annual flood damage evaluation of $3000 for the Rushoon flood plain [Table 5.3, at end of Section 5, summarizes these calculations].

5.3 Evaluation of Average Annual Flood Damages (Cont'd)

Flood damages for the 1983 flood were also estimated from the synthetic cost curves and were found to total $3,150 for three houses and the Parish Hall; this compares with $2,600 reported in the interview survey (2). This level of agreement is quite satisfactory and confirms, in a general way, the reasonableness of values derived using synthetic cost curves.

5.4 Sensitivity Analysis

The Guidelines for Benefit Cost Analysis of Flood Damage Reduction Projects (18), recommend that the reliability of findings of analytical studies be subjected to sensitivity analysis to investigate the impact on such findings of uncertainties in the main study parameters. In the case of the Rushoon hydrotechnical study, the outputs of analytical hydrology and ice studies were determinations of flood stages (water levels) and the area of flooding. Evaluation of the annual average flood damage for the Rushoon study area was also closely tied to these results.

The principal study parameters affecting flood level determinations were judged to be:

- the value of the proration factor K, used for transposing peak flows from Rattle Brook to Rushoon Brook;

- the value of Pi, probability that the annual maximum stage is caused by an ice season flood; and

- the value of Chezy "C" used in the ice model calculations.

5.4 Sensitivity Analysis (Cont'd)

Sensitivity analyses were carried out to test the impact of changes in these parameters on the determination of the 1 in 20 years and 1 in 100 years flood risk levels. In these tests the parameter values were changed to appropriate upper or lower parameter limits, based on engineering judgment, as below:

Test 1

K was increased from 0.93 used in the study analysis to 1.22, based on observation of the 1985 peak spring flow on Rushoon and Rattle Brooks. K = 0.93 was judged to be on the low side.

Test 2

Pi was reduced from 0.67 to 0.40 to reflect the possibility that ice jam floods occur less frequently than indicated by the study criteria. The estimate of 0.40 was based on information on past ice jam floods as far as could be deduced from reports of residents of Rushoon, taken during the interview survey (2). Pi = 0.67 was judged to be on the high side.

Test 3

Chezy's "C" used in the ice model was reduced from C = 35 to C = 30 based on experience from earlier studies carried out by Lasalle Hydraulic Laboratory Limited which indicated values of Chezy's "C" of less than 30 were unlikely to be encountered on real rivers.

The consequent impact on area of flooding and evaluation of average annual flood damages was then investigated in

5.4 Sensitivity Analysis (Cont'd)

a second series of sensitivity analyses which took into account changes in stage-probability relationships. The results of these analyses are summarized in Table 5.4, at end of Section 5 .

5.5 Discussion of Results

The following observations can be drawn from the sensitivity analyses:

(i) flood level determinations are relatively insensitive to variations of the principal study parameters;

(ii) impact on the area of land flooded resulting from changes in the principal study parameters is very small;

(iii) significant changes in flood damages are produced by small changes in flood water levels.

The significance of these observations are further discussed below:

5.5.1 Impact on Flood Level Determinations

Impact on flood level determinations was found to be relatively small with the greatest variation from study results produced by changes in the proration factor K. The observed impacts mainly affect the determination of the 1 in 20 year flood levels. Tests #1 and #2 both indicate that the 1 in 100 year flood level, which

5.5.1 Impact on Flood Level Determinations (Cont'd)

corresponds to the level behind an ice jam at limiting stability, is close to a self-limiting maximum, that would be exceeded neither by added flows nor by increased hydraulic roughness. In fact, one would expect from inspection of site topography, that flood flows should not greatly exceed the general elevation of the flood plain, since above this level the river will be able to bypass the ice jam. Some evidence of this occurring can be observed in photographs of the 1983 flood -Figure 2.4.

5.5.2 Impact on Flooded Land Area

Impact on area of flooding produced by variation of model parameters is minimal, + 0.2 ha (+3%), for 1 in 20 year flood levels and nil for the 1 in 100 year flood levels. Topography generally controls the extent of flooding and once the river terrace (or flood plain) is inundated higher flood stages will not significantly increase the extent of flooding, as can be seen from the flood risk map, Figure 5.1.

5.5.3 Impact on Evaluations of Annual Average Flood Damages

The sensitivity analyses showed the evaluation of annual flood damages to be relatively sensitive to variations in the value of the principal study parameters, indicating a range of +53% to -33% on the study results. It is believed nonetheless that the true value, insofar as this value depends on hydrologic and ice study determinations, will be much closer to the study estimate than these differences might indicate. The two tendencies noted

5.5.3 Impact on Evaluations of Annual Average Flood Damages (Cont'd)

above are compensatory since it is probable that flow estimates are low while the value of Pi is high. If this is indeed the case, the net uncertainty on the evaluation of annual average flood damages, arising from determinations in the hydrologic and ice studies, would be in the order of +53% - 33% = +20%. This deviation is similar to the probable error of synthetic flood damage estimates ± 25%, which is considered satisfactory for the level of benefit cost analysis required in small scale planning studies for flood control.

TABLE 5.3

DETERMINATION OF AVERAGE ANNUAL FLOOD DAMAGES

House #	Entry Level (m) G.S.C.	Probability of Flood Stages Between Indicated Depths, $\Delta P(h)$ 0.0-0.3m	0.3-0.6m	>0.6m	Probability W.L. Above Fender Wall P(h)	Remarks	
4	4.8	0.06	0.06	0.03	0.05	Direct Damages to Ground Floors:	
5	4.8	0.06	0.05	0.03	0.06		
11	4.5	0.07	0.07	0.04	0.08	0.59 x $ 800	= $ 472
12	4.4	0.08	0.05	0.03	0.10	0.47 x $2260	= $1,062
13	4.1	0.08	0.07	0.04	0.11	0.25 x $2900	= $ 725
14	4.2	0.09	0.04	0.02	0.12		
						Sub-Total:	= $2,259
15	4.0	0.04	0.08	0.03	0.12	Direct Damage to Basements:	
16	4.8	0.02	-	-	0.02		
17	4.3	0.02	0.02	-	0.03	0.20 x $ 750	= $ 150
Parish Hall	3.1	0.07	0.03	0.03	0.14	Sub-Total:	= $2,409
						Indirect Damages:	
						@ 10% Direct	= $ 241
Sub-Total		0.59	0.47	0.25	0.83	Exterior Damages	
3 Basement Only	4.4			0.20		0.83 x $ 400	= $ 332
		Average Annual Flood Damages					= $2,982
						use	$\boxed{\$3,000}$

NOTE: Values of $\Delta P(h)$ are taken from the probability profiles shown in Figure 5.2, which were determined by applying the procedures explained in Section 5.2. These procedures produced joint probability relationships which exhibited a marked "dog leg" appearance when plotted, and hence, produced the irregular pattern of the probability profiles and the inconsistent variation in values for P(h) extracted from Figure 5.2.

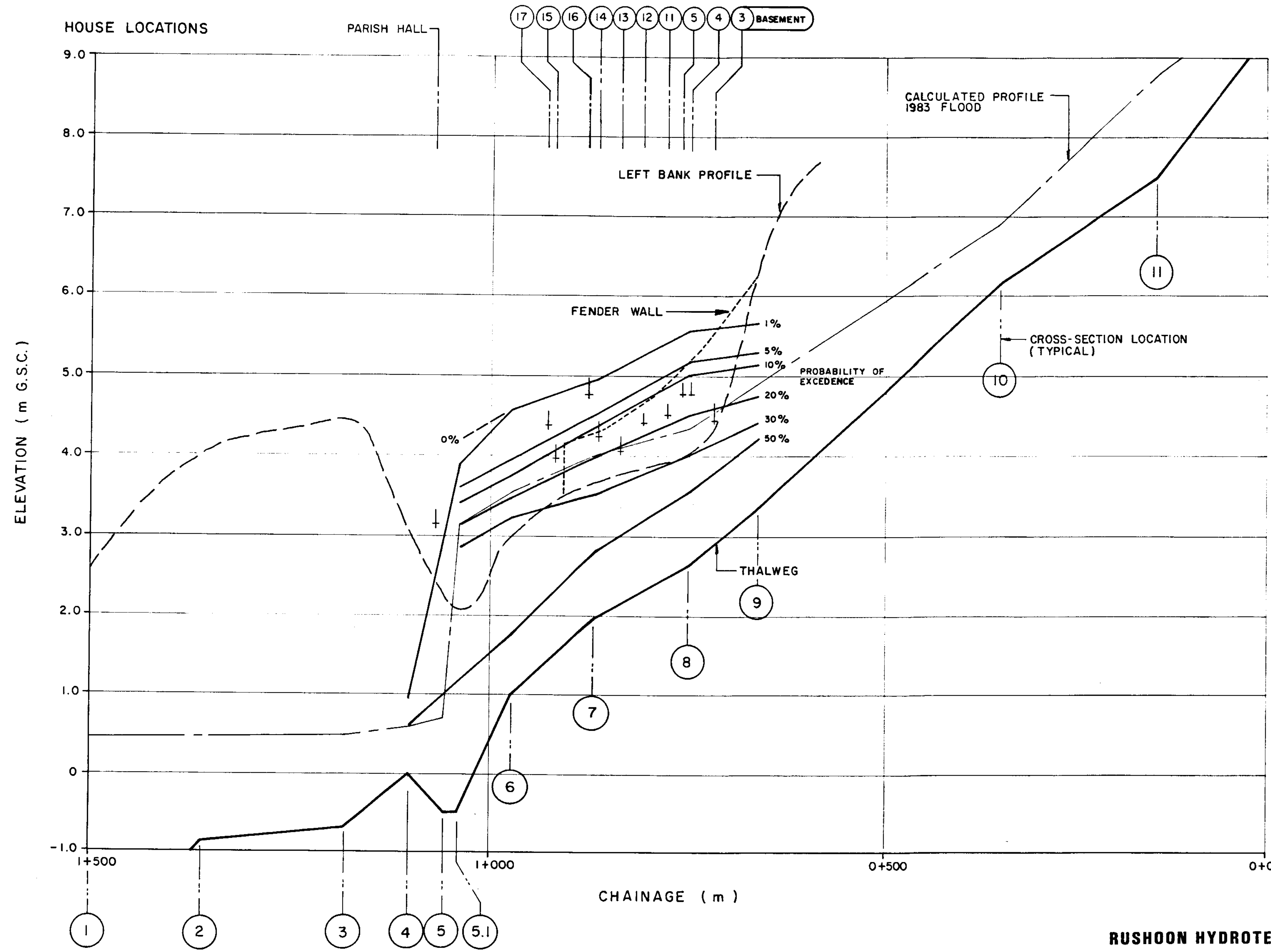
TABLE 5.4

RESULTS OF SENSITIVITY ANALYSES

TEST	IMPACT ON FLOOD LEVELS		REMARKS
	1 in 20 years	1 in 100 years	
#1, Proration Factor K increased from 0.93 to 1.22	+ 0.2 m	Nil	For 1 in 100 year flood level is governed by Q = 40 m^3/s, limiting flow for stable ice jam at Section 5.1
#2, Probability of annual max. stage in ice season, Pi reduced from 0.67 to 0.4	- 0.13 m	- 0.13 m	
#3, Chezy's "C" reduced from C = 35 to C = 30	+ 0.10 m	Negligible	Limiting flow for stable jam at Section 5.1 is reduced from 40 m^3/s to 38 m^3/s which compensates for greater flow depths.
	IMPACT ON FLOODED LAND AREA		
#4, Changed Stage Probability relationships due to increase of K from 0.93 to 1.22	+ 0.2 ha (+ 3%)	Nil	Flooded areas: 1 in 20 year flood level, 6.00 ha 1 in 100 year flood level, 7.50 ha Note: 1 in 100 year level governed by Q = 40 m^3/s, as above.
	IMPACT ON EVALUATION OF AVERAGE ANNUAL FLOOD DAMAGES		
#5, Changed Stage Probability relationships due to increase of K from 0.93 to 1.22	+ $1,600	(+ 53%)	Average annual, average flood damage = $3,000.
#6, Changed Stage Probability relationships due to decrease of Pi from 0.67 to 0.4	- $1,000	(- 33%)	Average annual, average flood damage = $3,000.

LIST OF FIGURES

FIGURE 5.2



RUSHOON HYDROTECHNICAL STUDY

FLOOD PROBABILITY PROFILES
(BETWEEN SECTIONS 4 & 9)

6. REMEDIAL MEASURES

6.1 Background

Eight remedial measures have been considered for flood control/protection in the Study Area. These measures were:

Alternative #1	-	Channel Improvements
Alternative #2	-	Ice Storage Weirs
Alternative #3	-	Perimeter Dyke
Alternative #4	-	Flood Control Dam
Alternative #5	-	Flood Proofing
Alternative #6	-	Flood Warning with Contingency Planning
Alternative #7	-	Control of Development
Alternative #8	-	Placing of Boulders in Tidal Pool

Preliminary conceptual design, layouts and, where necessary, cost estimates were prepared to provide the basis for a preliminary screening of the above alternatives. In this screening process, carried out in Phase I of the study, some alternatives were eliminated because they were found to be technically infeasible, while others were found to be far too expensive for the benefits produced.

Three alternatives appeared sufficiently promising to merit further study. These alternatives were:

Alternative #3(b)	-	Raise Fender Wall
Alternative #5	-	Flood Proofing Selected Homes
Alternative #6	-	Flood Warning with Contingency Planning.

6.1 Background (Cont'd)

These alternatives were investigated in greater detail during Phase II of the study.

Other tasks carried out during Phase II included economic analyses of the selected alternatives together with preparation of a plan of action for dealing with flooding in the Community of Rushoon.

Section 6, essentially reports on Phase II of the study under the following subsections:

- Description of Alternatives
- Economic Analysis
- Discussion of Results
- Recommendations

6.2 Description of Alternatives

For the sake of completeness, descriptions of all eight alternatives, both feasible and infeasible, are given in this section; however, the more extensive descriptions will be reserved for the selected alternatives #3, #5 and #6.

Conceptual layouts for Alternatives 1 to 4 and 7 are shown, schematically, in Figures 6.1 and 6.2. In addition, concise descriptions highlighting the design basis for these alternatives are given in the following sub-sections.

6.2 Description of Alternatives (Cont'd)

6.2.1 Alternative 1 - Channel Improvements: Figure 6.1(a)

Channel improvements designed to alter hydraulic conditions in the vicinity of Salmon Hole Point and Pete Moores Island were investigated with the ice model. The objective of these alterations was to reduce the stability of the ice cover in this area so as to prevent an ice jam forming at this point. The modifications tested included both widening and then deepening of the river channel between Cross-Sections 4 and 6, as shown in Figure 6.1(a). This approach was not found to be effective.

More extensive channel improvements, such as a general widening and deepening of the river bottom, were judged impractical because the cost of such measures would be in excess of $300,000; hence, this option was not further investigated. The present value of benefit to be derived would be the complete elimination of flood hazards worth $29,700.*

6.2.2 Alternative 2 - Ice Storage Weirs: Figure 6.1(b)

The possibility of reducing the volume of ice in the jam and thereby reducing the level of flood stages in the Study Area was also investigated by the ice model. For ice storage, two weirs were located at chainages of 0+300 and 0-500 (upstream of Rough Rocks), and the model run to determine ice storage capacities at each weir and the net effect on water levels in the flood prone area between sections 5 and 9. This approach was found to substantially reduce flood water levels in the flood prone area, as shown in Figure 6.3.

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* Capitalized flood damages (i = 10%, n = 50 years) = $29,700
  See sub-section 6.3.

6.2 Description of Alternatives (Cont'd)

6.2.2 Alternative 2 - Ice Storage Weirs (Cont'd)

Ice storage weir design was based on timber crib construction using treated timber. Both weirs are designed to be watertight structures, so as to provide the aesthetic and recreational benefits associated with small ponds/swimming holes. The estimated capital cost for this design is $285,000.

Some cost reductions could be obtained if the water proofing features of this design were eliminated and the ice storage weir design reduced to a simple permeable timber crib barrier across the Brook. The estimated capital cost for the ice weirs built to this minimal standard is $220,000.

The present value benefit from this alternative, assuming both ice weirs were constructed, would approach complete elimination of the flood problem, worth almost $29,700.

6.2.3 Alternative 3(a) - Perimeter Dyke: Figure 6.1(c)

Full protection of the flood prone area can be achieved by construction of a perimeter dyke enclosing this area. This approach requires:

- raising and water proofing the existing fender wall to provide a watertight flood barrier;
- extension of the above barrier by means of a low dyke 340 m in length;
- construction of drainage ditches to collect local runoff and direct it to an outfall culvert draining into Tidal Pool, at a point safely below the ice jam site.

6.2.3 Alternative 3(a) - Perimeter Dyke (Cont'd)

The estimated capital cost of these works is $195,000, and the benefit = $29,700 as before.

Alternative 3(b) - Raise Fender Wall: Figure 6.4

A more limited objective - namely, complete exclusion of ice pieces from the flood prone area can be considered as a partial implementation of Alternative 3. This would be accomplished by raising the existing ice fender wall by amounts varying from nil to 1.0 m to a level high enough to safeguard the area against ice influx from the 1 in 100 year flood; however this approach would be ineffective in lowering flood water levels.

The estimated capital cost of this work is $50,000, based on volume of crib work, with a service life of twenty-five years. Cost is based on execution of the work by a contractor. If carried out under a L.I.P. grant, some cost reductions of the order of 25% may be possible.

Tangible benefits from this alternative would be the elimination of damages from ice debris. The economic benefit, based on a 50 year comparison period, was estimated to be $3,300. The comparable present value of costs at an interest rate of 10%, including for replacement of the facility after 25 years, would be $54,600.

Intangible benefits would consist of the elimination of a major hazard to public safety.

6.2.4 Alternative 4 - Flood Control Dam: Figure 6.2(a)

Ideally, a flood control dam should be located at the outlet of a lake so as to able to develop a large storage

6.2.4 Alternative 4 - Flood Control Dam (Cont'd)

volume with the lowest possible dam. A good site for dam construction is also required. In practice, topographical conditions usually dictate site selection.

A suitable site for a flood control dam on Rushoon Brook would be at Rushoon Bridge on the Burin Peninsula Highway. Favourable features of this site include:

- control of a small lake about 200 m upstream of the Bridge;
- the possibility of utilizing the road embankments (after some modifications) as portion of the reservoir dyke;
- solid foundations under the bridge for supporting an outlet sluice and overflow spillway.

A preliminary design of this structure was developed based on the following criteria:

- 1 in 100 year flood
- limiting the maximum outflow to 10 m^3/s.

Preliminary design calculations indicated a 1.20 m square orifice would give the required outflow control and that a flood surcharge of 5.4 m above the centerline of the orifice would be required (corresponding to a flood storage volume of 3,600,000 m^3). This would require a dam having a height of at least 9.0 m. To incorporate a dam of this height into the existing Rushoon Bridge and its embankments is not considered a safe and practical proposition, hence this approach was rejected.

6.2 Description of Alternatives (Cont'd)

6.2.5 Alternative 5 - Flood Proofing:

As shown in Figure 5.2 eleven buildings are vulnerable to flooding. With the exception of House #3, none of these buildings have basements, hence the most practical form of flood proofing would be to raise these buildings above the maximum flood level. For houses without basements, full protection against flood damage can be obtained by raising their foundations by amounts varying from 0.5 m to 1.4 m. For House #3 further flood proofing of the basement would be required; however, the main floor is already at a safe level.

It is estimated that the cost of raising a typical building in Rushoon would be $5,500, which covers raising the building, extending or replacing foundations and shirting walls, disconnection and reconnection of water, septic tank hookup and electrical services (as required). The Municipal Hall is treated as a typical building since it has a similar floor area to a typical house. The Parish Hall has a floor area about four times as great as a typical house and hence would cost about $22,000 to raise.

Costing is based upon the approach recently used in Placentia where costs of raising homes for flood protection were shared on a 60%/40% basis by Government and the individual owner. In this case, the responsibility for carrying out the work was left in the hands of the individual owner, who was subsequently reimbursed for 60% of the costs up to $3,000, corresponding to a maximum project cost of $5,000 per house. (This program was administered for Government by the Newfoundland and Labrador Housing Corporation).

6.2.5 Alternative 5 - Flood Proofing (Cont'd)

Flood proofing of the basement of House #3 was estimated to cost about $1,600 which includes for construction of one window well, one external stairwell and the installation of a sump pump. This estimate was based on a preliminary layout assuming appropriate unit costs for each work item.

Benefits were based on the value of flood damage reduction to houses and contents only, as exterior damages are not protected in Alternative 5.

The economics of flood proofing buildings were assessed by benefit cost analysis on a building by building basis, as shown in Section 6.3 (and Table 6.1).

If the economic feasibility of this alternative is based upon the "time preference" discount rate, i = 10%, as recommended in the Guidelines (18), then only flood proofing of the basement of House #3 can be considered feasible. However, if the economic feasibility is based instead upon the "real interest" rate, i = 5%, then in addition to flood proofing the basement of House #3, raising of Houses #13, #11, #15 and #4 would also be feasible (see Figure 6.4). This argument is further debated in Section 6.4.

6.2.6 Alternative 6 - Flood Warning System

In Phase I of the study it was estimated that a flood warning system with contingency planning would be marginally viable once the startup costs have been written off. Following their review of the Interim Report the Technical Committee requested that the feasibility of this alternative be further investigated during Phase II of the study.

6.2.6 Alternative 6 - Flood Warning System (Cont'd)

The purpose of these additional studies were to better assess the effectiveness and costs associated with operation of a flood warning system. In particular, the following questions were of concern:

- how long in advance could reliable warnings be issued?
- would this warning be sufficient to implement effective contingency plans?
- how expensive would the system be to set up and operate? and,
- would it be worthwhile operating a flood warning system simply to supply advanced warning to individual home owners?

The findings of these studies are discussed below under the headings:

- forecasting system
- contingency plans
- costs and benefits

(a) Forecasting System

The first requirement of the forecasting system would be an assessment of the hazard. At Rushoon, the hazard consists of the amount of ice in the river available to form an ice jam at Salmon Hole Point. This assessment should be relatively simple since the ice volumes can be readily observed and measured.

The second requirement is that a reliable set of criteria be developed for predicting breakup. Observations of past

6.2.6 Alternative 6 - Flood Warning System (Cont'd)

(a) Forecasting System (Cont'd)

flooding on Rushoon Brook indicate that major winter floods are associated with significant rainstorms (and consequent snowmelt) generally following a period of prolonged sub-freezing weather. A critical combination of antecedent thawing degree days and rainfall has been found to provide a satisfactory set of criteria elsewhere (19) and should work at Rushoon as well. Examination of weather conditions associated with the 1983, 1975 and 1973 flood suggest the following preliminary criteria:

- thawing degree days — 5-10°C. days
- rainfall — > 25 mm

Further hydrological studies would be required to verify or improve the above preliminary criteria. It should furthermore be noted that the initial set of criteria may not be wholly satisfactory and that ice and weather observations would have to be reviewed periodically and the criteria updated.

The type of forecasting system envisaged would utilize regular weather forecasts from the Atmospheric Environment Service Office in Gander, in conjunction with on site observations of ice conditions, temperature precipitation and perhaps flows. Instrumentation to support this system would be minimal, typically as found in ordinary Climatological Stations plus a staff gauge for measuring flow (for cost data see Table 6.2). A key requirement for the satisfactory functioning of the system would be a dependable ice-cum-weather observer.

6.2 Description of Alternatives (Cont'd)

6.2.6 Alternative 6 - Flood Warning System (Cont'd)

(a) Forecasting System (Cont'd)

The effectiveness of the forecast system depends on two factors, first its reliability and secondly the amount of advanced warning provided. Mr. S. Porter, Scientific Officer, AES, St. John's, advised that forecasting amounts of rainfall remains one of the most difficult forecasting problems, particularly where there are no "upstream stations" - the very condition on the Burin Peninsula where most weather systems come from offshore! He noted that the forecasting horizon for accurate rainfall quantity predictions would be short, probably in the order of six hours.

Fortunately, the warning period is not solely determined by weather forecast parameters but also includes the hydrological response time of the river. From examination of flow records on Rattle Brook, it is anticipated about nine hours would elapse after start of a rainstorm before breakup, thus allowing a warning period of 9 to 15 hours for implementation of contingency plans.

(b) Contingency Plans

To significantly reduce the ice jam problem a significant portion of Tidal Pool would have to be entirely cleared of ice. The feasibility of executing this was investigated for two types of construction equipment; a Caterpillar 235 Backhoe with 1.6 m^3 bucket and a Caterpillar D8K Bulldozer with a universal blade. Work amounts were

6.2.6 Alternative 6 - Flood Warning System (Cont'd)

(b) Contingency Plans (Cont'd)

assessed for both types of equipment utilizing production factors given in the Caterpillar Handbook (20) applied to an eight hour period (the estimated time available prior to the onset of jamming taking into account logistical requirements). The Caterpillar D8K bulldozer was found to be significantly more productive with the capacity of clearing between 2000 m^3 to 4000 m^3 in eight hours. This approaches the amount of ice that would need to be cleared to open a 10 m wide passage through Tidal Pool. There remains some doubt as to whether incoming ice will flow freely through an opening only 10 m wide, hence the benefits were conservatively assumed to be limited to providing additional ice storage volume of an equal amount to the ice cleared. Given that the incoming volume of ice may range from 20,500 m^3 to 83,900 m^3* this benefit may prove to be inadequate. Some additional benefits may be obtainable however,if the bulldozer was available to break up incipient ice jams as they form and thus to keep the river clear. Additional investigations are still required to fully investigate the effectiveness of this contingency plan, hence, a field trial would be recommended prior to its implementation.

A further difficulty with this approach relates to the availability of a bulldozer of sufficient power that can be mobilized rapidly and is affordable.

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* Estimated Volume in 1983 jam was 20,500 m^3.
  Measured Volume in March 1985 ice survey was 83,900 m^3.

6.2.6 Alternative 6 - Flood Warning System (Cont'd)

(b) Contingency Plans (Cont'd)

Finally, the use of blasting was not considered since merely breaking up ice in Tidal Pool would not be effective.

(c) Costs and Benefits

Costs and benefits were estimated in order to complete an economic analysis of this alternative. Costs allow for manhours, equipment materials plus normal markups for profit and overhead. Cost calculations further assumed that:

(i) that the ice observer would be hired locally and work on a part time basis, and
(ii) that equipment can be readily obtained from provincial, municipal authorities or a local contractor on an "as required" basis.

For purposes of economic analysis the flood warning and contingency plan was assumed to be fully effective; while for flood warning only, damages to house contents were assumed to be eliminated and exterior damages reduced by 50%. A success rate of 80% in forecast warnings was also assumed which recognizes the uncertainties (and consequent risk of failure) in weather forecasting.

Table 6.2 (at the end of Section 6) summarizes costs, benefits and the findings of the economic analysis of Alternative 6.

6.2 Description of Alternatives (Cont'd)

6.2.6 Alternative 6 - Flood Warning System (Cont'd)

(c) Costs and Benefits (Cont'd)

Even with the optimistic assumption on costs and benefits these analyses conclude that Alternative 6 is not cost effective.

6.2.7 Alternative 7 - Control of Development: Figure 6.2(c)

Strictly speaking, control of development does not constitute a remedial measure; however, prevention is better than a cure and hence the control of new developments is an effective means of avoiding future problems.

It is recommended that the Rushoon Municipal Plan be amended to control development in two zones, as described below:

Plan Zone 1 - development should be limited to structures of flood proof design;

Plan Zone 2 - no buildings should be permitted in this area (A potential flood problem exists in this area due to ice jams at Rough Rocks which may divert the river through this area).

Delineation of these plan zones is shown in Figure 6.6. Note, plan zones are delineated taking into account administrative convenience and should not be confused with flood risk zones.

6.2.8 Alternative 8 - Placement of Boulders in Tidal Pool

The possibility positioning an array of large boulders in Tidal Pool to break up the ice cover as it rides up and

6.2 Description of Alternatives (Cont'd)

6.2.8 Alternative 8 - (Cont'd)

down on the tide and thereby lessen the flood problems, was suggested as a possible alternative. This approach was judged to be ineffective because simply breaking up the ice in the Tidal Pool is not enough, the ice must be cleared as well.

6.3 Economic Analysis

The economic feasibility of the technically viable alternatives were assessed by benefit cost analyses as recommended in the Guidelines (18).

Cost input for these analyses were capital cost estimates based on the following inputs:

- conceptual designs based on available 1:2500 site mapping;
- visual inspection of foundation and site conditions;
- comparative unit prices from recent construction projects in the Island of Newfoundland;
- all alternatives would be completed in one construction season.

Other cost inputs were maintenance and operating costs. These were estimated from staffing, equipment and material requirements.

Tangible benefits were taken to be the reduction in annual average flood damages that would be obtained upon implementing a given alternative.

Present value techniques were used to reduce future costs and benefits to equivalent present values to facilitate comparison of benefits and costs for alternatives having differing cash flow patterns. This is a standard technique used in engineering economic studies, details of which may be found in most texts in engineering economics or in Reference (21).

As recommended in the Guidelines (18), present values of benefits and costs for "base" analyses were computed using a "time preference" discount rate, i = 10%. The impact on the results of the base analyses were then examined by a sensitivity study, utilizing:

- i = 15% - "social opportunity cost rate"
- i = 5% - "real interest rate".

In all cases examined a comparison period of fifty years was used. For structures with service lives less than fifty years, the present value of costs allows for replacement of the structure at the end of its service life. For non-structural alternatives the comparison period was also set at 50 years.

Detailed economic analyses, as above, were only carried out for Alternatives 5 and 6. For the remaining alternatives, which were all clearly uneconomic, approximate benefit cost ratios were estimated as the ratio of benefits/capital costs (for i = 10%) ignoring maintenance and interim replacement costs, which were relatively small in these cases. The results of these analyses are shown in Figures 6.1 and 6.2.

6.3 Economic Analysis (Cont'd)

Details of the analysis for Alternatives 5 and 6 are shown in Table 6.1 and Table 6.2 respectively, to be found at the end of Section 6.

Finally, since the Guidelines do not explicitly consider the impact of inflation, it was decided to extend the scope of the economic analysis to address this problem. The approach taken was to assume "typical" interest and inflation rates, as below:

- i = 10% - "typical Bank of Canada interest rate, ca 1985"
- e = 5% - "typical inflation rate, ca 1985",

6.4 Discussion of Results

Alternatives 1-4 were found either to be technically unfeasible or uneconomic or generally both. Economic analyses however, are limited to comparisons of tangible costs and benefits to which dollar values can be attached. The Guidelines (18) also require that intangible benefits, which do not lend themselves as easily to objective analysis, be taken into consideration.

As noted in Sections 2.3 and 4.6, surges of ice and water cascading through the Community constitute a serious hazard to life and limb. This hazard can be completely eliminated by raising the fender wall as proposed in Alternative 3(b). This fender wall would then completely eliminate the danger from ice entering among the houses and also attenuate the rate of rise of water levels. While this same intangible benefit can also be attributed to several other alternatives, it would be obtained for the least cost by Alternative 3(b). On this basis, implementation of Alternative 3(b) - "Raise Fender Wall", is recommended at an estimated cost of $50,000.

Other intangible benefits, such as reduction of nuisance and worry due to the floods, were not considered to be of sufficient importance to warrant reconsideration of other economically infeasible alternatives.

Economic analyses of Alternative 5. Table 6.1 (at end of Section 6) shows that the results are very sensitive to assumptions of discount and/or inflation rates. If decision making is based upon the base case, i = 10% as required in the Guidelines, then only flood proofing of the basement of House #3 is economically justified. If, on the other hand, the criterion of economic feasibility is taken to be the "real interest rate", i = 5%, then flood proofing of five houses would be economically justified.

It was felt that the omission of an explicit treatment of the impact of inflation is a significant shortcoming in the method of analysis prescribed by the Guidelines. This omission contradicts the common observation that the effects of inflation on future costs and benefits are important. Accordingly, it was decided to investigate these effects by assuming a "typical" discount rate of i = 10%, which is indicative of the cost of money to the Federal Government (the Bank of Canada interest rate) together with a "typical" inflation rate, e = 5%, indicative of the prevailing rate of inflation in 1985. The results from this approach were found to be similar to those obtained for the "real interest rate" case, i = 5%, suggested in the Guidelines (18).

Support for treating the impact of inflation in this manner is provided by Johnson (22) in a paper on power utility economics. He argues that interest rates are "driven" by inflation and that the "real interest rate"* when inflation is discounted corresponds to an interest rate of about 5% which he terms the "real rate of interest" (or rate of return). It may be argued that Government should not expect to earn larger "profit" on investments in such basic facilities as flood control and protection works; it is therefore concluded that the "real interest rate" case represents an appropriate basis for evaluating the economic feasibility of Alternative 5.

Accordingly, it is recommended that in addition to flood proofing the basement of House #3, Houses #13, #11, #15 and #4 also be raised. It is also suggested that Government consider extending aid to the owners of Houses #5 and #12 which showed benefit-cost ratios marginally less than 1.00.

Table 6.2 summarizes the economic analysis of Alternative 6. The results show that Alternative 6 is not economically feasible for any of the discount rates considered, in fact, the results tend to be relatively insensitive to changes in discount rate. When the two approaches to contingency planning (ice clearing and warning only) are compared, the ice clearing option appears marginally superior, although this result may be false, given doubts about the effectiveness of the contingency plan. Thus both from cost and effectiveness viewpoints this alternative was found to be unsatisfactory. Accordingly, it is recommended that this alternative be abandoned.

6.5 Recommendations

It is recommended that the following measures be implemented:

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* "real interest rate" in this context is sometimes referred to as "effective interest rate", a terminology which is perhaps more appropriate.

6.5 Recommendations (Cont'd)

(i) Alternative 3(b) - Raise Fender Wall as indicated in Figure 6.4. The capital cost of this work is estimated to be $50,000.

(ii) Alternative 5 - Flood proofing as in the following table, based on a "real interest" rate of 5%.

House	Owner	Scope of Work
# 3	Mr. Joe Hayden	Floodproof Basement
#13	Mr. Gary Lake	Raise house to sill elev = 5.40 m (lift 1.4m)
#11	Mrs. Sarah Cheeseman	Raise house to sill elev = 5.70 m (lift 1.2m)
#15	Mr. Roy Barrow	Raise house to sill elev = 5.05 m (lift 1.1m)
# 4	Mrs. Melinda Hayden (store)	Raise house to sill elev = 5.90 m (lift 1.1m)

Government should also consider extending aid to owners of Houses #5 and #12.

The estimated cost of this work = $23,600. For these house locations see Figure 6.5.

It should be noted that only flood proofing of House #3 would be economically justified if the method of economic analysis given in the Guidelines was strictly followed.

(iii) Alternative 7 - Control of Development. Modify the Municipal Plan for the Community of Rushoon to incorporate the zoning requirements shown in Figure 6.6.

TABLE 6.1

ALTERNATIVE #5 FLOOD PROOFING

ECONOMIC ANALYSIS

HOUSES	PROBABILITY			REDUCTION IN FLOOD DAMAGES	COSTS	BENEFIT COST ANALYSIS											
						Time Preference Rate, i=10%			Social Opport. Cost Rate =15%			Real Interest = 5%			With Inflation, i=10%, e=5%		
	0.0m 0.3m	0.3m 0.6m	> 0.6m			P.V. Benefits	Net P.V.	B/C	P.V. Benefits	Net P.V.	B/C	P.V. Benefits	Net P.V.	B/C	P.V. Benefits	Net P.V.	B/C
#3 (Basement)	-	-	0.20	$ 165	$1600	$ 1640	$ 40	1.03	$ 1100	-500	0.69	$ 3010	1410	1.88	$ 3130	1530	1.96
#13	0.08	0.07	0.04	372	5500	3690	-1810	0.67	2480	-3020	0.45	6790	1290	1.24	7050	1550	1.28
#11	0.07	0.07	0.04	364	5500	3610	-1890	0.66	2420	-3080	0.44	6650	1150	1.21	6900	1400	1.26
#15	0.04	0.08	0.03	330	5500	3270	-2230	0.60	2200	-3300	0.40	6020	520	1.10	6250	750	1.14
#4	0.06	0.06	0.03	298	5500	2950	-2550	0.54	1980	-3520	0.36	5440	-60	0.99	5650	150	1.03
#12	0.08	0.05	0.03	291	5500	2890	-2610	0.53	1940	-3560	0.35	5310	-190	0.97	5510	10	1.00
#5	0.06	0.05	0.03	273	5500	2710	-2790	0.49	1820	-3680	0.33	4980	- 520	0.91	5170	-330	0.94
#14	0.09	0.04	0.02	243	5500	2410	-3090	0.44	1620	-3880	0.30	4440	-1060	0.81	4610	-890	0.84
Parish Hall	0.07	0.03	0.03	232	22000	2300	-1970	0.10	1550	-20450	0.07	4240	-17760	0.19	4400	-17600	0.20
#17	0.02	0.02	-	67	5500	664	-4836	0.12	446	-5054	0.08	1220	-4280	0.22	1270	-4230	0.23
#16	0.02	-	-	18	5500	178	-5322	0.03	120	-5380	0.02	329	-5171	0.06	341	-5159	0.06

NOTES:

(i) Direct + indirect damages to ground floor: Water Depth 0.0 - 0.3 m, damages $ 880
Water Depth 0.3 - 0.6 m, damages $2490
Water Depth > 0.6 m, damages $3190

Direct + indirect damages to basement: Water Depth > 0.6 m, damages $ 825

(ii) Economic Life = 50 Years

(iii) Net P.V.: Net Present Value = Present Value of Benefits - Present Value of Costs

(iv) B/C = Present Value of Benefits/Present Value of Costs

(v) The economic analysis with i = 10% and e = 5%, follows Johnson (22)

TABLE 6.2

ALTERNATIVE #6 FLOOD WARNING SYSTEM

BASIC DATA

ITEMS		FLOOD WARNING & CONTINGENCY PLANS	FLOOD WARNING ONLY
COSTS	Initial	Equipment = $ 1,500 Engineering = $10,500 Field Trial = $ 4,000 $16,000	Equipment = $ 1,500 Engineering = $10,500 $12,000
	Operating	Ice Observer = $ 1,700 Ice Clearing = $ 1,000 Engineering Review = $ 800 $ 3,500	Ice Observer = $ 1,700 Engineering Review = $ 800 $ 2,500
BENEFITS	Reduction in Damage to Houses	80% of $2650 = $2120	80% of $1629 = $1303
	Reduction in Damages to Yards	80% of $332 = $266	80% of $166 = $133
	TOTAL	$2,386 (say $2,400)	$1,436 (say $1,450)

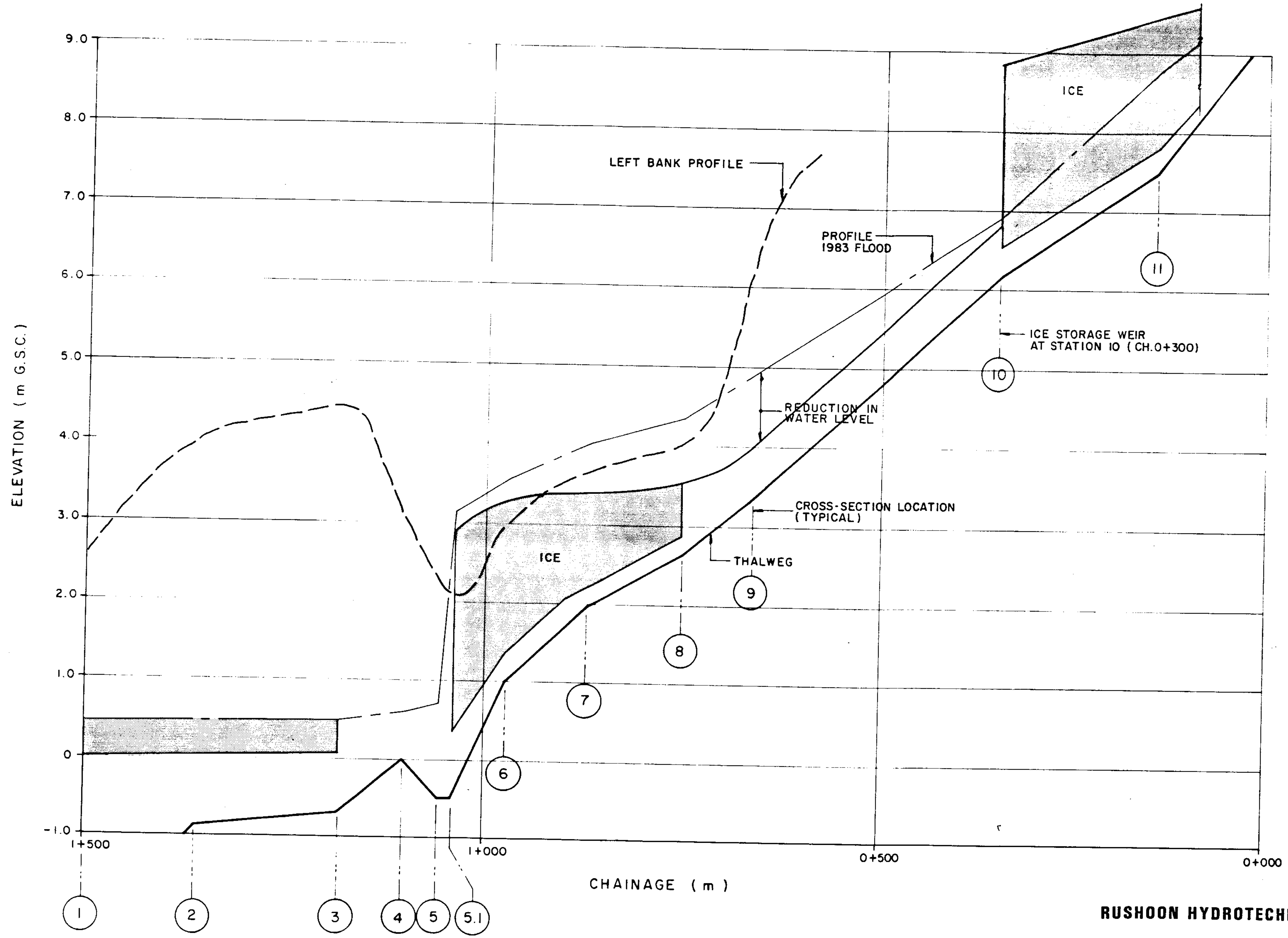
ECONOMIC ANALYSIS

ITEMS		FLOOD WARNING & CONTINGENCY PLANS	FLOOD WARNING ONLY
Time Preference Rate = 10%	P.V. Benefits	$ 23,800	$ 14,400
	P.V. Costs	$ 50,700	$ 36,800
	Net P.V.	-$ 26,900	-$ 22,400
	B/C	0.47	0.39
Social Opport. Cost Rate = 15%	P.V. Benefits	$ 16,000	$ 9,700
	P.V. Costs	$ 39,300	$ 28,700
	Net P.V.	-$ 23,300	-$ 19,000
	B/C	0.41	0.34
Real Interest = 15% = 5%	P.V. Benefits	$ 43,800	$ 26,500
	P.V. Costs	$ 63,900	$ 45,600
	Net P.V.	-$ 20,100	-$ 19,100
	B/C	0.69	0.58
With Inflation i = 10% e = 5%	P.V. Benefits	$ 45,500	$ 27,500
	P.V. Costs	$ 66,300	$ 47,400
	Net P.V.	-$ 20,800	-$ 19,900
	B/C	0.69	0.58

(i) For Flood Warning Only - benefits assume damages to contents of homes eliminated and exterior damages reduced by 50%.
(ii) Economic Comparison is based on an economic life of 50 years.
(iii) Net P.V.: Net Present Value = Present Value of Benefits - Present Value of Costs.
(iv) B/C = Present Value of Benefits/Present Value of Costs.
(v) Benefits assume 80% reliability of forecast and 100% response rate
(vi) Ice clearing required once every three years, hence 3000 ÷ 3 on average
(vii) The economic analysis with i = 10%, e = 5% follows Johnson (22).
(viii) Estimates of costs for meteorological equipment were obtained from A.E.S. Scientific Services, St. John's
(ix) Estimates of engineering costs are based on an estimate of engineering manhours
(x) Estimates for equipment rental and manhours were based on recent construction experience in Newfoundland.

<u>LIST OF FIGURES</u>

FIGURE 6.3

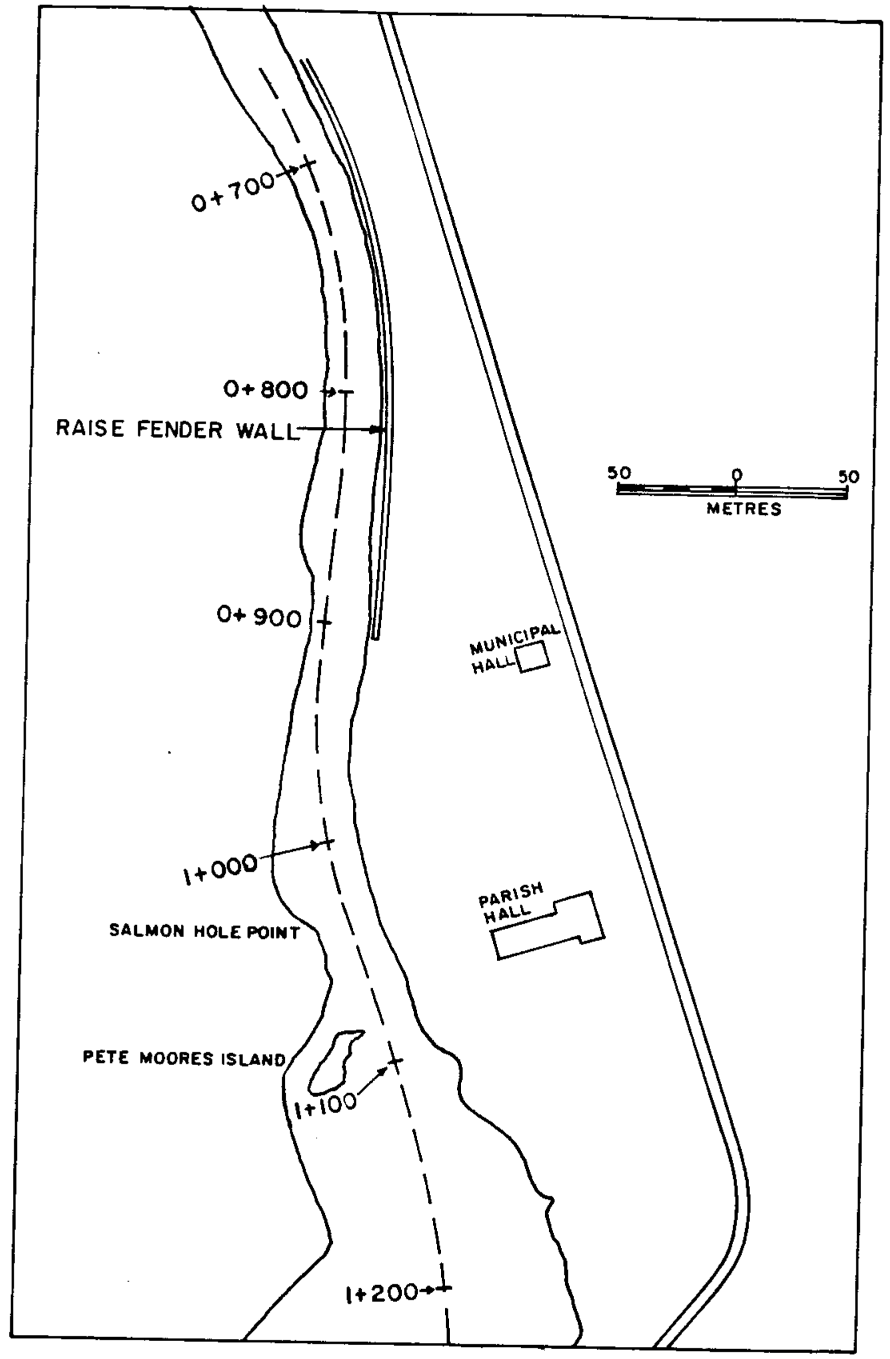


RUSHOON HYDROTECHNICAL STUDY

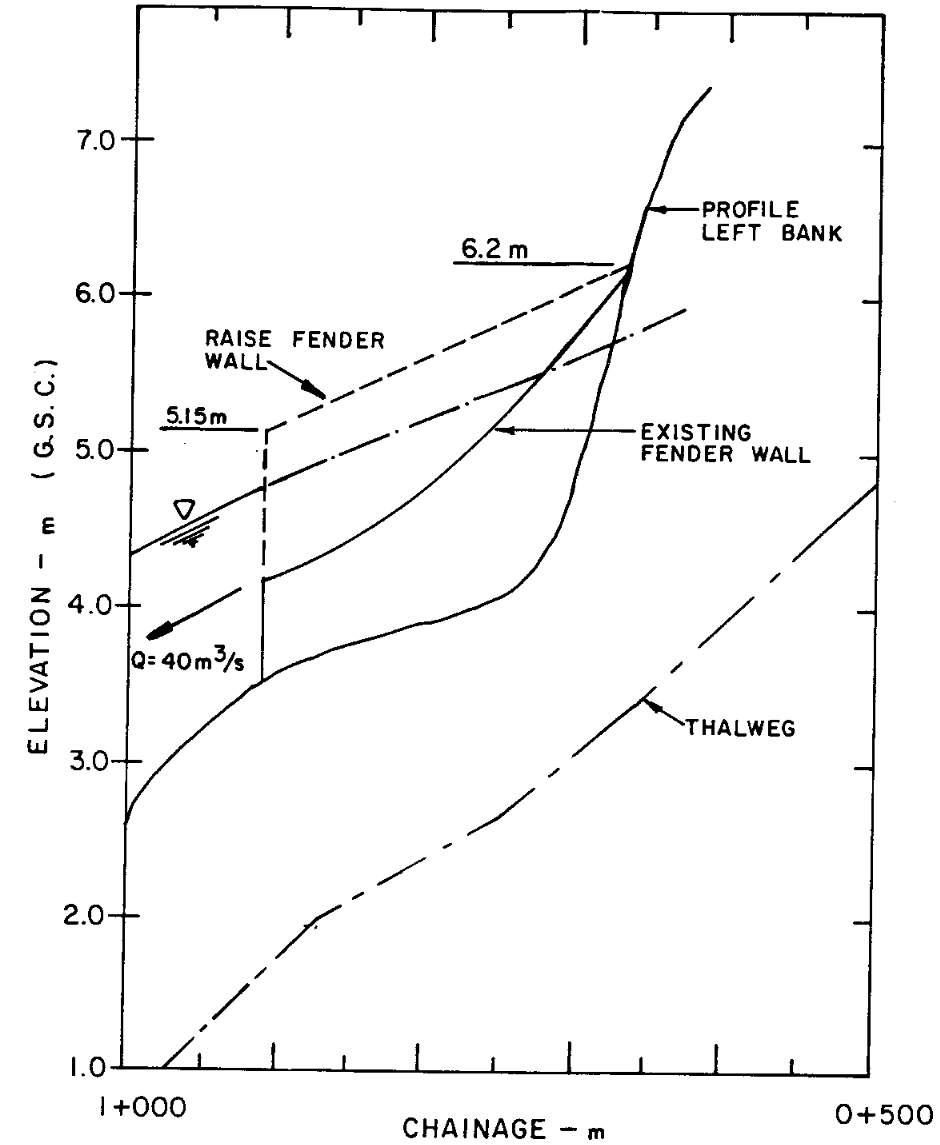
REDUCTION IN W.L. WITH ICE WEIRS

$Q = 15\ m^3/s$

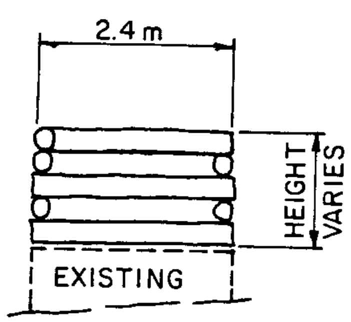
FIGURE 6.4



PLAN VIEW



PROFILE THROUGH FENDER WALL



NOTES:

1: FILL WITH CLEAN COBBLES AND SMALL BOULDERS

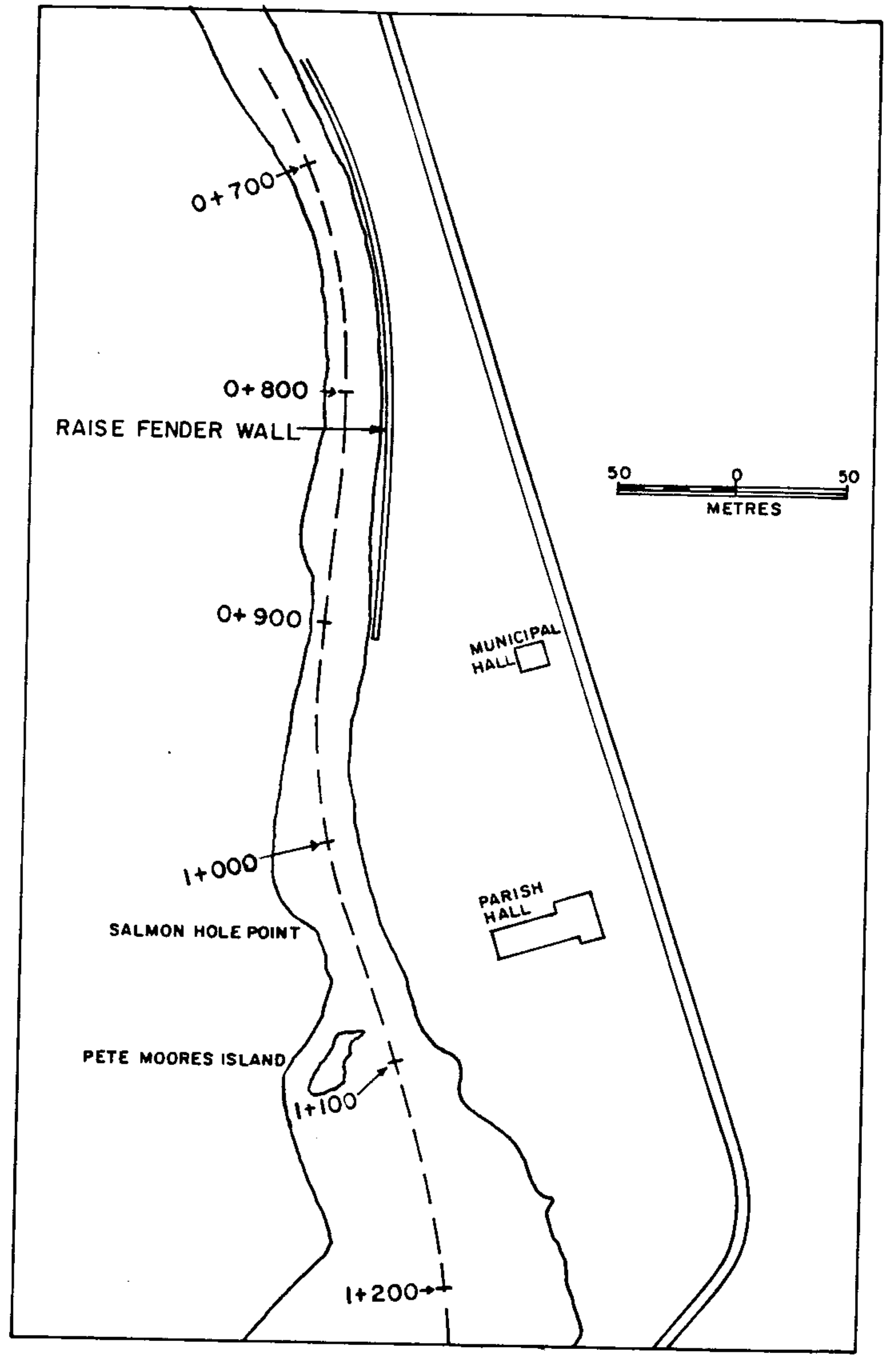
2: TREAT TIMBER WITH APPROVED WOOD PRESERVATIVE

X-SECTION THROUGH TIMBER CRIB (TYPICAL)

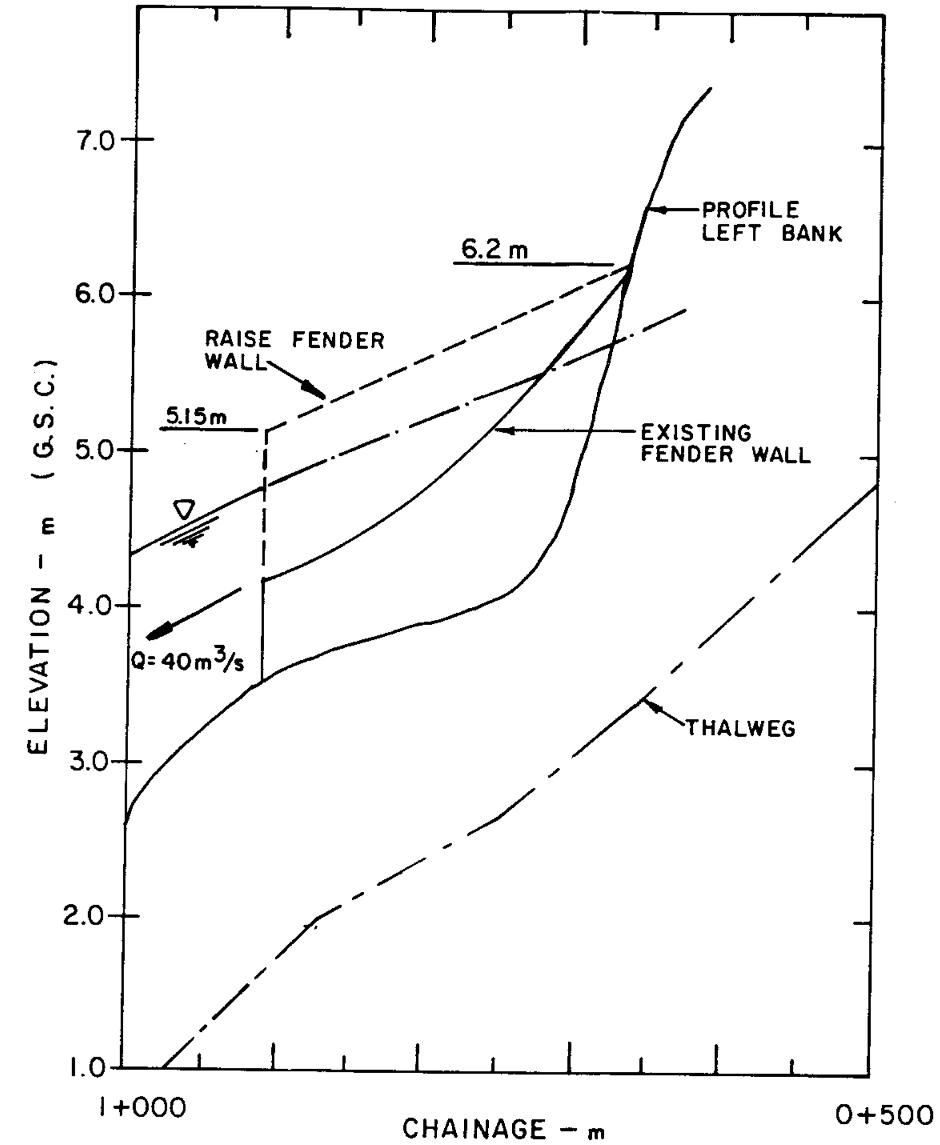
RUSHOON HYDROTECHNICAL STUDY

ALTERNATIVE 3(b)-RAISE FENDER WALL

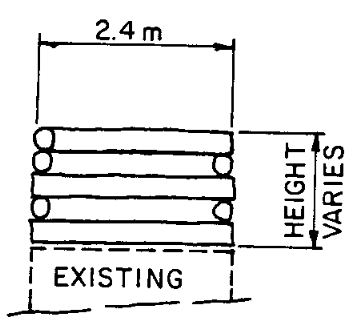
FIGURE 6.4



PLAN VIEW



PROFILE THROUGH FENDER WALL



NOTES:

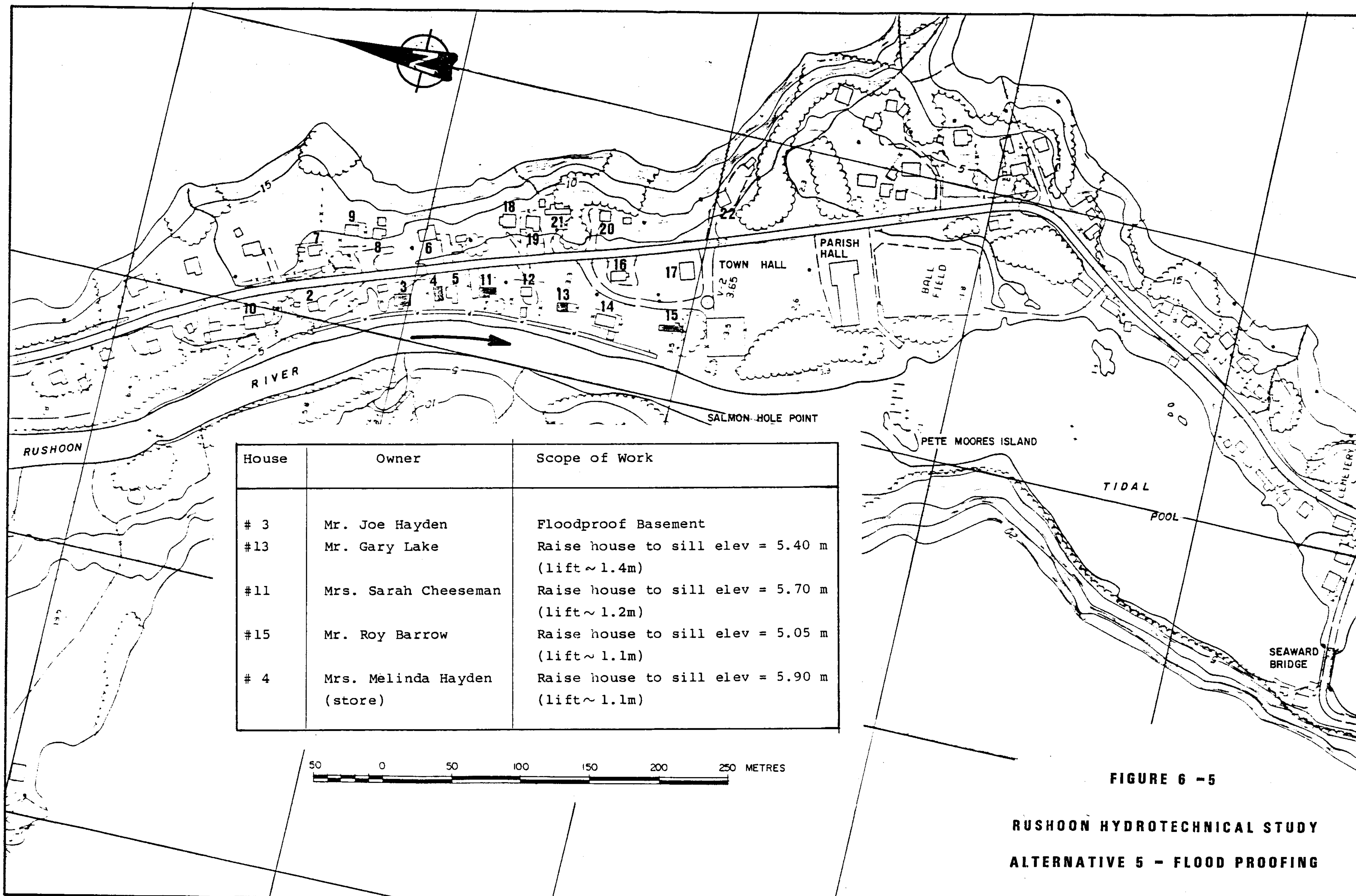
1: FILL WITH CLEAN COBBLES AND SMALL BOULDERS

2: TREAT TIMBER WITH APPROVED WOOD PRESERVATIVE

X-SECTION THROUGH TIMBER CRIB (TYPICAL)

RUSHOON HYDROTECHNICAL STUDY

ALTERNATIVE 3(b)–RAISE FENDER WALL



House	Owner	Scope of Work
# 3	Mr. Joe Hayden	Floodproof Basement
#13	Mr. Gary Lake	Raise house to sill elev = 5.40 m (lift ~ 1.4m)
#11	Mrs. Sarah Cheeseman	Raise house to sill elev = 5.70 m (lift ~ 1.2m)
#15	Mr. Roy Barrow	Raise house to sill elev = 5.05 m (lift ~ 1.1m)
# 4	Mrs. Melinda Hayden (store)	Raise house to sill elev = 5.90 m (lift ~ 1.1m)

FIGURE 6 -5

RUSHOON HYDROTECHNICAL STUDY

ALTERNATIVE 5 - FLOOD PROOFING

References

1. Technical Committee Canada - Newfoundland Flood Damage Reduction Program, "Terms of Reference for a Hydrotechnical Study of the Rushoon Area", St. John's, Nfld. (1984).

2. ShawMont Newfoundland Limited, "Rushoon Hydrotechnical Study - Field Report", prepared for Canada-Newfoundland Flood Damage Reduction Program, St. John's, (1985).

3. Murphy, F. Department of Environment (Nfld), "Rushoon Hydrotechnical Study - Observer's Scrapbook", St. John's, (1985).

4. Panu, U.S., Smith, D.A. and Ambler, D.C. Canada-Newfoundland Flood Damage Reduction Program, "Regional Flood Frequency Analysis for the Island of Newfoundland", Halifax and St. John's (1984).

5. Lear, W.H. and Day, F.A. Technical Report #697, Environment Canada - Fisheries and Marine Service, "An Analysis of Biological and Environmental Data Collected at North Harbour River, Newfoundland during 1959-1975", St. John's, (1977).

6. Bengtsson, L. IAHR Ice Symposium, "Experiences on the Winter Ice Regimes of Rivers and Lakes with Emphasis on Scandinavian Conditions", Quebec City, (1981).

7. Calkins, D.J. U.S. Army Corps of Engineers, Cold Region Research and Engineering Laboratory, "Accelerated Ice Growth in Rivers", Hanover, N.H. (ca. 1979).

References (Cont'd)

8. Bilello, M.A. CRREL Report 80-6-1980, U.S. Army Corps of Engineers, Cold Regions Research and Engineering Laboratory, "Maximum Ice Thickness and Subsequent Decay of Lakes, Rivers and Fast Sea Ice - Canada and Alaska", Hanover, N.H.

9. Tang, P.W. and Davar, K.S. Workshop on the Hydraulics of River Ice, "Forecasting the Initiation of Ice Breakup on the Nashwaak River, N.B.", Fredericton, N.B. (1984).

10. Condie, R. Nix, G.A. and Boone, L.G. Inland Waters Directorate, Environment Canada, "Flood Damage Reduction Program Frequency Analysis - (program FDRPFFA)", Ottawa, (1979).

11. Parisset, E. and Hausser, R. Transactions Engineering Institute of Canada, "Formation and Evolution of Ice Covers on Rivers", Vol. 5, No. 1961.

12. Pariset, E. Hausser, R, and Gagnon, A. A.S.C.E. Journal of Hydraulics Division, "Formation of Ice Covers and Ice Jams on Rivers", Nov. 1966.

13. Henderson, F.M. and Gerard, R. Proceedings IAHR Symposium on Ice Volume 1, pp. 277-287, "Flood Waves Caused by Ice Jam Formation and Failure", Quebec, (1981).

14. Hagen, V.K. International Commission on Large Dams, 14th Congress, "Re-Evaluation of Design Floods and Dam Safety", Rio de Janeiro (1982).

References (Cont'd)

15. Mrazik, B.R. Seventh Annual Meeting, Association of State Flood Plain Managers, "Hydrology and Hydraulics of Unique Flood Hazards", (1983).

16. Day, J.J. et al, Water Resources Research, "Evaluation of Benefits of a Flood Warning System", (1969), Vol. 5, No. 5,

17. Acres Ltd., Prepared for Govt. of Canada & Ontario Joint Task Force on Water Conservation Projects in Ontario, "Guidelines for Analysis", Vol. 2 Flood Damages, (1968).

18. Anon, Inland Water Directorate, Environment Canada, "Guidelines for the Benefit Cost Analysis of Flood Damage Reduction Projects", Ottawa, (1979).

19. Anon, Ontario Ministry of Natural Resources, "Ice Management Manual", Toronto, Ontario (1982).

20. Anon, Caterpillar Tractor Co. "Caterpillar Performance Handbook", Peoria, Illinois (1981).

21. Anon, Royal Architectural Institute of Canada, "Energy Economics and Life Cycle Costing", Ottawa (1980).

22. Johnson, H.D. Canadian Electrical Association Spring Meeting, "Discount Rate for Engineering Economic Studies by Private Utilities and Large Industries", Montreal, (1980).