INVESTIGATION OF 1989 WINTER FLOOD IN RUSHOON

CANADA NEWFOUNDLAND FLOOD DAMAGE REDUCTION PROGRAM

INVESTIGATION OF 1989 WINTER FLOOD IN RUSHOON

LHL-1065

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LaSalle, April 5, 1990

Technical Committee
Canada-Newfoundland Flood Damage Reduction Program
C/O Mr. O.P.J. Kelland, Minister
Department of Environment and Lands
P.O. Box 8700
St. John's (Newfoundland)
A1B 4J6

Reference : Investigation of 1989 Winter Flood
In Rushoon - Our ref. 86-e

Dear Members of the Committee,

We are pleased to submit our report of the "Investigation of 1989 Winter Flood in Rushoon".

We trust the findings of this study will explain some of the events leading up to the flood and the reasons for the record high water levels.

We have appreciated the opportunity of working with you on this interesting study and hope to be of service in the future.

Yours very truly,

Graham K. Holder, Eng.

GKH/cr

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1.0 INTRODUCTION

1.1 Scope of Work

The scope of work in this investigation comprises three aspects :

1. an update of hydrology (by ShawMont);
2. hydraulic studies to investigate modifications to the Rushoon ice fender wall to enhance protection from ice jam floods (by LaSalle Hydraulic Laboratory Ltd.) and;
3. assessment of stability of fender wall (by ShawMont).

1.2 Description of 1989 Winter Flood

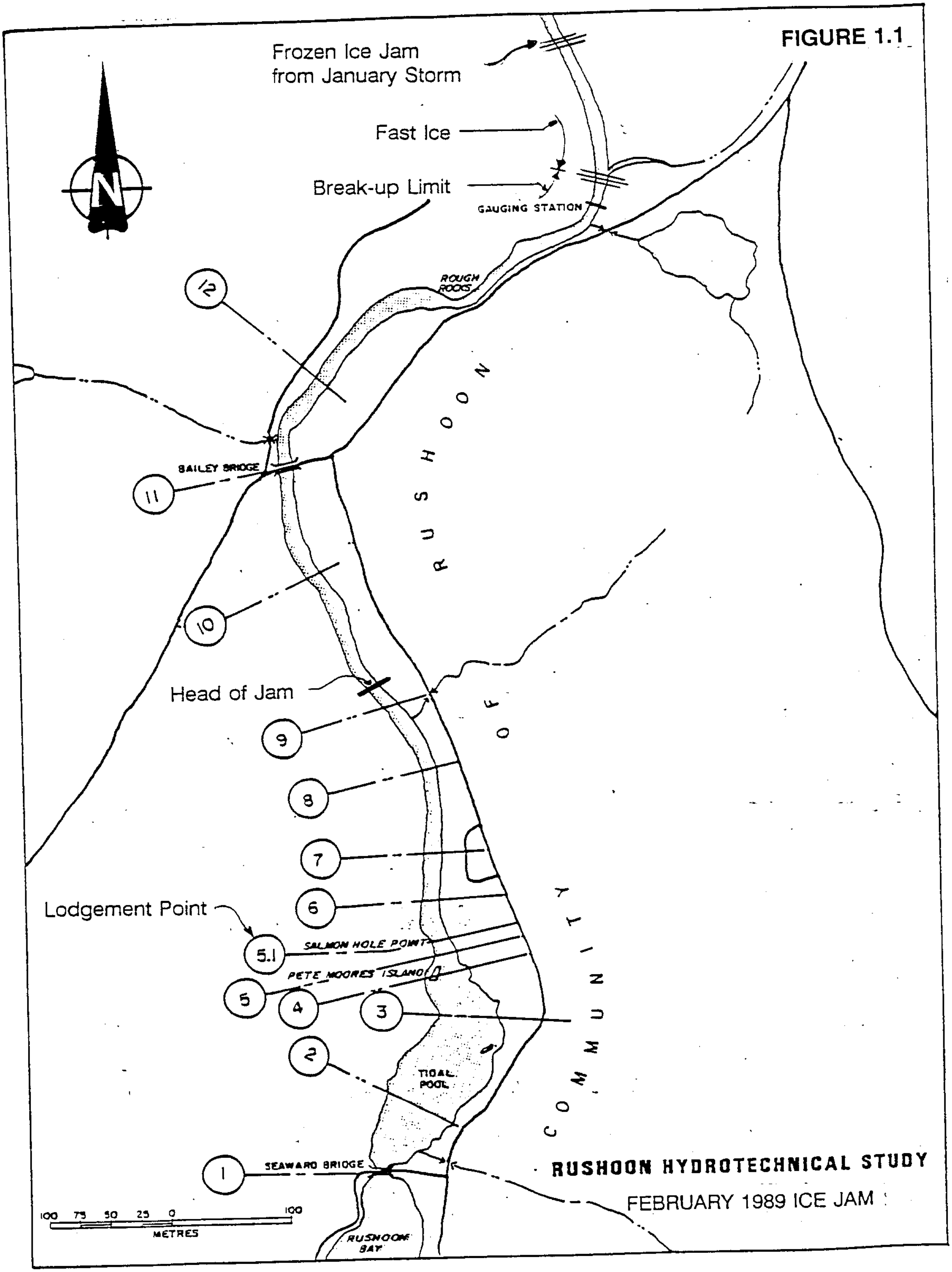
A winter ice jam flood occurred in late February 1989 following a warm spell with rain which caused the ice cover on Rushoon Brook to break up. As in past floods, the head of the jam was in the vicinity of Salmon Hole Point, just upstream of the Tidal Pool. The jam extended upstream from Salmon Hole Point (sta. 1 + 050), about 400 m to sta. 0 + 630.

The ice fender wall functioned as designed and retained the ice within the river channel. Beyond the lower end of the wall some ice pans were swept round on to the ball field and into the yards of the Community Centre and Municipal Building. Water levels in the community from this flood were higher than from any previous flood. Nonetheless, there was little damage since the houses at risk had been raised in line with recommendations of the 1985 flood study (by ShawMont and LaSalle Hydraulic Laboratory).

A contributing factor to flooding on this occasion may have been the occurrence of a minor ice jam in January 1989 which froze in place and restricted the channel capacity of the river. According to the Mayor of Rushoon, Mr. Frank Murphy, some ice accumulated in the lower part of Rushoon Brook within the community, as a result of a January breakup event. A second jam, located about 150 m above the location of the gauging station (Figure 1.1) also formed on this occasion. This jam froze solid and was not dislodged in the February 22, 1989 flood. The February 1989 ice jam was composed of ice released from just below the Gauging Station plus ice already in place.

This flood has been extensively documented by photographs and videotape taken by staff of the Department of Environment and Lands. Unfortunately, these records did not capture the flood at its peak (on February 22, 1989) but record the aftermath on February 23, 1989.

FIGURE 1.1



RUSHOON HYDROTECHNICAL STUDY

FEBRUARY 1989 ICE JAM

2.0 HYDROLOGY UPDATE

2.1 Objective

The purpose of this phase of the study was to estimate the magnitude of the peak flow that occurred during the ice jam flood of February 22, 1989 and to determine the portion of that flow escaping from the river channel and flowing through the community.

2.2 Approaches

Unfortunately no direct measurements of peak flow was made, since the gauge established during the 1984/85 studies was out of commission. Hence flow estimates had to be derived by less direct methods. The most appropriate approaches were judges to be :

- estimation of flood flow from observation of videotaped flow velocities and water levels, and;
- estimation of the peak flow by proration from adjacent gauges.

The calculations made to determine the estimates of the above are presented in Appendix 1.

2.3 Results

2.3.1 Estimation from Videotaped Observations

The videotape of flooding was carefully reviewed to identify sequences from which flow velocities could be assessed. Only one short sequence of a few seconds duration was found that was suitable for velocity determinations. This flow sequence shows water flowing in front of Mrs. Osbourne Hayden's shop.

Determination of flow magnitude involved the following steps :

- estimation of the time for floating debris to transit in front of the shop;

- measurement in the field of the shop front and side dimensions and ground cross-section;

- applying the measured water velocity to the measured cross-section area (with appropriate velocity corrections for depth) to determine the observed flow on February 23;

- adjusting the above flow for the maximum flood level (on February 22, 1989);

- estimating the magnitude of under ice flow in the river channel.

The following results were obtained :

		Observed	Expected
Observed Flow (Feb. 23)	=	5.50 m^3/s	
Manning's "n"	=	0.014	0.020
Peak Bypass Flow (Feb. 22)	=	49 m^3/s	35 m^3/s
Channel Flow (see Figure 2.1)			
Mean width	=	30 m	
Mean slope	=	0.0044	
Height of Jam	=	2.0 m (estimated)	2.7 m
Channel Flow	=	4 m^3/s	11 m^3/s

The maximum flood level was estimated to be 0.30 m above the crown of the road, based on the Mayor's observation that ... "the road was just passable for a jeep but too deep for a (conventional) motor car". Back calculation gave a Manning's "n" value of 0.014 which appears low. A value of 0.020 was expected based on data in "Open Channel Flow" by Ven T. Chow.

Under ice river channel flow was estimated using Beltaos's experience curve, Figure 2.1 which is applicable to rivers such as Rushoon. This estimate is an order of magnitude estimate only.

2.3.2 Estimation by Proration from Rattle Brook and Garnish River Flow Gauges

Proration factors were estimated by comparing measured flows for the May 5, 1989 spring flood on Rushoon Brook versus the coincident flow on Rattle Brook (Boat Harbour) and Garnish River. By way of confirmation, proration factors were also estimated by computing the ratios of the mean annual floods (Q_2) as determined by the regional flood frequency (RFF) formula. The proration factors determined by both methods were :

$$\frac{Q_{Rushoon}}{Q_{Rattle}} = 1.13$$

$$\frac{Q_{Rushoon}}{Q_{Garnish}} = 0.60$$

The peak flows on Rattle Brook and Garnish River were determined from water level gauge readings at the streamflow gauging stations on Rattle Brook and Garnish River. Unfortunately, records at both gauges for the February 22 to 23, 1989 flood event are affected by ice which reduces the reliability of flow determinations. Our best estimate indicated that the peak February 22, 1989 flows were :

Rattle Brook = 30 m^3/s whence Rushoon = 34 m^3/s
Garnish River = 67 m^3/s whence Rushoon = 40 m^3/s

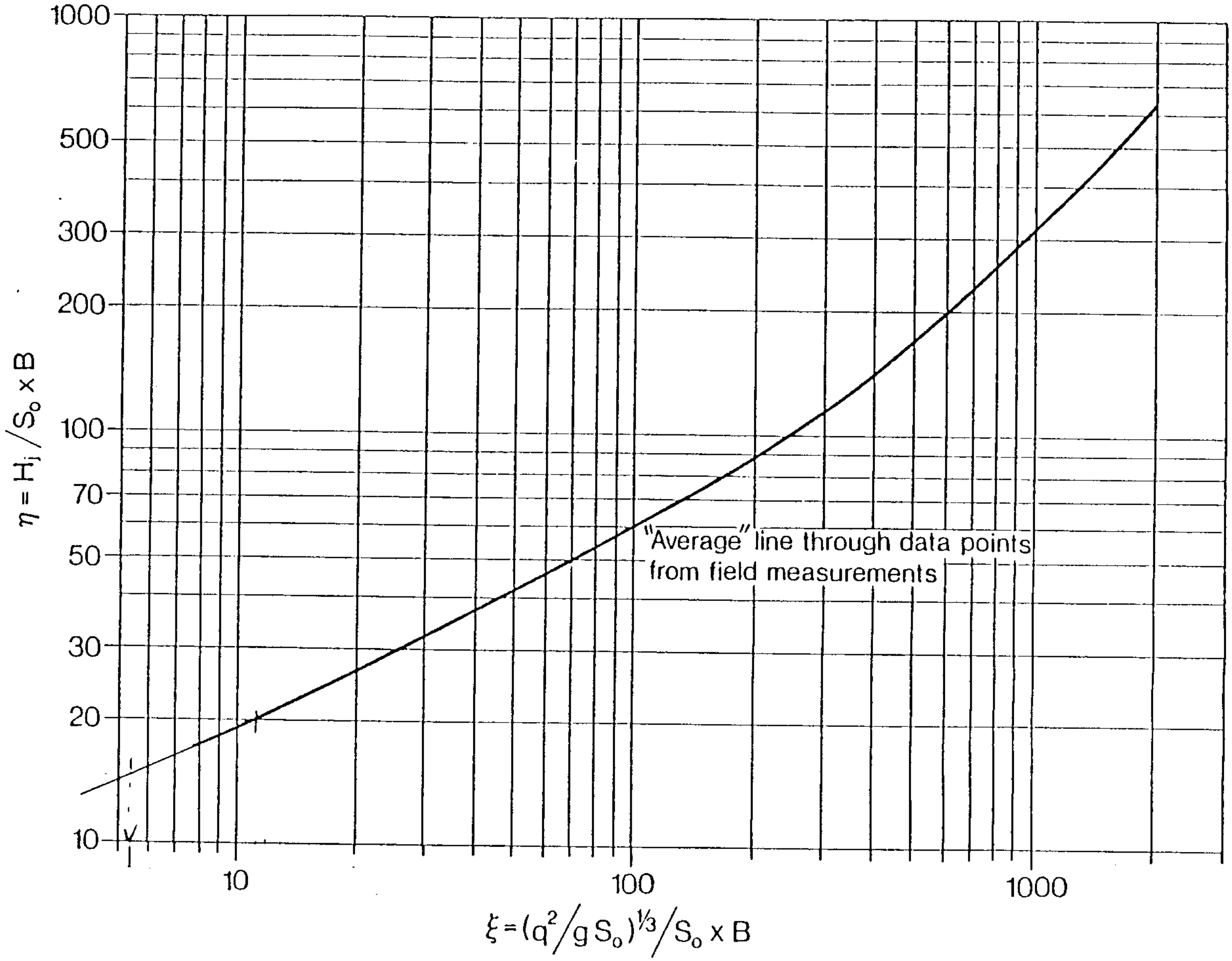
2.4 Observations and Conclusions

It was found to be impossible to make a precise estimate of the peak flood flow occurring during the ice jam flood of February 22, 1989 ... because of poor quality of available observations. Nonetheless rough agreement has been obtained between estimates derived from videotape observations and peak flow estimates obtain by proration from nearby streamflow gauges.

Our "best guess" estimate (see Appendix 1) is that the peak flow was about 40 m^3/s on February 22, 1989. Observations recorded on videotape in conjunction with personal observations of residents of Rushoon, indicate that a substantial proportion of the flow escapes from the river channel and flows through the community of Rushoon, approximately 35 m^3/s of a total flow of 40 m^3/s.

Backyard depressions have been in filled and levelled since the original studies in 1985. These changes in ground topography have reduced the flow capacity of the "escape" channel through the town, consequently, higher water levels are produced for the same bypass flows compared to the past (pre-backfill) conditions.

It is recommended that hydraulic model studies examine the river/ice regime for a flow of 40 m^3/s and that sensitivity studies consider the impacts of flow changes of ±33 1/3 %.



Source: "Lecture Notes on ice Jams" by S. Beltaos
National Water Research Institute, Burlington, Ont. (1984)

Figure 2.1: DIMENSIONLESS WATER DEPTH DUE TO AN EQUILIBRIUM, FLOATING, WIDE CHANNEL JAM vs. DIMENSIONLESS DISCHARGE

3.0 HYDRAULIC STUDIES

3.1 Introduction

Following the ice jam flood of February 22, 1989 at Rushoon and the subsequent re-evaluation of the hydrological events leading up to it, accumulation ice cover calculations were made to reproduce the observed conditions using the LaSalle Hydraulic Laboratory ice program.

During the flood, some ice pans were swept around the end of the fender wall, onto the ball field and into the grounds of the Municipal Hall. To prevent a reoccurrence in a future flood, tests were made to determine the possibility of extending the wall 100 m or 200 m further downstream.

3.2 Model Set-up and Calculation Methodology

The cross-sections used in the original flood study conducted in 1984/85 were adjusted to reflect changes in channel morphology (ShawMont ground survey of 1987) and infilling of backyard depressions (ShawMont ground survey of 1989) and an additional section (9.1) added to represent the upstream limit of the jam (Figure 3.1).

The hydrology update section of this report analyzed in some detail the flow conditions existing during the flood and gave "order of magnitude" values of the discharges contained within banks and through the town.

This analysis lent support to the conclusions made in the 1984/85 study that the Chezy "C" value of 35 used in the calculations was probably too smooth and that modifications to the calculation techniques were necessary. The decision was taken to simulate the 1989 flood as follows :

a) Assuming all the flow was contained within banks, find the maximum discharge prior to overtopping.

b) Calculate conditions in the river following overtopping.

c) Compare ice jam volumes with quantities of available ice.

d) Repeat (a) with the fender wall extended to section 5.1 and then 4.0.

In the model, this was achieved by in (a), raising the fender wall above its present elevation and all the land behind it to this level and in (b) taking the estimated channel discharge and calculating a new profile with the ice cover formed at the maximum flood discharge.

3.3 Simulation of the February 22, 1989 Flood

First tests were made to simulate the approximate water level profile taken during the February flood, using the discharge estimates and the same calculation techniques described in the original study (Reference 1). This was obtained by trial, running up from a discharge of 25 m^3/s to 45 m^3/s with a Chezy "C" value of 25 ($n = 0.05$). The results, given in Figure 3.1 show that the top of the fender wall was reached at a discharge of 40 m^3/s and at this discharge the full ice jam developed. It was then assumed that a breach occurred somewhere upstream, the flow came over the banks, and that the channel behind the fender wall contributed to the total flow section.

The calculations made in the hydrology update section showed that with the river over the banks, a flow of 35 m^3/s was estimated for overland flow behind the fender wall. Using the results from the calculations made with the fully developed ice jam, an attempt at calculating the water surface profile with the remaining flow of 5 m^3/s was made. The surface profile was impossible to obtain and the results indicated that the cover was grounded on the bottom and the flow percolated through the voids in the ice, which in fact corresponds to field observations.

Next an analysis of the ice volumes was made. The results showed that with a discharge of 40 m^3/s, an ice volume of 40,000 m^3 (including voids), was stored between the head of the jam at Salmon Hole Point and the leading edge. Translating this volume into the availability of ice upstream between Salmon Hole Point and the gauging station, based on a reach length of 1 600 m and channel width of 23 m and assuming 40 % voids in the ice jam, an ice thickness of approximately 0.7 m would have been required, which is unrealistic.

The contributing influence of the smaller January ice jam was then considered. It was estimated that when the jam formed, the ice thickness was approximately 30 cm thick and that nearly all the ice downstream of "The Falls" was available to form the jam. Hence the second jam, located 150 m upstream of the gauging station, was considered to have formed after the flood when the majority of the ice had passed downstream. Therefore, approximately 18,000 m^3 of ice was available to the jam. Next it was assumed that the jam was lodged at Salmon Hole Point and that the leading edge was at section 9 (see Figure 3.2). Computer simulations presented in Appendix 2 showed that a discharge of 15 m^3/s would result in an ice accumulation of this volume. All of this ice was still in place when the flood of February 22nd struck Rushoon.

At break-up in late February, the ice cover in the river had reformed and probably reached a thickness of about 40 cm. This ice was consequently broken up and washed downstream to add to the ice already in place since January. Therefore the cumulative effects of the January jam, the high break-up discharge and the additional supply of ice (estimated at 10 000 m^3) washed downstream from the reach below the gauging station, resulted in the large, thick volume of ice and the record high flood levels.

3.4 Fender Wall Extension

A series of tests were then carried out with the fender wall extended to section 5.1 and then 4.0 (Figure 3.2) with discharges of 40 and 45 m^3/s which cover the 1/100 ice season flood peak (Reference 1). The results, checked against the same conditions without the wall extension, showed water level differences of ±1 cm and ice volumes of ±1 %. Therefore no significant changes will occur if the wall is lengthened.

3.5 Observations and Comments

The results from the ice studies agreed closely with the estimated flood discharge of 40 m^3/s for the February ice jam flood event. At this flow, the thick accumulation ice cover that lodged at Salmon Hole Point and filled the brook up through the village consisted of broken ice from both the January and February break up. If the ice from the January break-up had not been present and assuming only ice from below the gauging station was available, the ice jam event would have been insignificant. It was the combination of the ice from both events that was the major factor.

Once the banks were overtopped, a preferential flow channel formed down through the village behind the fender wall. The discharge in the main channel decreased and the cover settled on the bottom. Flow percolated through the individual interstices in the cover.

Tests made with a lengthened fender wall showed that there will be insignificant changes in the water levels and ice volumes when compared to the tests results with the original wall length.

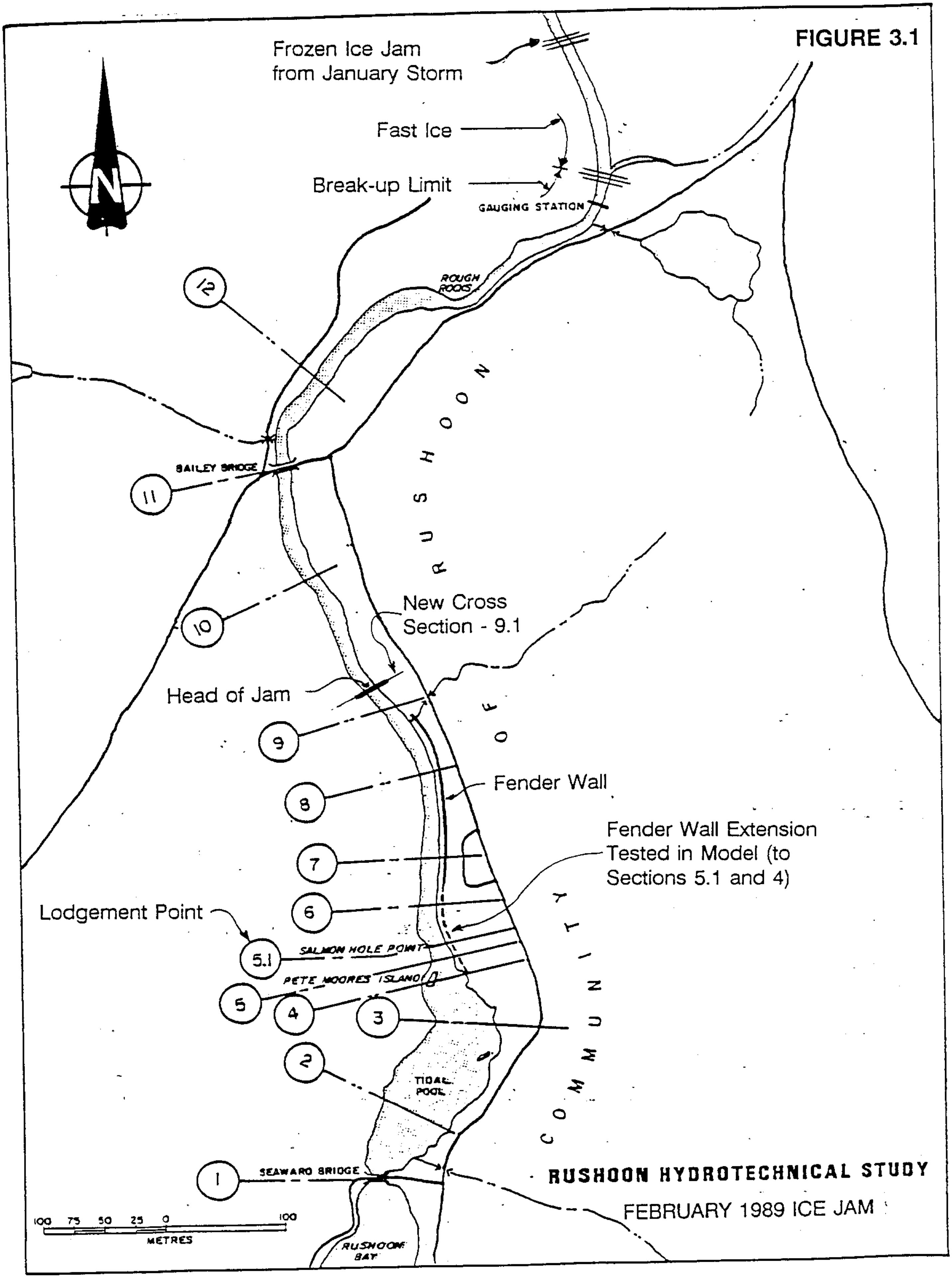
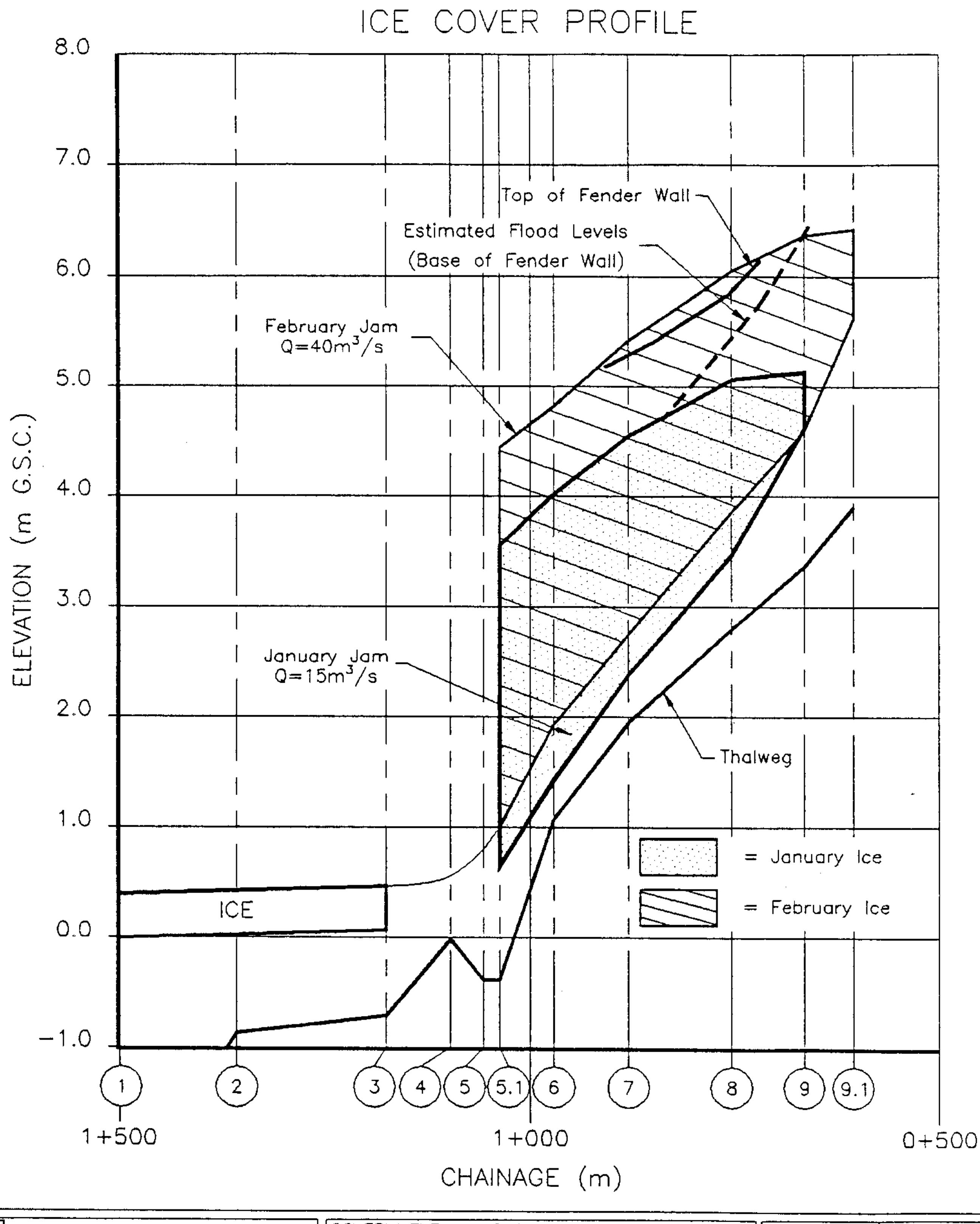


FIGURE 3.1
Frozen Ice Jam from January Storm
Fast Ice
Break-up Limit
GAUGING STATION
ROUGH ROCKS
12
BAILEY BRIDGE
11
RUSHOON
10
New Cross Section - 9.1
Head of Jam
9
OF
Fender Wall
8
Fender Wall Extension Tested in Model (to Sections 5.1 and 4)
7
6
Lodgement Point
5.1
SALMON HOLE POINT
PETE MOORES ISLAND
5
4
3
COMMUNITY
2
TIDAL POOL
1
SEAWARD BRIDGE
100 75 50 25 0 100
METRES
RUSHOON BAY
RUSHOON HYDROTECHNICAL STUDY
FEBRUARY 1989 ICE JAM



LASALLE HYDRAULIC LABORATORY LTD	GOVERNMENT of NEWFOUNDLAND AND LABRADOR DEPARTMENT of ENVIRONMENT AND LANDS	Contract: — — — —	Ref. L.H.L No. 86-e-01
INVESTIGATION of the 1989 WINTER FLOOD IN RUSHOON	RUSHOON 1989 FLOOD ICE COVER PROFILES	Drawn by: P. T.	Date: Jan. 1990
		Scale: — — — —	FIGURE: 3.2

4.0 CONCLUSIONS AND RECOMMENDATIONS

The February 22, 1989 ice jam flood at Rushoon resulted in water levels that were higher than in any previous flood. An hydrological study estimated the flood peak at about 40 m^3/s and calculations made using the ice program confirmed this value.

The results from the simulations showed that the major contributing factor to the flood was the large volume of ice available to form the jam. This volume consisted of ice from two ice jam events; ice from a relatively minor January break-up that was held up at Salmon Hole Point and ice from the February spring break up. The combination of these two ice volumes and the large February spring break-up water discharge, equal to a 1/100 ice season flood, resulted in the high flood levels.

Our opinion is that a flood of this type with a large ice volume comprising of two break-ups, coupled with a high break-up 1/100 year flood discharge, would be an extremely rare occurrence as two independent events took place. We would recommend that the fender wall be maintained at its present elevation. However, the presence of ice pans that has swept around the end of the fender wall into the Municipal Hall yard represents a potential hazard, and extending the wall downstream would offer some protection. The ice model showed that no change in the hydraulic regime will occur with the wall extended either 100 m or 200 m further downstream.

APPENDIX 1

FLOOD AND PEAK FLOW ESTIMATES

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FLOOD AND PEAK FLOW ESTIMATES

Estimate of Bypass Flow

Flow Velocity - 2.8 m/s From Analysis of Video Photos

X Section Area

7.8 x (0.15 + 0.19)/2	===>	1.33	
1.2 x (0.21 + 0.23)/2	===>	0.26	2.76 m^2
10.2 x (0.23)/2	===>	1.17	$y \approx 0.16$ $q \approx 0.44$

w = 17.5

Q = 2.76 x 2.8 ===> 7.7 m^3/s - February 23, 1989

Max Bypass Flow - February 22, 1989

X - Section Area

27.8 < 0.28 + 2.76 ===> 10.5 m^2 $y \approx 0.38$ m

From Manning's Equation

$$q/q_0 = (y/y_0)^{5/3}$$

$$q = 0.44 \times \left(\frac{0.38}{0.16}\right)^{5/3} \implies 6.86 \text{ m}^3\text{/s per m}$$

If max. water level was +0.20 m above level observed

27.8 x 0.18 + 2.78 ===> 7.76 m^3 $y \approx 0.28$ m

From Manning's Equation

$$q/q_o = 0.44 \times \left(\frac{0.28}{0.16}\right)^{5/3} \implies 1.12$$

$$Q = 1.12 \times 27.8 \implies 31.1 \text{ m}^3\text{/s}$$

Check on value of Manning's "n"

$$q = \frac{1}{n} \times y^{1.67} \times s^{0.5}$$

$$n = \frac{y^{1.67} \times s^{0.5}}{q}$$

therefore $n = (0.16)^{1.67} \times (0.0044)^{0.5} \div 0.44$

<u>therefore</u> <u>n = 0.007 seems too</u> low

<u>Recalculate</u> observed flow

- assume most flow carried in front and back yards, some in road ditch

at point of observation $y \approx 0.24$ n and $v = 2.8$ m/s

elsewhere use $v = 2.8 \times \left(\frac{y}{0.24}\right)^{5/3}$ ===> $30.3 \times y^{1.67}$

in backyard $\bar{y} = 0.16$

$v = 1.4$ m/s

$a = \frac{(0.18 + 0.14)}{2} \times 7.8$

$a = 1.28$ therefore $q = 1.28 \times 1.4$ ===> $1.77\ m^3/s$

in front yard $\bar{y} = 0.24$

$v = 2.8$

$a = \frac{(0.24 + 0.25)}{2} \times 1.2$

$a = 0.29\ m^2$ therefore $q = 0.29 \times 2.8$ ===> $0.81\ m^3/s$

in front yard $\bar{y} = 0.22$

$v = 2.41$ m/s

$a = 0.22 \times 2.5$

$= 0.55$ therefore $q = 0.55 \times 2.41 \Longrightarrow 1.33\ m^3/s$

$\bar{y} = 0.16$

$v = 1.42$

$a = 0.16 \times 2.5$

$= 0.41$ therefore $q = 0.41 \times 1.42 \Longrightarrow 0.58\ m^3/s$

$\bar{y} = 0.115$

$v = 0.81$ m/s

$a = 0.115 \times 2.5$

$= 0.29$ therefore $q = 0.29 \times 0.81 \Longrightarrow 0.23\ m^3/s$

$\bar{y} = 0.045$

$v = 0.17$ m/s

$a = 0.045 \times 2.5$

$= 0.113$ therefore $q = 0.113 \times 0.17 \Longrightarrow 0.02\ m^3/s$

in ditch:

$|\longleftarrow 3.5 \longrightarrow|$

$$q = \frac{1}{0.045} \times 1.14 \times 0.305^{2/3} \times (0.0044)^{1/2}$$

$\updownarrow$ 0.65

therefore $q = 0.76\ m^3/s$

$p = 3.73$	$A = 1.138$
$R = 0.305$	$n = 0.045$

---

therefore $Q_{TOT} = 5.50\ m^3/s$

<u>Check on Manning's "n"</u>

Use typical section - say backyard

$$n = y^{1.67} \times 5^{0.5}/q$$

therefore $n = 0.16^{1.67} \times (0.0044)^{0.5} \div 0.227$

" $n = 0.014$ versus expected 0.020

<u>Estimate of Maximum Flood Flood</u>

Maximum flood level in yard - 4.95 m

Flows:

- in backyard $\bar{y} = 0.38$ m

$v = 6.0$ m/s

$a = 0.38 \times 7.8$

$= 2.96$ therefore $q = 17.8\ m^3/s$

- in frontyard $\bar{y} = 0.47$ m

$v = 8.5$ m/s

$a = 0.47 \times 2.5$

$= 1.18$ therefore $q = 10.0\ m^3/s$

$\bar{y} = 0.41$

$v = 6.8$

$a = 1.03$ therefore $q = 7.0\ m^3/s$

$\bar{y} = 0.32$

$v = 4.5$

$a = 0.32 \times 5$

$= 1.6$ therefore $q = 7.2\ m^3/s$

- road $\bar{y} = 0.28$

$v = 3.6$

$a = 5.5 \times 0.28$ therefore $q = 5.5\ m^3/s$

- in ditch $q = \dfrac{1}{0.045} \times 2.02 \times 0.50^{2/3} \times (0.0044)^{1/2}$ ===> 1.9

a = 1.14 + 0.88
p = 4.0
R = 0.50
n = 0.045

Q_{TOT} = 49.4 m^3/s contraction effects $\dfrac{27.8 - 1 \times 0.1 \times 0.45}{27.8}$ (negligible)

If n ≈ 0.02, Q_{TOT} ≈ 34.5 m^3/s

Flow in channel under ice

Using Figure 3 from "Lecture Notes on Ice Jams" by S. Beltaos

$$\eta = \frac{H}{S_o B}$$

$$\xi = \frac{\left(\frac{q^2}{g S_o}\right)^{\frac{1}{3}}}{(S_o B)} \quad \sim \quad Q = \xi^{\frac{3}{2}} \cdot B^{\frac{3}{2}} \cdot g^{\frac{1}{2}} \cdot S_o^{2}$$

$S_o = 0.044\ m/m$

$B = 30\ m$

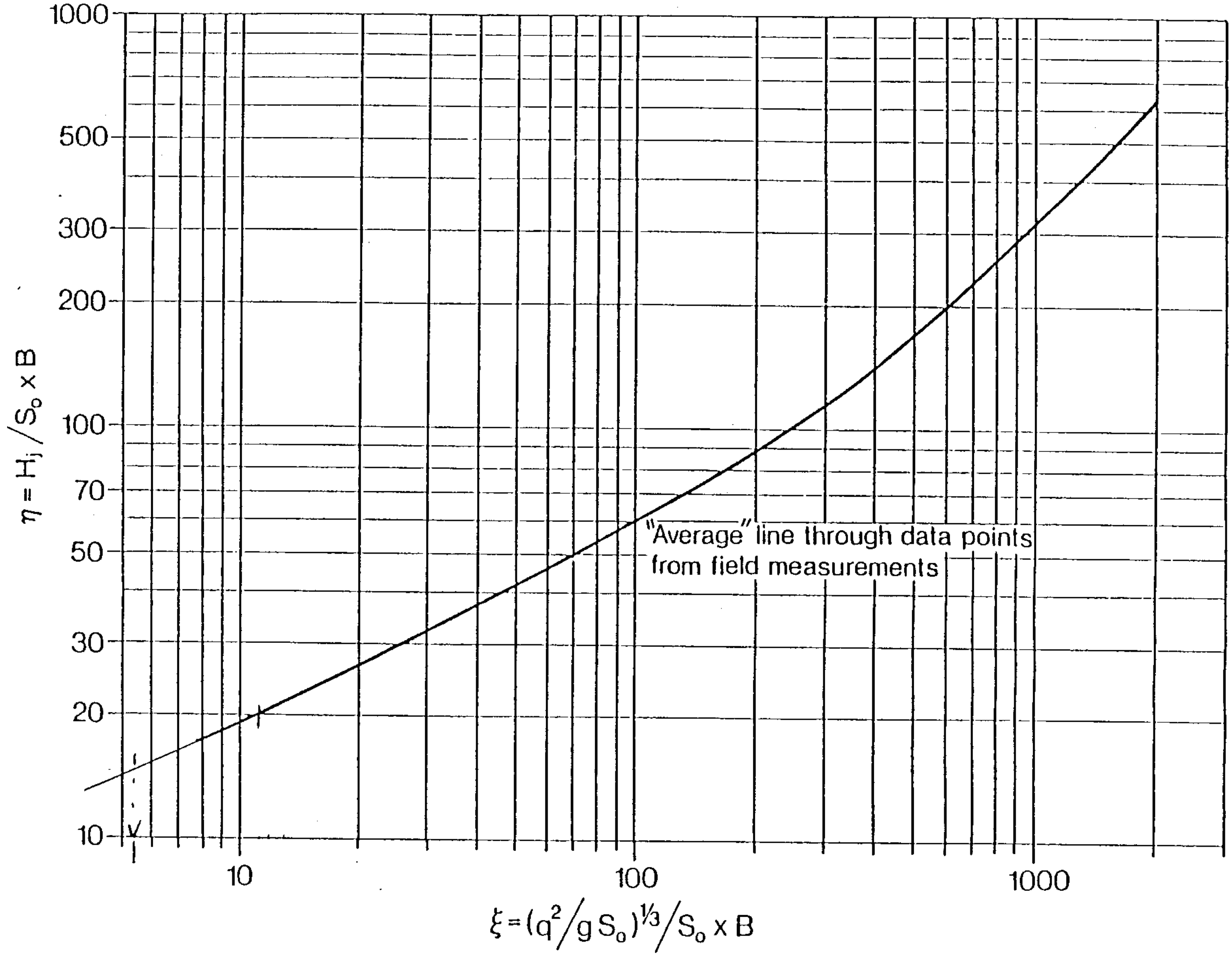
$H = 2\ m(est'd)$

$\eta \sim 2 + (0.0044 \times 30) \rightarrow 15$

$\xi \sim 5.5$

$\therefore q \sim (5.5 \times 30^{\frac{3}{2}} \times 9.81^{\frac{1}{2}} \times (0.0044)^2 \rightarrow 0.013$

$\therefore Q_{RIVER} \sim 3.8 m^3/s \sim 4 m^3/s$



Source: "Lecture Notes on ice Jams" by S. Beltaos
National Water Research Institute, Burlington, Ont. (1984)

Figure 2.1: DIMENSIONLESS WATER DEPTH DUE TO AN EQUILIBRIUM, FLOATING, WIDE CHANNEL JAM vs. DIMENSIONLESS DISCHARGE

$$\begin{aligned} \textit{If} \quad & H = 2.7 \\ & \eta = 2.7 \div (0.0044 \times 30) \quad \rightarrow \quad 20 \\ & \xi \sim 11 \\ & q \sim 0.36 \quad \textit{and} \quad Q_{RIVER} \sim 10.8 m^3/s \end{aligned}$$

From foregoing observations, range of flood flows could be

	49 + 11	===>	60 m^3/s
	35 + 4	===>	39 m^3/s
best guess	35 + 11	===>	45 m^3/s

<u>Estimation by Correlation with Nearby Flow Gauges</u>

(a) Rattle Brook

- by measurement, May 5, 1984

$$\frac{Q_{Rushoon}}{Q_{Rattle}} = \frac{24.1}{21.3} \implies 1.13$$

- by RFFA Formula [Upper Section Rushoon Brook]

$$\frac{Q_{Rushoon}}{Q_{Rattle}} = \frac{23.1}{27.8} \implies 0.83$$

$$\text{Adjustment } 0.83 \times \left(\frac{56}{49}\right)^{0.8} \implies 0.92$$

- by RFFA formula [Lower Section]

$$\frac{Q_{Rushoon}}{Q_{Rattle}} = \frac{30.4}{27.8} \Longrightarrow 1.09$$

- by drainage area ratio

$$\frac{Q_{Rushoon}}{Q_{Rattle}} = \left(\frac{56.2}{45}\right)^{0.8} \Longrightarrow 1.19$$

Use X 1.13 based on measurement.

Measured peak flow on Rattle Brook

- gauge reading 2.22 m therefore $Q = 19$
- gauge reading 2.5 " $Q = 30\ m^3/s$

therefore $Q_{Rushoon} \approx 1.13 \times 30$

$\approx 34\ m^3/s$

(b) Garnish River

By RFFA

$$\frac{Q_{Rushoon}}{Q_{Garnish}} \approx \frac{31.4}{53} \approx 0.59$$

By Measurement

$$\frac{Q_{Rushoon}}{Q_{Garnish}} \approx \frac{24.1}{37.0} = 0.65$$

Max. flood in February ≈ 67 m^3/s

therefore $Q_{Rushoon} \approx 39\ m^3/s$ ===> $44 m^3/s$

SUMMARY

Estimates of Peak Flood at Rushoon

From Observation - "best guess"	= 45 m^3/s
From Rattle Brook	= 34 m^3/s
From Garnish	= 39 m^3/s

For design use Q = 40 m^3/s ± 33 1/3%

APPENDIX 2

ICE COVER CALCULATIONS

1989 WINTER FLOOD IN RUSHOON
FEBRUARY JAM Q=40M3/S
ICE COVER CALCULATIONS

EF.: \GLOVER\DOEMOD2 .SEC

SECTION NO T (GLACE)	POSITION T/H	NI.EAU (D)GLACON	AIRE NCRI.FOR.	LARGEUR i limite	H.MOY TALWEG	VO VU	I DIST.AM.	n TO	F C
EBIT: 40									
ACCUMULATION									
5.10	5,100.00	4.44	117.70	29.54	3.98	0.34	0.01051	0.05000	0.05
3.44	0.86	1.15	3.56	0.00023	-0.38	1.64	66.00	0.00	25.0
6.00	6,000.00	4.82	152.82	44.67	3.42	0.26	0.00578	0.05000	0.05
2.89	0.84	0.59	3.95	0.00016	1.07	1.17	91.00	0.00	25.0
7.00	7,000.00	5.42	115.94	34.97	3.32	0.35	0.00665	0.05000	0.06
2.67	0.81	0.76	4.49	0.00112	1.95	1.33	126.00	0.00	25.0
8.00	8,000.00	6.05	107.21	36.60	2.93	0.37	0.00498	0.05000	0.07
2.19	0.75	0.61	5.39	0.00120	2.80	1.19	89.00	0.00	25.0
9.00	9,000.00	6.36	102.24	39.37	2.60	0.39	0.00350	0.05000	0.08
1.76	0.68	0.46	6.03	0.00110	3.36	1.04	60.00	0.00	25.0
SECTION DE CONTROLE									
9.10	9,100.00	6.42	98.90	46.98	2.02	0.40	0.00097	0.05000	0.09
0.80	0.40	0.17	6.42	0.00091	3.70	0.62	255.00	0.00	25.0

1989 WINTER FLOOD IN RUSHOON
FEBRUARY JAM Q=40M3/S
ICE COVER CALCULATIONS

SECTION	LARGEUR	EPAISSEUR	LONGUEUR	VOL. DE GLACE	VOL. CUMULATIF
5.1	29.542	3.436	33	3349.345	3349.345
6	44.672	2.889	78.5	10131.48	13480.83
7	34.966	2.672	108.5	10138.31	23619.14
8	36.599	2.189	107.5	8611.47	32230.61
9	39.369	1.756	74.5	5150.832	37381.44
9.1	46.977	.803	157.5	5938.959	43320.4

VOLUME (APPARENT OU REEL) EN PLACE = 43,320 METRES CUBES

1989 WINTER FLOOD IN RUSHOON
JANUARY JAM Q=15M3/S
ICE COVER CALCULATIONS

:EF.: \GLOVER\DOEMOD2 .SEC

SECTION NO T (GLACE)	POSITION T/H	NI.EAU (D)GLACON	AIRE NCRI.FOR.	LARGEUR i limite	H.MOY TALWEG	VO VU	I DIST.AM.	n TO	F C
'EBIT: 15									
ACCUMULATION									
5.10	5,100.00	3.55	88.13	28.25	3.12	0.17	0.00966	0.05000	0.03
2.90	0.93	0.58	2.86	0.00016	-0.38	1.17	66.00	0.00	25.0
6.00	6,000.00	4.02	94.94	33.56	2.83	0.16	0.00721	0.05000	0.03
2.59	0.92	0.43	3.30	0.00011	1.07	1.00	91.00	0.00	25.0
7.00	7,000.00	4.55	83.56	33.98	2.46	0.18	0.00579	0.05000	0.04
2.16	0.88	0.37	3.50	0.00071	1.95	0.93	126.00	0.00	25.0
8.00	8,000.00	5.06	67.74	33.86	2.00	0.22	0.00406	0.05000	0.05
1.59	0.80	0.29	4.49	0.00077	2.80	0.82	89.00	0.00	25.0
SECTION DE CONTROLE									
9.00	9,000.00	5.13	52.56	38.88	1.28	0.29	0.00075	0.05000	0.08
0.51	0.40	0.08	5.13	0.00070	3.36	0.44	60.00	0.00	25.0

1989 WINTER FLOOD IN RUSHOON
JANUARY JAM Q=15M3/S
ICE COVER CALCULATIONS

SECTION	LARGEUR	EPAISSEUR	LONGUEUR	VOL. DE GLACE	VOL. CUMULATIF
5.1	28.252	2.898	33	2701.537	2701.537
6	33.558	2.591	78.5	6824.514	9526.051
7	33.976	2.156	108.5	7948.335	17474.39
8	33.86	1.591	107.5	5790.209	23264.6
9	38.883	.511	74.5	1480.233	24744.83

VOLUME (APPARENT OU REEL) EN PLACE = 24,744 METRES CUBES

1989 WINTER FLOOD IN RUSHOON
FEBRUARY JAM Q=40M3/S;FENDER WALL EXTENDED TO SECTION 5.1
ICE COVER CALCULATIONS

EF.: 7.sec

SECTION NO T (GLACE)	POSITION T/H	NI.EAU (D)GLACON	AIRE NCRI.FOR.	LARGEUR i limite	H.MOY TALWEG	VO VU	I DIST.AM.	n TO	F C
EBIT: 40									
ACCUMULATION									
5.10	5,100.00	4.43	117.42	29.56	3.97	0.34	0.01046	0.05000	0.05
3.42	0.86	1.15	2.90	0.00128	-0.38	1.64	66.00	0.00	25.0
6.00	6,000.00	4.82	150.37	43.80	3.43	0.27	0.00591	0.05000	0.05
2.90	0.84	0.60	3.75	0.00043	1.07	1.19	91.00	0.00	25.0
7.00	7,000.00	5.42	115.94	34.97	3.32	0.35	0.00665	0.05000	0.06
2.67	0.81	0.76	4.49	0.00112	1.95	1.33	126.00	0.00	25.0
8.00	8,000.00	6.05	107.21	36.60	2.93	0.37	0.00498	0.05000	0.07
2.19	0.75	0.61	5.39	0.00120	2.80	1.19	89.00	0.00	25.0
9.00	9,000.00	6.36	102.24	39.37	2.60	0.39	0.00350	0.05000	0.08
1.76	0.68	0.46	6.03	0.00110	3.36	1.04	60.00	0.00	25.0
SECTION DE CONTROLE									
9.10	9,100.00	6.42	98.90	46.98	2.02	0.40	0.00097	0.05000	0.09
0.80	0.40	0.17	6.42	0.00091	3.70	0.62	255.00	0.00	25.0

1989 WINTER FLOOD IN RUSHOON
FEBRUARY JAM Q=40M3/S;FENDER WALL EXTENDED TO SECTION 5.1
ICE COVER CALCULATIONS

SECTION	LARGEUR	EPAISSEUR	LONGUEUR	VOL. DE GLACE	VOL. CUMULATIF
5.1	29.56	3.421	33	3337.584	3337.584
6	43.805	2.897	78.5	9963.091	13300.67
7	34.966	2.672	108.5	10138.31	23438.99
8	36.599	2.189	107.5	8611.47	32050.46
9	39.369	1.756	74.5	5150.832	37201.29
9.1	46.977	.803	157.5	5938.959	43140.25

VOLUME (APPARENT OU REEL) EN PLACE = 43,140 METRES CUBES

APPENDIX 3

STABILITY ASSESSMENT

CHECK ON FENDER WALL STABILITY

1. APPROACH

Stability analysis involves determination of the loads on the fender wall and then a check on the resistance of the structure. With the excepting of ice thrust, loads on the fender wall area readily determined. Resistance of the fender wall depends on its geometry, which was determined by measurement in the field.

2. DETERMINATION OF ICE THRUST

Ice thrust is determined from the features of the equilibrium ice jam formed in the river. The method outlined by Beltaos (1984) and (1988) will be used to approximate observed ice jam conditions in February 1989.

(i) Estimate bed and ice roughness

- from observation $k_b = 0.15$

- from Beltaos (1988) estimate

$$d_{i,84} = 1.43\ [1 - e^{-(0.73)(t - 0.15)}]$$

$$t = 2.5$$

therefore $d_{i,84} = 1.14$ (too high)

- use $k_i \approx 0.5$ m - from observation

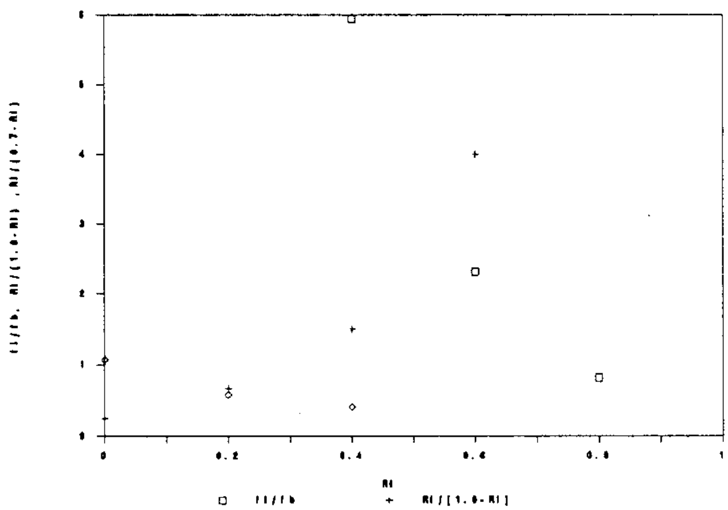
(ii) <u>Estimate f_i, f_b, and f_o</u>

- use Limerinos formula, Beltaos (1988), page D4.

$$f = [1.16 + 2 \log (R/d_{84})]^{-2}$$

- $d_{i,84} = 0.5$, $d_{b,84} = 0.15$ - try $R_i + R_b = 1.00$

R_i	$[1.0 - R_i]$	$R_i/[1.0 - R_i]$	f_i	f_b	f_i/f_b
0.2	0.8	0.25	7.54	0.15	50.3
0.4	0.6	0.67	1.07	0.18	5.94
0.6	0.4	1.50	0.58	0.25	2.32
0.8	0.2	4.00	0.41	0.50	0.82



$R_i = 0.65$ - therefore... $f_i = 0.52$
$R_b = 0.35$ - therefore... $f_b = 0.28$
therefore... $f_o = 0.40$

Alternatively [from Ashton 1986, page 333]

$k_o = k_b[(1 + k_r^{1/4})/2]^4$ $k_b = 0.15$
$k_o = 0.287$ $k_r = 0.5/0.15$

$f_o = 0.37$ versus 0.40

(iii) Suppose Q = 10 m^3/s

$$\therefore \quad q = 10 \div 30 \rightarrow 0.333\ m^3/s \quad m$$

$$\therefore \quad h = 0.63 f_o^{\frac{1}{3}} \left(\frac{q^2}{g S_o} \right)^{\frac{1}{3}}$$

$$\therefore \quad h = 0.63 \times (0.40)^{\frac{1}{3}} \times \left(\frac{0.333^2}{9.81 \times 0.0044} \right)^{\frac{1}{3}}$$

$$\therefore \quad = 0.64\ m \quad versus \quad 1.00\ m$$

By trial and error:

h	f_o	h^*
1.00	0.40	0.64
0.80	0.48	0.67
0.70	0.56	0.71

===>

R_i	$(0.7-R_i)$	$R_i/(0.70-R_i)$	f_i	f_b	f_i/f_b
0.2	0.5	0.40	7.54	0.21	35.9
0.4	0.3	1.33	1.07	0.32	3.34
0.5	0.2	2.50	0.74	0.50	1.48

$R_i = 0.46$ $\quad$ $R_b = 0.24$

$$f_i/f_o = \frac{0.46}{0.35} \Longrightarrow 1.31$$

(iv) Calculate H_j

$$\xi = \left(\frac{q^2}{q S_o} \right)^{\frac{1}{3}} \div (S_o B)$$

$$= \left(\frac{0.333^2}{9.81 \times 0.0044} \right)^{\frac{1}{3}} \div (0.0044 \times 30)$$

$$\therefore \eta = 1.28$$

$$\therefore \frac{H_j}{S_o B} = 0.63 f_o^{\frac{1}{3}} \times \xi + \frac{5.75}{\eta} \left[1 + \sqrt{1 + 0.11 \eta f_o^{\frac{1}{3}} \left(\frac{f_i}{f_o} \right) \xi} \right]$$

$$\therefore \frac{H_j}{S_{oB}} = 0.63 \times (0.56)^{\frac{1}{3}} \times 10.4 + \frac{5.75}{1.28} \left[1 + \sqrt{1 + 0.11 \times 1.28 (0.56)^{\frac{1}{3}} \times 1.31 \times 10.4} \right.$$

$$\therefore \frac{H_j}{S_{oB}} = 5.40 + 11.71$$

$$\therefore H_j = 17.1 \times 0.0044 \times 30 = 2.3m$$

Observed H_j was between 2.0m and 2.7m (OK)

(v) Following Beltaos (1984), page 11

$$\bar{\sigma}_z = \tfrac{1}{2}(1 - p)(1 - S_i)S_i\rho g t$$

where p = porosity ≈ 0.4
S_i = specific gravity of ice ≈ 0.92
t = thickness of ice cover ≈ 2.3 m

therefore $\bar{\sigma}_z = \tfrac{1}{2}(1 - 0.4)(1 - 0.92) \times 0.92 \times 1000 \times 9.81 \times 2.3$ Pa

therefore $\bar{\sigma}_z$ = 498 Pa

Now $\bar{\sigma}_z = K_x \times \bar{\sigma}_z$,

where K_x = coefficient of passive resistance

$$= \frac{1 + \sin\phi}{1 - \sin\phi}$$

$= 4.27 \quad (\phi = 38.3°)$

therefore $\sigma_x = 4.27 \times 498$ ⟶ 2130 Pa

Also $\sigma_y = K_o \times \sigma_x$

where K_o = coef. of lateral thrust ≈ 0.63

therefore $\sigma_y = 0.63 \times 2130$

therefore σ_y = 1340 Pa [≡ 28 psf]

Including for impact use 1340 x 1.33

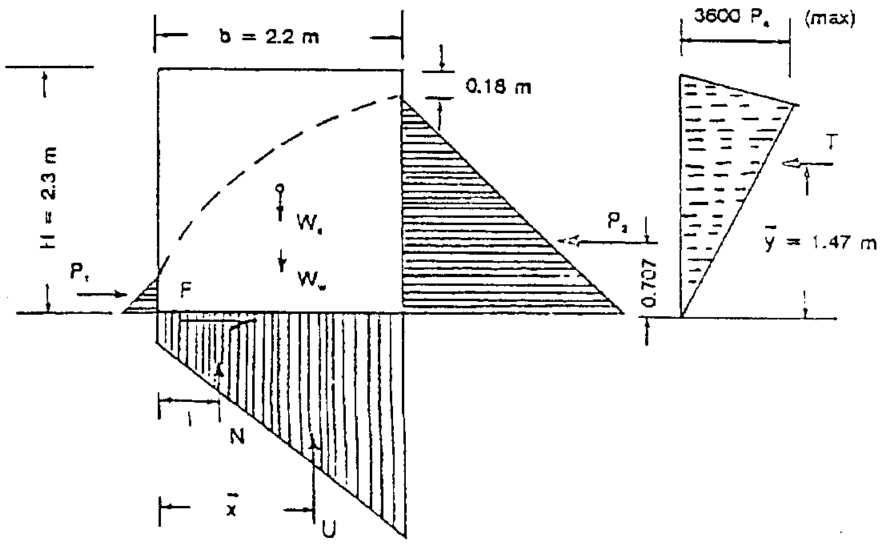
therefore σ_y = 1800 Pa [≡ 37.6 psf]

3. STABILITY CALCULATIONS

(a) Material Properties

Specific gravities - ice	= 0.92
- stone	= 2.65
Porosity, n	= 30%
Effective friction angle, ϕ	= 35°(for sliding analysis)
Mean stone size	= 10 cm (≡0.100 m)
Bulk density	= (1.00 - 0.30) x 2.65 x 1000 x 9.81)
	= 18200 n/m³ [≡116 lbs/ft³]

(b) Loads



$$P_2 = \frac{\rho g y_2^2}{2} \times (1 - r)$$

- min contact area would be small, probably governed by crushing strength of ice in the limit. Conservatively (1 - r) = 1

therefore $P_2 = \frac{1000 \times 9.81 \times 2.12^2}{2}$ ===> 22000 N

$T = 1800 \times 2.3$ ===> 4100 N

<u>For P_1</u> - Assume flow at downstream face is critical and solve using Figure 2.4 from Stephenson.

$b = 2.2$, $k \approx 3$, $d \approx 0.1$ m and $x = 0$ (upstream face)

therefore $(b - x).3k/d = 198$, but $\left(\frac{Y}{y_c}\right)^3 = 198$

" $y_c = \frac{y}{(198)^{1/3}}$ ===> $\frac{2.12}{5.83}$

" $y_c = 0.36$ m

" $P_1 = \frac{1000 \times 9.81 \times 0.36^2}{2}$ ===> 650 N

" $C_1 = \left(\frac{0.36 + 2.12}{2}\right) \times 1000 \times 9.81 \times 2.2$ ===> 26800 N

Also

$W_s = 2.2 \times 2.3 \times 18200$ ===> 92100 N

and

$W_w = (2.12 + 0.36) \times 0.5 \times 2.2 \times 0.30 \times 1000 \times 9.81$ ===> 8000 N

(c) Check sliding:

$\underline{\Sigma F_V}$: $N = W_s + W_w - U$

$= 92100 + 8000 - 26800$

Therefore - $\underline{N = 73300\ N}$

$\underline{\Sigma F_H}$: $F = P_2 + T - P_1$

$= 22000 + 4100 - 650$

therefore $\underline{F = 25450\ N}$

Frictional sliding resistance:

$Q = N\tan\phi$

$= 73300 \times \tan 35°$

$Q = 51300\ N$

Factor of safety against sliding:

$= 51300/25450$

$= 2.0$ OK[1]

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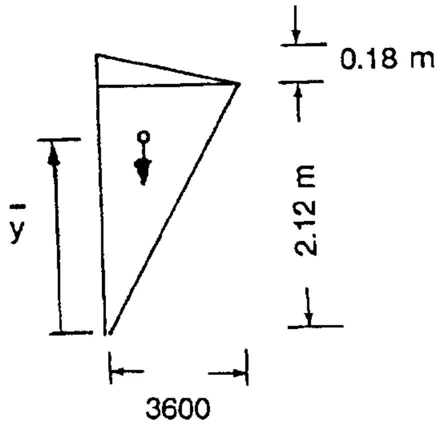
[1] All assumptions are conservative, loads P_2, H and ϕ is low.

(d) Check for Overturning

Find $\bar{y}$



$A\bar{y} = \Sigma a_y$

$[4140\ \bar{y}] = 2.12 \times 3600 \times 1/2 \times 2/3 \times 2.12$
$0.18 \times 3600 \times 1/2 \times (2.12 + 0.06)$

$= 5393 + 706$

Therefore $\bar{y} = 1.47$ m

Find $\bar{x}$

$A\bar{x} = \Sigma ax$

Therefore $26800\bar{x} = 0.36 \times 1000 \times 9.81 \times 1/2 \times 2.2 \times 1/3 \times 2.2$ ===> 2800
$+$
$2.12 \times 1000 \times 9.81 \times 1/2 \times 2.2 \times 2/3 \times 2.2$ ===> 33600

Therefore $\bar{x} = 36400 \div 26800$ ===> 1.36 m

Take Moments about downstream toe of crib

$N * l = (W_s * x_1) + (W_w * x_2) + (P_1 \times y_1) - (T * \bar{y}) - (P_2 * y_2) - (U\bar{x})$

therefore 73300 x l = 92100 x 1.1 ===> + 101300
8000 x 1.3 6 ===> + 10900
650 x 0.12 ===> + 100
- 22000 x 0.707 ===> - 15600
- 4100 x 1.47 ===> - 6000
- 26800 x 1.36 ===> - 36400

therefore 733000 x l = 54300 whence l = 0.74 m > $\frac{2.2}{3}$ [0.73 m]

Therefore, resultant will fall within the middle third, thus ensuring the "no tension" condition for stability.

(e) Check Foundation

Cribs are founded on original ground, which appears to be a relatively compact silty, sandy-gravel. For this material $\phi = 30°$ and allowable bearing pressures $p = 180\ kP_a$ (dry) and $90\ kP_a$ (wet) are reasonable.

Maximum bearing pressure M at toe of structure:

$$f_b = \frac{N}{A} \pm \frac{Ne}{Z}$$

Where:

Net axial load, $N = 73300$ N

Eccentricity, $e = 1.1 - 0.74 \implies 0.36$ m

Section Modulus, $Z = \frac{(1) \times 2.2^2}{6} \implies 0.807\ m^3/m$

Area, $A = (1) \times 2.2 \implies 2.2\ m^2/m$

therefore $f_b = \frac{73300}{2.2} \pm \frac{73300 \times 0.36}{0.807}$

" $f_b = 33300 \pm 32700$

" f_b max $= 66000\ N/m^2 < 90k$ Pa OK

f_b min $= 600\ N/m^2 > 0$ OK

Sliding resistance = 733000 tan 30°

= 42300 N

Factor of safety = 42300/25450

= 1.7 OK

(f) Containment

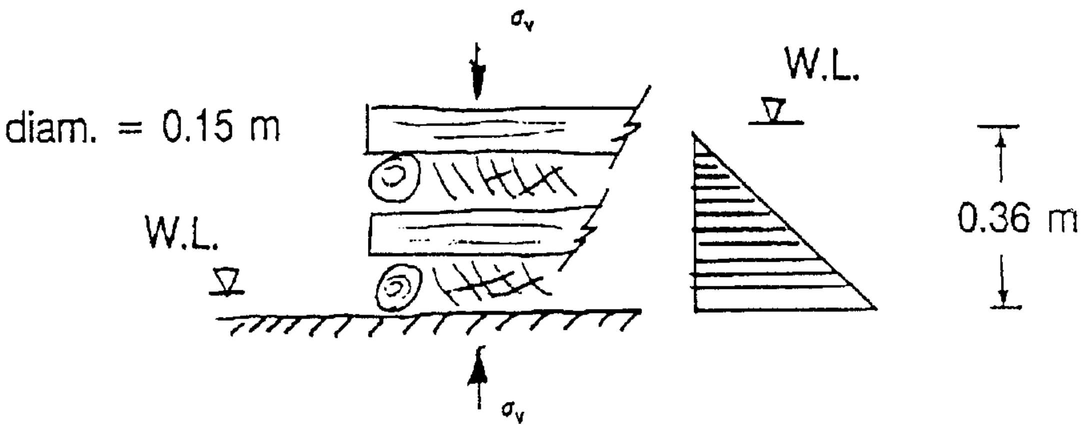
Materials

Logs - untreated round spruce
- diam = 0.15 m (min)

Allowable stresses (construction grade):
- f_b = 5200 kP_a
- T_h = 440 kP_a [Timber Construction Manual C.I.T.C. 1959]

Spikes[2] Standard manufacture and strength (Galvanized)
Diameter = 9.5 mm (3/8")
Lengths = 254 mm and 305 mm

Loading Condition



Consider downstream load
- uniform load from hydrostatic and rock pressures, use ϕ = 40° per Figure 3.8, Stephenson (1979).

$$= \frac{(0.36 + 0.21)}{2} \times 1000 \times 9.81 \times 0.15 \quad \Longrightarrow \quad 419$$

$$+ 39000 \times \left(\frac{1 - \sin 40^\circ}{1 + \sin 40^\circ}\right) \times 0.30 \quad \Longrightarrow \quad 2544$$

therefore W = 2960 N/m

---

[2] Details on spikes obtained from F. Murphy, Mayor of Rushoon, by telephone - Nov 24, 1989.

Check bending

$$M_{max} = \frac{\omega\, l^2}{8}$$

$$= \frac{2960 \times 2.0^2}{8} \rightarrow 1480 m.N$$

$$Z = \frac{\pi D^3}{32}$$

$$= \frac{\pi \times (0.15)^3)}{32} \rightarrow 3.31 \times 10^{-4} m^3$$

$$\therefore f_b = \frac{M_{max}}{Z}$$

$$= \frac{1480}{3.31 \times 10^{-4}} \rightarrow 4500\ kPa < 5200\ kPa$$

(OK)

Check horizontal shear

$$\tau_H = \frac{VQ}{I_b}$$

$$Max\ Shear,\ V = \frac{wl}{2}$$

$$V = \frac{2960 \times 2.0}{2} \rightarrow 2960\ N$$

$$Q = \frac{\pi r^2}{2} \times r\left(1 - \frac{4}{3\pi}\right) \rightarrow 0.904\ r^3$$

$$I = \frac{\pi\ r^4}{4} \rightarrow 0.785\ r^3$$

$$b = r$$

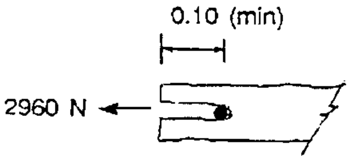
c = r(1-4/3π)

$$\therefore \tau_H = \frac{2960 \times 0.904 \times (0.075)^3}{0.785\ (0.075)^4 \times 0.150}$$

$$\therefore \tau H = 304\ kPa < 440\ kPa$$

(OK)

Check pull out on tie back log



Pullout resistance $= \tau_{all} \times 2 \times 0.15 \times 0.10$

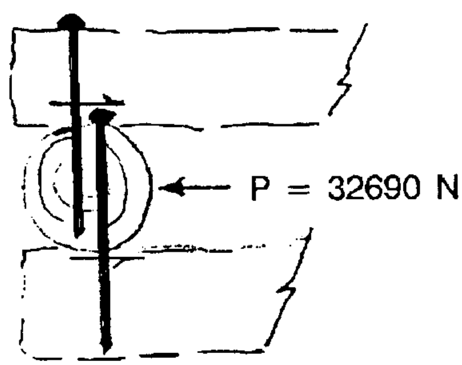
$= 440000 \times 2 \times 0.15 \times 0.10$

$= 13200\ N > 2960\ N$ (OK)

Check Adequacy of Spiked Connection

Following U.S. National Design Specifications selected by Garfinkel (1973), Table A.6.19, page 501

- allowable lateral load = 310 lb x 0.9 [≡1250 N]
- embedment to develop full capacity = 13 ϕ
  = 13 x 9.5 ===> 124 mm OK
- assume two x single shear values for connection constructed as below :



∴ *Allowable capacity* = 2 × 1250

= 2500 *N versus* 2960 *N*

Overstress = 18% *overstress* (OK)

Garfinkel (1973) page 173 suggests that the actual factor of safety for the recommended load on a nailed connector ≈ 6.0.

Where standard spikes are used that meet specified dimensions and strength requirements, connector failure would normally occur in wood before capacity of spike is reached. It is thus unnecessary to analyze stresses in spike.

4. CONCLUSIONS

The stability of the Rushoon fender wall was investigated taking into account ice, hydraulic, gravity, uplift and rode loads. In addition, the adequacy of the foundation and containment elements were analyzed. The analysis considered the stability of the wall at its greatest height ($\approx$ 2.3 m) at the downstream end of the wall.

The wall was found to be stable although some design criteria are only just met, as noted below:

- overturning resistance is at its acceptable limit since the resultant just falls within the middle third,

- the capacity of nailed connections are marginally lower than the recommended standard.

In areas where the wall is lower than 2.3 m, its factor of safety is improved.

The following recommendations should be noted:

(i) In the event that the fender wall is raised to a height above 2.3m, a wider base should be provided to ensure stability.

(ii) A few cribs were found to be incompletely filled. These cribs should be "topped up" as soon as possible.

(iii) A few cribs have been constructed with a filling of relatively small shingles and cobbles. There is a risk that some of this material will fall out between the timbers of these cribs. Where this happens, the lost material should be replaced with larger stones (10 cm +) and the cribs refilled.

(iv) Based on experience with timber crib dams, it is believed that a service life of about fifteen years is all that can be expected from outdoor construction with untreated timber. Thus, the fender wall will probably require replacement by the year 2000. The wall should be maintained to ensure its contained integrity.

(v) Should further extensions be added to the fender wall, it is suggested that such extensions be constructed to a design and under the supervision of a professional engineer.

<u>Analysis by</u>: P.C. Helwig, P.Eng.
Nov. 25, 1989

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3. Ashton, G.D. (1986) - edition, "River and Lake Ice Engineering", Water Resources Publication, Fort Collius, Colorado

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5. Canadian Institute of Timber Construction (1959), "Timber Construction Manual", C.I.T.C., Ottawa

6. Garfinkel, G. (1973), "Wood Engineering", Souther Forest Products Association, New Orleans, Louisiana