Canada-Newfoundland
Agreement Respecting
Water Resource Management

Flood Risk Mapping Study

Goulds, Petty Harbour and Ferryland

Volume I - Main Report

Government of
Newfoundland
and Labrador

Department of
Environment

Environment Canada
Environnement Canada

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29 March 1996

Government of Newfoundland and Labrador
Department of Environment
Water Resources Management Division
P.O. Box 8700
St. John's, Newfoundland
A1B 4J6

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

Re: Flood Risk Mapping Study: Goulds, Petty Harbour and Ferryland

Dear Dr. Ullah:

We are pleased to submit the attached Report and Technical Appendices for this interesting and challenging study. The main report provides details about the watersheds, hydrologic modelling of flood flows, and flood levels for the 1:20 year and 1:100 year flood conditions at Petty Harbour and Goulds. At Ferryland, we have provided the surge and wave uprush levels associated with the 1:20 and 1:100 year sea levels. The Technical Appendices provide background analysis and details about the modelling.

It has been a distinct pleasure working with your staff and we particularly wish to thank Mr. Bob Picco for his assistance and constructive comments throughout the course of this study.

Yours very truly,

BAE NEWPLAN GROUP LTD.

for
R. Noseworthy, P.Eng.
Project Director

encl.

c.c. D.B. Hodgins

Flood Risk Mapping Study

Goulds, Petty Harbour and Ferryland

Volume I - Main Report

FLOOD RISK MAPPING STUDY OF THE
GOULDS, PETTY HARBOUR AND FERRYLAND

Table of Contents

Page

VOLUME I - MAIN REPORT

FLOOD RISK MAPPING STUDY OF THE GOULDS, PETTY HARBOUR AND FERRYLAND

Table of Contents

Page

FLOOD RISK MAPPING STUDY OF THE GOULDS, PETTY HARBOUR AND FERRYLAND

Table of Contents

VOLUME II - TECHNICAL APPENDICES

List of Tables

Page/Follows Page

FLOOD RISK MAPPING STUDY OF THE
GOULDS, PETTY HARBOUR AND FERRYLAND

Table of Contents

Page/Follows Page

List of Figures

Follows Page

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The study team is particularly appreciative of the assistance and guidance provided by Mr. Bob Picco, Senior Project Engineer with the Department of Environment, and the members of the Department's review team - Dr. Abdel-Razek and Mr. Stephen Dyke.

We also wish to thank the local flood observers at Goulds. They gave their time to better conditions in their community and we thank:

- Mrs. Ryan
- Mr. James
- Mr. Howlet
- Mr. Bennett
- Mr. Clarke

We further wish to acknowledge the technical assistance provided by our SNC-Lavalin affiliates - Fenco MacLaren Inc. for their analysis of flood flows and levels (Mr. Doug Hodgins), and MacLaren Plansearch Inc. for their analysis of water levels at Ferryland (Dr. Bassem Eid).

1.0 SUMMARY OF FINDINGS, CONCLUSIONS AND RECOMMENDATIONS

1.1 Introduction

The Goulds, Petty Harbour, and Ferryland communities are located on the east coast of the Avalon Peninsula, south of St. John's.

The Goulds study area is fed by five watercourses: Fourth Pond Brook, Doyles River (north and west branches), Dirty Bridge Brook, Cochrane Pond Brook, and Raymond Brook. All these watercourses contribute flows to Third Pond which forms the downstream boundary of the Goulds study area. It is evident from review of the literature and local reports that flooding in the Goulds area has been frequent and, to some degree, is an annual occurrence. Although flooding is regular, its effects appear limited to periodic roadway overtopping, some basement flooding, and regular flooding of the barns and parking lot at the Avalon Raceway. There has also been loss of life at Goulds when a young child drowned (in April 1964) after she fell into a swollen stream.

The Petty Harbour study area is traversed by the Petty Harbour River. Flows from the Petty Harbour River are regulated by a hydropower generating facility operated by the Newfoundland Light and Power Company. Both the Goulds and Petty Harbour study areas belong to the same watershed/river system, with the latter located on the downstream side. At Petty Harbour, there has been riverine flooding from rainfall or snowmelt-rainfall events in the early 1940s and in 1986, and flood damage from heavy seas in 1955 and possibly in 1977. There is also information from local residents that there is flood damage each year from small channels which become filled with snow and ice and cause some basement flooding.

Ferryland is located some 35 km south of Goulds and Petty Harbour. Unlike the other study areas which are affected by riverine flood processes, it is a coastal area and flooding is dictated by oceanographic/coastal processes. Strong winds generated exceptionally high seas and flooding in 1989 and 1955, and may have caused damage in 1977.

The overall thrust of this study is to assess the potential for flooding in the three communities by defining the flood risk areas associated with the 1:20 year and 1:100 year recurrence interval flood levels.

1.2 Summary

1) The QUALHYMO hydrologic computer model was employed to simulate and recreate historical runoff volumes, hydrographs and peak flows throughout the Goulds study area.

QUALHYMO is a watershed-scale continuous simulation model which incorporates historical meteorological data with physical watershed parameters (such as soil type, land use, etc.) and, more importantly, accounts for the accumulation and ablation of snow pack depths on the ground.

2) Petty Harbour flood flows were determined by statistical analysis of maximum daily peak flows from Water Survey of Canada records for the years 1963-1993.

3) The peak flow rates determined for various locations in the study area are as follows:

River	1:20 year Flow m^3/s)	1:100 year Flow (m^3/s)
Cochrane Pond Brook[1]	72	91
Doyles River[1]	68	84
Raymond Brook[1]	61	83
Dirty Bridge River[1]	9.6	12.6
Fourth Pond Brook[1]	16.7	13.8
Petty Harbour[2]	76	101

[1] Flood Frequency Peak Flow Estimates based on QUALHYMO simulated annual peaks.
[2] WSC Records at Petty Harbour River

4) A statistical analysis was also performed on the annual peak outflows from Third Pond to derive the 1:20 year and 1:100 year return period outflow rates. These were 147 m^3/s and 168 m^3/s, respectively. These translate to water levels of 69.6 m and 69.78 m for the 1:20 year and 1:100 year return period events, respectively. Similarly, the water level (2-year condition) which corresponds to mean annual discharges from the pond was determined to be 69.08 m.

5) The starting water level for backwater flood analysis at Goulds is 69.08 m (GSCD). The starting water level for backwater modelling at Petty Harbour was taken from water levels at St. John's Harbour where the mean high tide level is 0.62 m (GSCD). As it is quite possible that this water level would be present during the course of high river flows, 0.62 m (GSCD) was selected as the backwater starting level.

6) Analysis of tides and storm surges at Ferryland gives the following water levels which were used to delineate flood risk areas at the mouth of the Petty Harbour River.

Petty Harbour River
Design Sea Water Levels at the Mouth

Return Period	High Tide and Surge Level (GSCD)
1:20 year	1.93 m
1:100 year	2.17 m

7) The following provides a summary of the design water level prediction due to tide and storm surges at Ferryland. It also provides wave height estimates for various return periods (or risk levels).

Summary of Study Results

RETURN PERIOD -->	2 Year	5 Year	20 Year	50 Year	100 Year
Wind (km/hr) (Average)	48.35	54.36	62.70	68.06	72.10
Surge (m)	0.74	0.90	1.12	1.26	1.36
Tide (m) GSCD datum	0.81	0.83	0.85	0.86	0.87
Waves (m)	3.20	3.20	3.29	4.83	4.99

The storm surge and high tides can be combined to provide the worst case scenario (as the tides are periodical events that occur quite often). Therefore, the 1:100 year return period water level is expected to be 2.23 m GSCD and the 1:20 year level to be 1.97 GSCD.

8) Large storm surge events result from strong on-shore (blowing from off-shore) winds which also generate high waves at Ferryland. Therefore, wave effects must be added to the above values. Although most high waves will break before reaching the beach areas, potential damage will occur due to wave run-up which adds between 0.5 m and 4.9 m to the storm surge and high tide levels. This will result in flooding in the high risk areas shown in enclosed mapping.

9) The water surface profiles for the 1:20 and 1:100 year return period floods were delineated on 1:2,500 scale mapping which can be obtained from the Department of Environment. This mapping work for all the study areas determined that there are a number of structures within the flood-prone areas. Although the number is now limited, there are many potential locations (particularly in the Goulds area) where it appears that development is being considered or ongoing.

10) The flood levels projected by the backwater analysis and plotted on the flood risk mapping sheets generally were found to be insensitive to modest variations in flood flows, channel roughness and downstream pond or sea levels. However, as many factors can influence flood flows and levels, it is recommended that each municipality provide an offset (e.g., 10 m or 20 m) from the mapped flood lines to account for this variability, ice effects and erosion.

11) The final step in this study calls for preliminary identification of flood damage reduction alternatives which could be employed within the flood hazard areas identified by the mapping.

The principal and recommended alternative to reduce the potential for loss of life and flood-related damages over the long term is to adopt a preventive approach which emphasizes long-range planning in the flood-prone area. Measures such as zoning by-laws, building codes and subdivision regulations can be used to control and direct land use within the flood hazard areas.

12) It is recommended that the flood elevations advanced herein be adopted by the river and shoreline communities so that developable areas which are prone to flooding can be zoned in the near future for special flood risk restrictions and design consideration (e.g., elevation flood proofing on fill, extended and reinforced foundation walls or piles). Consideration should be give to gradual acquisition (by the municipality) of the most flood susceptible lands and some of the most damage-prone buildings during this process.

13) In order to alleviate future flood damage at those structures which are now in the flood-prone area, it is recommended that consideration be given to flood proofing or relocating these buildings. The options and alternatives for the structures in each study area are discussed in detail in the concluding chapter of this report.

2.0 INTRODUCTION

In July 1993, the Province of Newfoundland and the Government of Canada entered into an Agreement Respecting Water Resources Management to help protect and conserve the province's water resources. Included in this new Agreement are provisions for conducting flood risk mapping and studies. This, therefore, provided an opportunity to study and map the 15 areas (made up of over 40 communities) in the province which have experienced flooding problems but were not included in the 1981 Flood Risk Mapping and the 1988 Flood Damage Reduction Programs.

This Flood Risk Mapping Study of the Goulds, Petty Harbour and Ferryland communities is among the series of flood risk mapping studies included under the new Agreement. Flooding occasions in these communities are becoming more frequent as development proceeds in their undefined flood risk areas. Definition of these areas is required and is, therefore, the main goal of this study.

2.1 Study Areas

The Goulds, Petty Harbour, and Ferryland communities are located on the east coast of the Avalon Peninsula, south of St. John's (Figure 2-1).

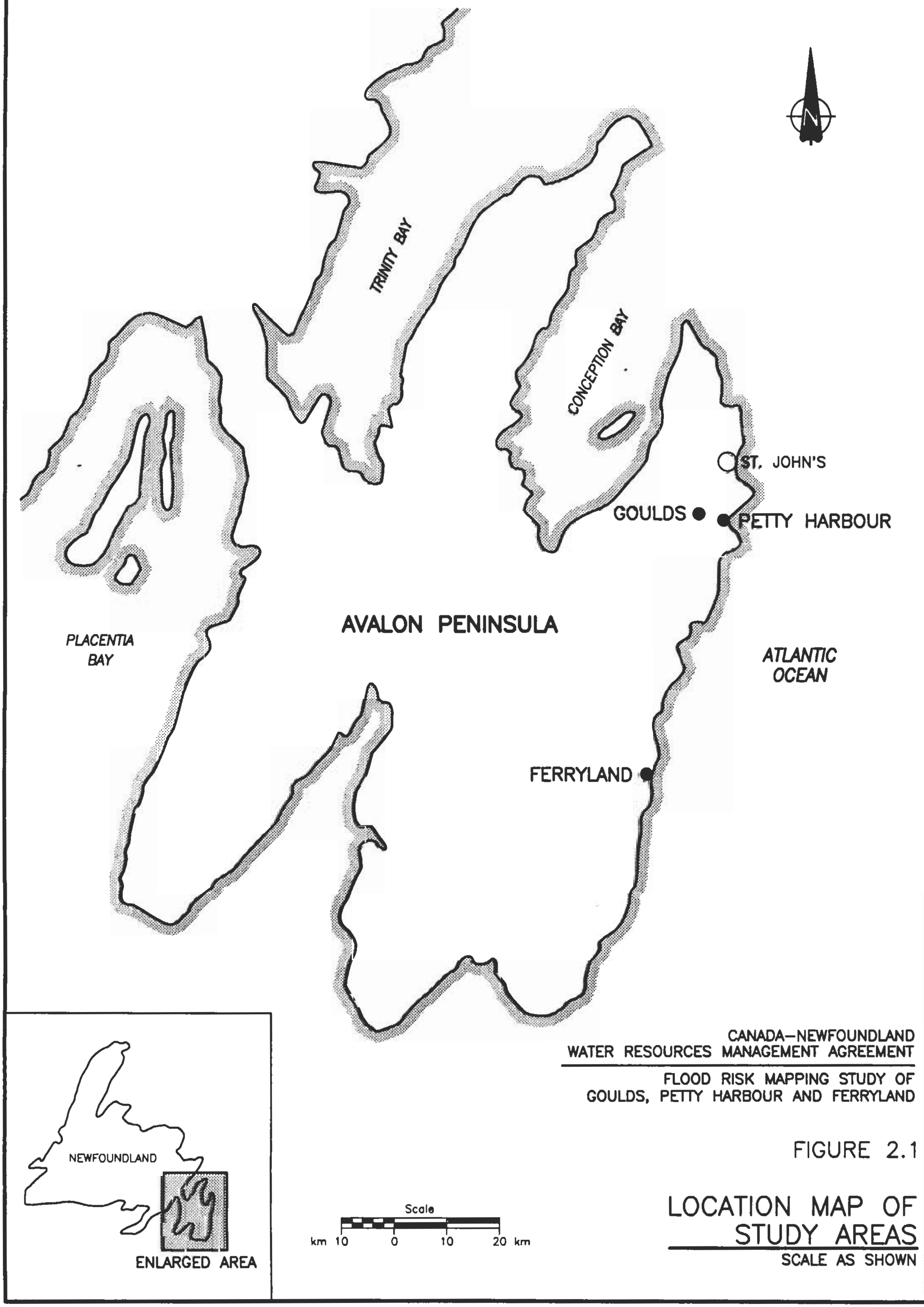
The Goulds study area is fed by five watercourses, as shown in Figure 2-2. These watercourses are: Fourth Pond Brook, Doyle's River (north and west branches), Dirty Bridge Brook, Cochrane Pond Brook, and Raymond Brook. All these watercourses contribute flows to Third Pond which forms the downstream boundary of the study area.

The Petty Harbour study area (Figure 2-3) is traversed by the Petty Harbour River. Flows from the Petty Harbour River are regulated by a hydropower generating facility operated by the Newfoundland Light and Power Company. Both the Goulds and Petty Harbour study areas belong to the same watershed/river system with the latter located on the downstream side.

Figure 2-4 depicts the extent of the Ferryland area which is located some 35 km south of Goulds and Petty Harbour. It is a coastal area and flooding is dictated by oceanographic/coastal processes.

2.2 Study Objectives

The overall thrust of this study is to assess the potential for flooding in the three communities by defining the flood risk areas associated with the 1:20 year and 1:100 year year recurrence interval flood levels. Essentially, the specific objectives are four-fold and are well outlined in the Terms of Reference:

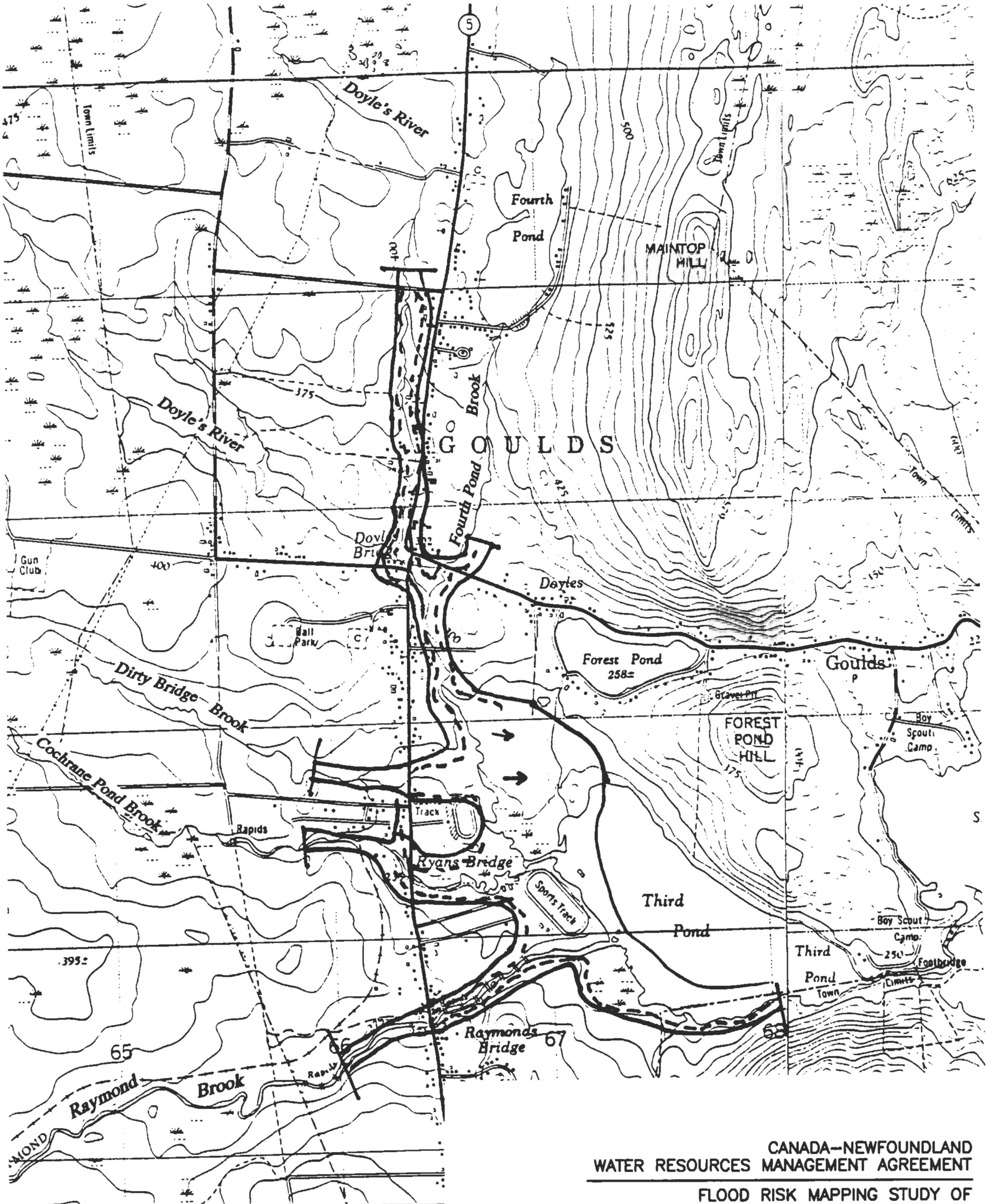


CANADA–NEWFOUNDLAND
WATER RESOURCES MANAGEMENT AGREEMENT

FLOOD RISK MAPPING STUDY OF
GOULDS, PETTY HARBOUR AND FERRYLAND

FIGURE 2.1

LOCATION MAP OF
STUDY AREAS
SCALE AS SHOWN



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FLOOD RISK MAPPING STUDY OF
GOULDS, PETTY HARBOUR AND FERRYLAND

Legend:

Study Limits ———

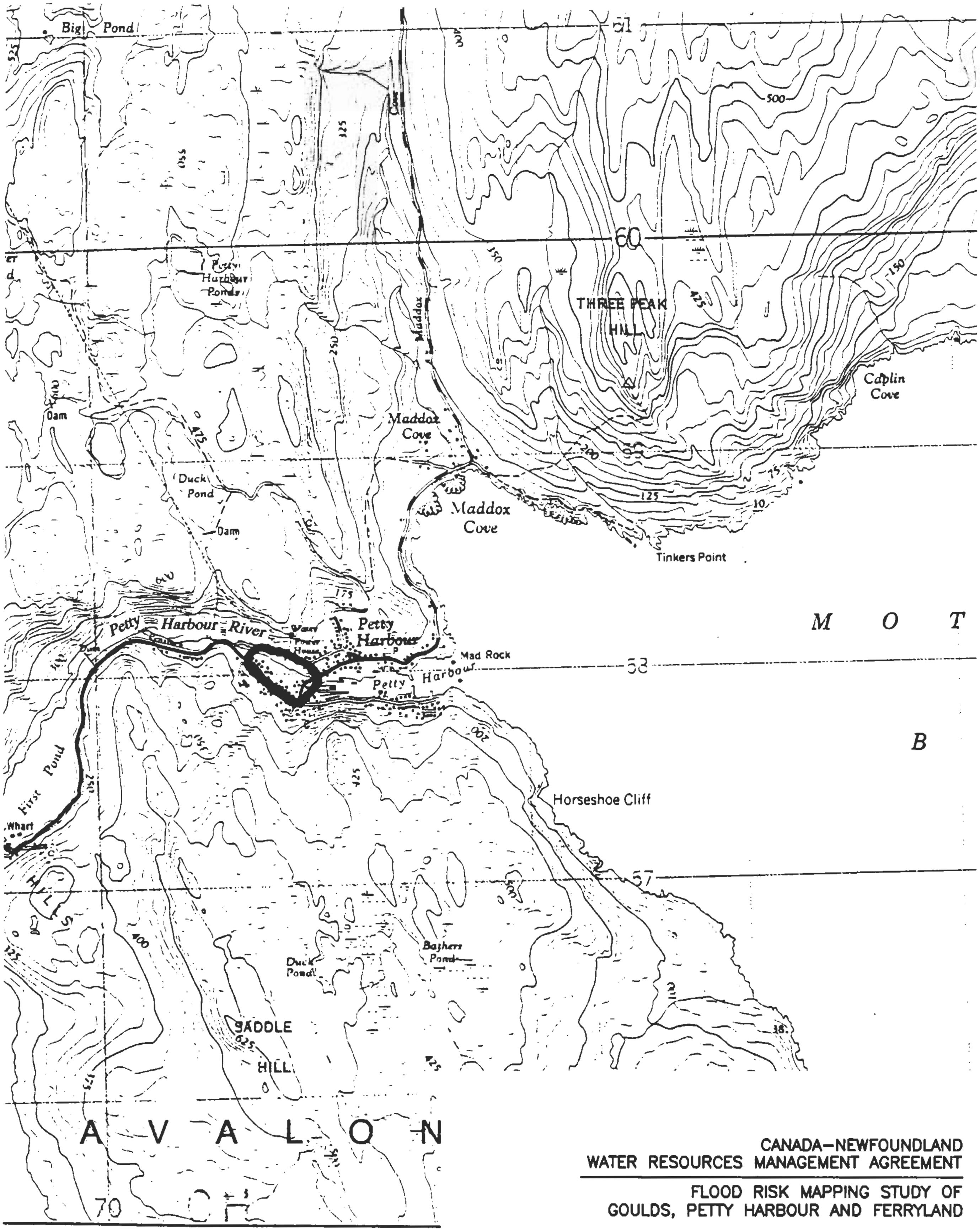
Previous Study Area - - -

FIGURE 2.2

GOULDS STUDY AREA

SCALE 1 : 25000

ILENAME: GOULD4.DWG



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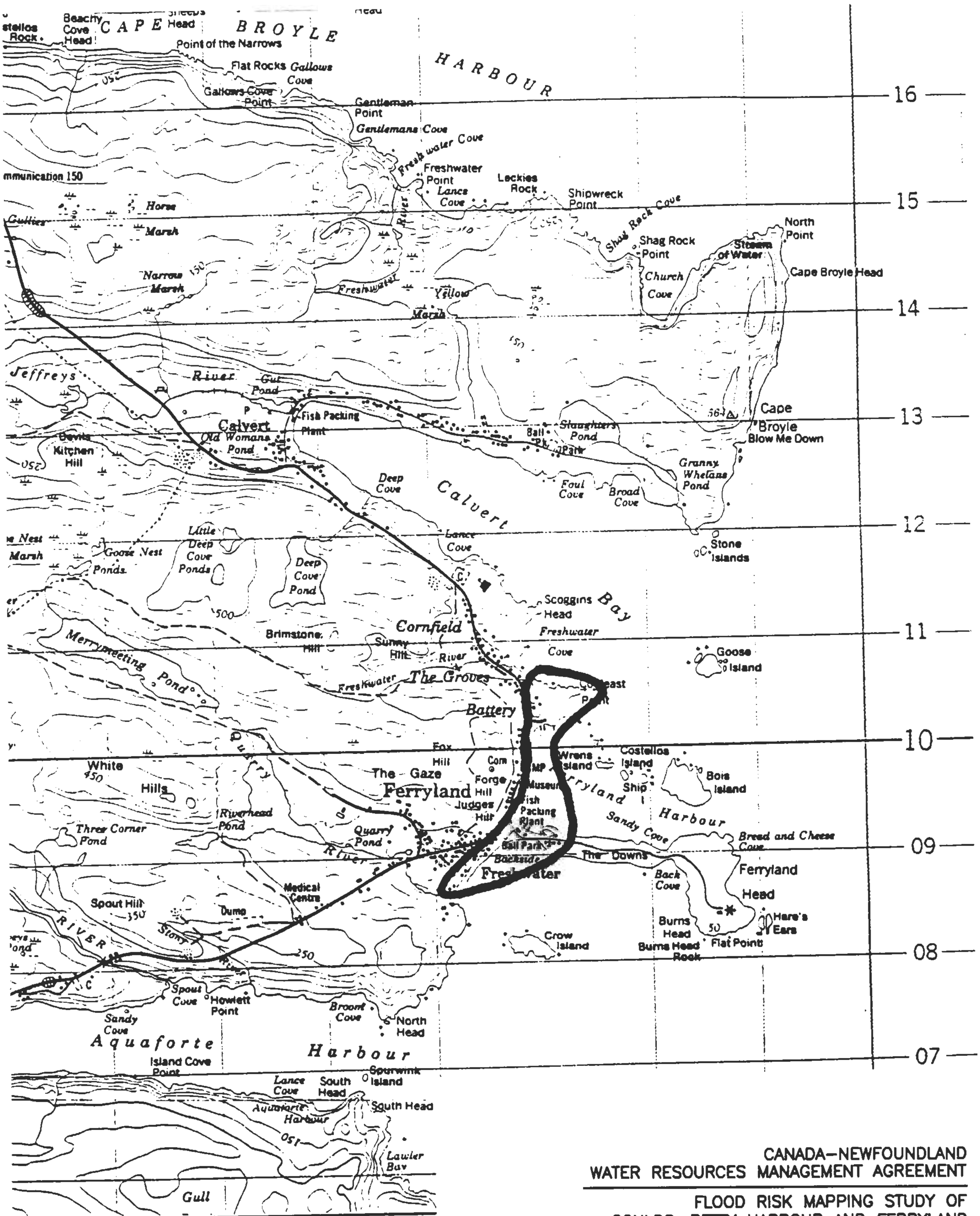
WATER RESOURCES MANAGEMENT AGREEMENT

FLOOD RISK MAPPING STUDY OF
GOULDS, PETTY HARBOUR AND FERRYLAND

Legend:
Study Limits

FIGURE 2.3

PETTY HARBOUR STUDY AREA
SCALE 1 : 25000



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FLOOD RISK MAPPING STUDY OF
GOULDS, PETTY HARBOUR AND FERRYLAND

Legend:
Study Limits

FIGURE 2.4

FERRYLAND STUDY AREA
SCALE 1 : 50000

FILENAME: GOULD6.DWG

- Determine the areal extent of past flooding events for all three study areas.

- Provide estimates of the 1:20 year and 1:100 year recurrence interval flood levels for comparison with the historical flood level (s).

- Plot the 1:20 year and 1:100 year flood lines on the 1:2500 scale provincial topographic maps provided by the Technical Committee.

- Suggest suitable remedial and preventive measures to reduce flood damage potential in the study areas for consideration in future studies.

2.3 Study Approach

The nature of flooding in the three study areas falls into two categories, namely: riverine flooding (for Goulds and Petty Harbour) and shoreline and coastal flooding (for Ferryland). The flooding mechanism for each category is not the same, hence necessitating a different strategy for each one.

Project activities for the Goulds and Petty Harbour study areas were as follows:

- A field program of measurements and observations was conducted to obtain local information about historical flooding in the areas (site interviews with residents), and collect supplementary cross-section and boundary definition data for modelling backwater condition (field survey).

- A streamflow and ice monitoring program has been set up with selected sites and local observers being identified. Monitoring data sheets were developed to assist in recording the observations.

- All pertinent background reports, records and documentation related to hydrology, hydraulics, and flooding were gathered and reviewed.

- A continuous historical database of meteorological conditions (precipitation and temperature) was created. Analysis and synthesis of the precipitation and temperature data from the Petty Harbour, St. John's Airport, and St. John's West CDA were required to create this database.

- A full watershed hydrologic model (QUALHYMO) was constructed to ensure that the hydrology of the study areas, as well as their drainage basins, was accurately assessed. The model appropriately incorporated all of the flooding areas in addition to the inter-connecting ponds and hydraulic features that control the movement and timing of runoff making its way downstream to Petty Harbour.

- The constructed hydrologic model was calibrated and verified to correctly simulate streamflows. Flows monitored by the Water Survey of Canada were employed to check the results at two locations in the river system.

- Flood flow estimates were computed based on frequency analyses of the peak flows generated by the hydrologic model (1961-1992). These 1:20 year and 1:100 year year design flows were found to be higher than those generated by the storms of the same return periods.

- The HEC-2 backwater model was used to obtain flood profiles within the study reach. The model was established from field-surveyed cross-sections and available topographic mapping. Using observed water levels for the period August 1994 to April 1995, the model was calibrated prior to the calculation of the 1:20 year and 1:100 year flood profiles.

- Sensitivity analyses were carried out to determine the impact of variations in flow, Manning's roughness, expansion and contraction coefficients.

- Flood risk areas for the 1:20 year and 1:100 year year flows were delineated on the 1:2500 scale topographic mapping.

- Remedial measures, which are appropriate and realistic for examination in future flood damage reduction studies, were then identified in a qualitative overview of the flood risk mapping.

For the Ferryland study area, the project tasks included the following:

- Long-term tide, water level and meteorological data for St. John's, the nearest community with adequate available data, were gathered. Although St. John's is not in the study area, it has approximately the same climate as Ferryland.

- Thirty years of water level data for St. John's Harbour were obtained from the Canadian Hydrographic Service (CHS) in Ottawa. Correspondingly, thirty years of meteorological data (i.e., hourly wind speed and direction) were obtained for the Atmospheric Environment Service (AES) station at St. John's Airport.

- A database was created and programs were written to process the information. An in-house tidal prediction program was used to generate tide heights from their constituents.

- Storm surges due to winds were determined by taking the difference between the water level data and the tidal data. This was done after cross references were made with CHS data and the generated tide data.

- The corresponding storm surges and winds were then used as input for extreme analysis (using Gumbel Distribution) to determine the required return period values.

- The Bretschneider Procedure (SMB) was used to estimate wave heights and peak periods from meteorological and hydrographic data. The wave hindcast for the study area was only done for the 20 m depth at the head of the peninsula.

- Waves were then carried onshore and run-up was computed using techniques developed by the Coastal Engineering Research Centre, U.S. Army Corps of Engineers.

The following sections of this report discuss these tasks and the results of each activity in greater detail.

3.0 OVERVIEW OF HISTORICAL FLOODING

3.1 Introduction

The history of flooding at Goulds, Petty Harbour and Ferryland has been drawn together from a variety of sources for this study. Principal use was made of:

- a comprehensive review of "*Flooding Events in Newfoundland and Labrador - An Historical Perspective*", prepared by the Water Planning and Management Branch, Environment Canada (Kindervater, 1980). This report gives the causes and effects of floods in the area from 1934 to 1979;

- a listing of similar information for flood events from 1985 to 1994 prepared by the Technical Committee as part of the Terms of Reference of this study;

- newspaper reports giving additional details of flood events from 1985 to 1991. These and earlier reports are archived on microfiche at Memorial University;

- recollections of recent flood events compiled from interviews of local residents during August 1994.

3.2 Historical Flooding - Goulds

It is evident from review of the literature and local reports that flooding in the Goulds area has been frequent and, to some degree, is an annual occurrence. Although flooding is regular, its effects appear limited to periodic roadway overtopping, some basement flooding, and regular flooding of the barns and parking lot at the Avalon Raceway. There has also been loss of life at Goulds when a young child drowned (in April 1964) after she fell into a swollen stream.

Table 3.1 provides a historical overview of potential flooding conditions in the Goulds area (1934 to 1982). The table references "potential" flooding occasions based on historical reports of flooding on the neighbouring Waterford River and in areas adjacent to St. John's (Kindervater, 1980). Table 3.2 provides a listing of confirmed flooding occasions (or high flow conditions) in 1964 and 1982 to 1994. It is surmised that the recent urban development in the Goulds area would have been affected by flooding conditions reported in Table 3.1 had the present urban development been in place during the earlier flood conditions.

TABLE 3.1
Flooding Potential - Goulds

Date	
Oct. 12-14, 1934	heavy rains in the Southside Hills and over the Avalon Peninsula (93.5 mm in 60 hours at Cape Race)
Aug. 2-4, 1941	torrential rains from St. John's area to Burin Peninsula (93.3 mm at Grand Banks)
Jan. 29-30, 1942	heavy rain with snowmelt caused flooding throughout St. John's with St. John's rainfall totalling 127.8 mm over 2 days
Oct. 4, 1942	rainfall and gale caused flooding of ponds and rivers in the suburbs. Torbay Airport reported 100.8 mm rainfall in one day
Dec. 2-3, 1942	suburban St. John's watercourses flooded roads and fields, resulting from 71.4 mm rainfall on 2^{nd} (perhaps 63.5 mm in 3.5 hours)
July 23-24, 1945	heavy flood damage reported in outskirts of St. John's where fields and roads everywhere were inundated (99.8 mm rain in 2 days)
July 27-29, 1946	rainstorm (~14 hr duration) over entire Avalon Peninsula caused numerous washouts/ general flooding (129 mm at Torbay Airport)
Dec. 1-2, 1946	rainstorm over entire Avalon Peninsula caused highest flows since Jan. 1942 (yet only 68.5 mm rain in 2 days)
Sept. 14-15, 1948	torrential rain (117.1 mm in 2 days) which followed a tropical storm two weeks earlier
Apr. 10-12, 1951	3-day rainfall of 170 mm (Torbay) raised local lakes to remarkably high elevations
Feb. 12-13, 1955	flooding reported at Mobile and on Waterford and Rennie's River, St. John's, caused by rain (at this time) which followed 2 to 3 other heavy rainfalls in January
Nov. 10-11, 1959	intense rainfall (70.1 mm over 2 days) followed 83.6 mm on 1-2 November and caused overbank flows and flooding in low lying areas on the Avalon Peninsula
Jan. 1-3, 1963	rain, melting snow and ice jams caused significant flooding in St. John's (e.g., Waterford River) and at many locations along the coastline from the Avalon Peninsula to Notre Dame Bay
Dec. 20-21, 1966	rainfall (102.8 mm) over 2 days caused widespread high flows, road flooding and washouts - likely including the Goulds area
Feb. 27-Mar. 2, 1970	4-day rainfall (114 mm) resulted in general flooding in the St. John's area
Jan. 31-Feb. 1, 1971	melting snow and rain (48.8 mm) caused flooding at many locations on the Avalon Peninsula (including St. John's)
Dec. 27-29, 1977	snowmelt and rain caused brooks and ponds to rise "far above normal" across the province
Jan. 28-30, 1979	91 mm rainfall over 6 days caused local flooding and high water levels on adjacent rivers

TABLE 3.1 (continued)
Historical Flooding Potential - Goulds

Date	
Oct. 10-11, 1981	over 121 mm rainfall in 2.5 days resulted in serious flooding in the Southside Hills area
Nov. 26, 1981	76 mm rainfall in one day caused overbank flooding in a number of areas in St. John's (e.g., Southside Roads)
Oct. 3-5, 1982	over 100 m rainfall reported on one day during severe storm and high winds, but little or no flooding was reported

references: Kindervater (1980)
Fenco Newfoundland (1988)

TABLE 3.2
Historical Flooding - Goulds

Date	
Apr. 16-17, 1964	rainfall increased streamflows at Goulds where a 2-year old child fell into a stream and drowned
May 24-25, 1985	84.8 mm rainfall in 33 hours caused flooding in St. John's and localized flooding in a number of areas around the Town of Goulds. The parking lot and entrance to the Avalon Raceway were blocked by high water.
Winter 1985/86	ice jams formed at the outlets of the rivers into Third Pond following a rainstorm. A house near Raymond's brook was flooded and the water flowed across the Avalon Raceway at Goulds.
Apr. 11, 1986	rainfall on frozen soil led to flooding in many areas in and around St. John's. Rainfall amounts from 70 to 109.6 mm (over a 22-hr. period) were recorded at climate stations in the St. John's area. Many areas in the Goulds reported flooding. A culvert crossing at Bishops Lane caused water to back up and flood a property. Other culvert blockages resulted in further flooding. Again, the Avalon Raceway was hit by flooding and it was reported that 30 horses were evacuated.
Feb. 26-27, 1987	an ice jam on Ryan's River behind the Avalon Raceway was removed by explosives after causing some basement flooding and ice pieces to jam up against some nearby houses.
Mar. 17-20, 1987	Record snowfalls over the winter left the river channels filled with snow and ice. During this melt period, there was also over 74 mm of rainfall and runoff quickly overtopped the riverbanks in the Goulds area. Again, the Avalon Raceway was flooded but there were only a few flooded basements
Feb. 26-Mar. 2, 1988	blasting was used on the Ryan's River behind the Avalon Raceway in Goulds to break up an ice jam which caused some basement flooding in nearby homes and precautionary evacuation of horses from the raceway. Problems were attributed to obstructions in the river, including remnants of a partially removed dam at the lower end of the Third Pond.
Jan. 9, 1989	Raymond's River, which flows into Third Pond, had overflowed its banks for a week (because of ice build-up) and had flooded one resident's septic tank. The Emergency Measures Division ice demolition team was called in to alleviate potential problems.
Mar. 4, 1990	melting snow and rainfall caused flooding of one barn at the Avalon Raceway and several feet of water across the parking lot
Feb. 15-16, 1991	Snowmelt combined with 40 mm of rainfall on frozen ground caused flooding in many areas of the Goulds. Doyle's River near a service station overflowed its banks and flooded the property. Many culverts and bridges were flowing at or near capacity. The Avalon Raceway was again hit by flooding from the Ryan's River and four barns were flooded.
Spring 1993	roadway overtopping at Doyles River reported to cause flow over the gasoline station each year

TABLE 3.2 (continued)
Historical Flooding - Goulds

Date	
Spring 1994	roadway flooding and a bridge was overtopped (or nearly overtopped) as a result of snow beneath it or debris and rafting ice. Water was 2.5 ft. deep at entrance to main barn at the raceway. ** Raceway staff indicate that the predominant source of flooding is the southern Raymond's Brook and not the more northern Ryan's Brook (which affects the back barn)

references:
- newspaper reports (i.e., Evening Telegram)
- local residents
- study Terms of Reference

It is possible to draw several preliminary conclusions from review of Table 3.1 and Table 3.2. Generally, flooding problems may be anticipated to result from synoptic rainfall events with durations of about two days. A rainfall total of about 70 mm appears to be the threshold value which will initiate flooding in adjacent areas. In addition, regular flooding can be anticipated because of natural accumulation of ice in the local rivers and culvert blockages by debris, snow and ice. As noted in a 1987 newspaper report, "flooding has been going on for many years but there was never any real need for concern before the Avalon Raceway was constructed".

It is also interesting to note that early references to flooding on the "outskirts of St. John's" have more recently been replaced by specific references to Goulds and other industrial and residential areas which have grown in these areas. In view of this urban growth, the frequency of previous flooding, and the possibility of damaging floods in the future, it is appropriate that the Technical Committee has initiated the flood risk mapping component of this study.

3.3 Historical Flooding - Petty Harbour

It is noted in the Terms of Reference for this study that, "the most severe flooding to occur in Petty Harbour occurred in April 1986. Prior to 1986 there were reports of high flows but no severe flooding. The flooding occurs along the lower sections of the river upstream of the bridge near the outlet into the harbour. The river channel is downstream of the forebay dam for the hydroelectric plant at Petty Harbour".

Interviews with local residents (August 1994) and the work by Kindervater (1980) provide supporting and additional information given below.

TABLE 3.3
Historical Flooding - Petty Harbour

Date	
~ 1940	local residents report that the worst riverine flood in recent memory is from an event which occurred prior to 1945 (estimated ~ 1940) when there was 3 to 4 feet of flow over the old upstream dam
Oct. 22, 1955	high winds, waves and water levels generated by a "nameless" hurricane damaged stages at Petty Harbour (Kindervater, 1980)
Jan. 20-21, 1977	high winds and heavy seas damaged wharves at St. John's and Placentia (and potentially at Petty Harbour) (Kindervater, 1980)
Apr. 11, 1986	this flooding event followed a rainstorm in which 71 mm of rain fell on the area in 11 hours. Snowmelt combined with frozen ground increased the severity of the flooding. The reservoir at Second Pond upstream of the town was also full at the time. Several residences, a council building and a senior citizens home were flooded. The position of grading material around the council building was considered the cause of this flooding rather than the river. Flood levels reached within several inches of the underside of the railing at the culvert crossing on the main road or 3 feet below the obvert of the old bridge.

Overall, there is confirmed riverine flooding from rainfall or snowmelt-with-rainfall events in the early 1940s and in 1986 and flood damage from heavy seas in 1955 and possibly in 1977. There is also information from local residents that there is minimal flood damage observed each year from small channels which become filled with snow and ice and cause some basement flooding.

In that this picturesque locale is undoubtedly a location where new residences may be proposed along the river, it is timely that the Province has launched this study to identify the limits of the riverine flood risk.

3.4 Historical Flooding - Ferryland

Ferryland has a rich history, dating back to its founding in the early 1600s. It is likely that the intervening ~ 400 years have been punctuated by flood damage from hurricanes and other storms which may be described in archival records. Other storms may not have been recorded or widely publicized because these events were considered as a natural result of living with the sea. Review of newspaper records and Kindervater (1980) identifies the following instances of coastal flooding (Table 3.4).

TABLE 3.4
Recent Historical Flooding - Ferryland

Date	
Oct. 22, 1955	high winds, waves and water levels generated by a "nameless" hurricane flooded lowlands at Ferryland (Kindervater, 1980)
Jan. 20-21, 1977	high winds and heavy seas damaged wharves in St. John's and Placentia - and possibly Ferryland as well (Kindervater, 1980)
Jan. 5, 1989	strong winds brought exceptionally high seas forcing 4 families to leave their homes after the town's breakwater was overtopped and broken. Highway 10 was covered by sand accumulations, and high winds and water moved 2 houses, flooded others and shifted several, flooded the town's ball field and damaged a shed at the fish plant. Report as worst storm the residents can remember.

In view of ongoing interest in the historical background of Ferryland and the potential for future residential home construction near the shore, it is again appropriate that the Technical Committee has initiated the flood risk mapping component of this study to minimize the potential for damage from future flooding.

4.0 HYDROLOGIC MODELLING - GOULDS AND PETTY HARBOUR STUDY AREAS

The QUALHYMO hydrologic computer model was employed to simulate and recreate runoff volumes and hydrographs throughout the study area. QUALHYMO is a watershed scale continuous simulation model which incorporates historical meteorologic data with physical watershed parameters (such as soil type, land use, etc.), and more importantly, accounts for the accumulation and ablation of snow pack depths on the ground by applying a degree-day snowmelt algorithm. This latter feature is of significant importance to this investigation due to the spring snowmelt period and periods of winter rain being the predominant factors attributed to flooding in the region.

The model is capable of calibration to observed flow rates and runoff volumes resulting from both snowmelt and precipitation influences; however, for this investigation, quality observed streamflow data is limited to the Water Survey of Canada (WSC) station on Raymond Brook (4 years) and the outlet of the First Pond (50 years of data provided by Newfoundland Light and Power Co. Ltd./WSC). Information pertaining to streamflows at the reported flooding sites in the Goulds area is not directly available beyond limited estimates of high water levels only. Therefore, to ensure that the hydrology of these intervening flooding areas is accurately assessed, a full watershed hydrologic model was developed. This model appropriately incorporates all of the flooding areas in addition to the inter-connecting ponds and hydraulic features that control the movement and timing of runoff making its way downstream to Petty Harbour. By applying this technique, the calibration of the model was carried out by manipulating watershed and meteorologic parameters to reflect observed historical snow pack characteristics and by comparing the simulated basin response to the WSC gauging stations.

As noted above, using the calibrated model, historical surface runoff volumes and peak flow rates resulting from meteorologic and climatic conditions were estimated through successive annual simulations (i.e., continuous simulation). Subsequent statistical analysis of these results (simulated runoff time series) provided an assessment of annual peak runoff events. This further enabled the estimation of the probability of occurrence of high or extreme runoff events (i.e., 100 year event, etc.) throughout the basin.

The key to accurately simulating the basin's runoff history started with the development of a continuous historical database of meteorological conditions (precipitation and temperature) in the vicinity of the study area. Following a detailed examination of the meteorologic records from the stations summarized in Table 4.1, a representative continuous database was developed for the study area. Unfortunately for all meteorological stations, there are some periods where data is missing from the records due to limited operating periods (seasonal), malfunctioning equipment, etc.

TABLE 4.1
Meteorological Stations Examined in this Study

Station	Record Period	Meteorological Elements
Petty Harbour	1955-1993	Precipitation & Temperature
St. John's	1950-1956	Precipitation & Temperature
St. John's Airport	1949-1993	Precipitation & Temperature
St. John's West CDA	1950-1993	Precipitation & Temperature

Petty Harbour Station is the closest meteorologic station to the study area and is considered most representative of the basin's historical climatic conditions. Consequently, this station was used as the principal source of data for this investigation.

The data from St. John's Airport and St. John's West CDA were enlisted to complete the historical continuous database, as required. Records from these stations were first adjusted by applying a factor to account for the regional differences in total annual/monthly precipitation trends. These factors were determined from the long-term climatic normal precipitation amounts as noted in Table 4.2. The records were merged to form a continuous meteorologic history for the period 1963 to 1992 (30 years).

4.1 Basin Discretization

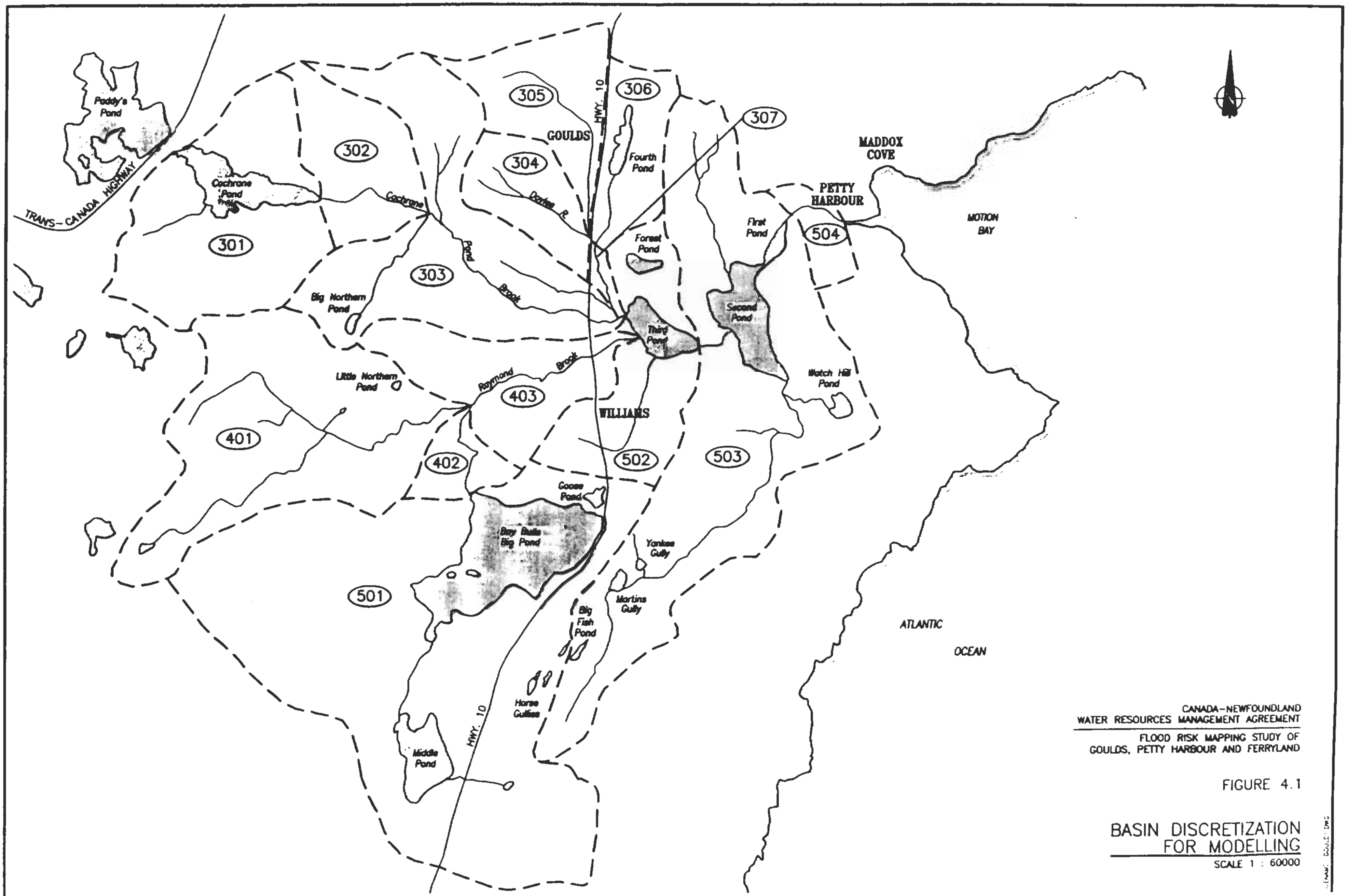
The entire watershed was sub-delineated into 14 subcatchments based on a review of the watershed physical characteristics, surficial geology, land use, previous watershed modelling studies, and the need to provide detailed flow points for input to the hydraulic modelling (Section 5.0). Figure 4-1 shows the geographical distribution of the discretized subcatchments.

4.2 Land Use and Surficial Geology

The geology of the study area is characterized, in general, by mainly red sandstone, red conglomerate, and greenish gray sandstone of the Cabot Group. The surficial soils consist of mainly very firm, and stony loam sand tills and some partly decomposed mass peat (soils of the Avalon Peninsula, Newfoundland). These soils are considered to be moderately to very poorly drained. A relatively higher surface runoff would be excepted from the study area, especially from those areas where the subsoil is impermeable or bedrock is close to the surface. For initial estimates of the hydrologic soil runoff curve number (CN), the surficial soils of the study area were assumed to be

TABLE 4.2
Climate Normals at Selected Meteorological Stations

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year
Petty Harbour Station													
Rainfall (mm)	83.9	74.4	82.8	92.4	93.0	85.9	74.2	108.7	115.7	146.5	134.4	107.1	1189.9
Snowfall (mm)	55.1	45.7	35.6	17.6	3.7	0.3	0.0	0.0	0.0	0.7	9.9	34.8	203.4
Precipitation (mm)	138.4	120.1	118.4	110.0	98.8	86.2	74.2	108.7	115.7	147.2	144.3	141.9	1403.8
St. John's West CDA													
Daily Mean (°C)	-4.0	-4.6	-2.0	1.8	6.4	11.3	15.8	15.6	11.8	7.3	3.3	-1.4	5.1
Rainfall (mm)	90.9	78.8	88.6	91.7	98.6	92.3	77.8	113.8	117.0	149.2	133.4	107.5	1239.6
Snowfall (cm)	85.3	73.8	53.8	31.8	8.8	1.2	0.0	0.0	0.0	2.3	18.7	53.3	329.0
Precipitation (mm)	179.4	154.9	146.3	124.5	107.0	93.5	77.8	113.8	117.0	149.0	152.8	163.5	1579.5
St. John's Airport													
Daily Mean (°C)	-4.3	-5.0	-2.5	1.3	5.8	10.9	15.4	15.3	11.6	7.0	3.1	-1.7	4.7
Rainfall (mm)	69.3	69.2	73.6	79.6	91.4	95.3	77.9	121.8	125.0	147.4	121.6	91.0	1163.1
Snowfall (cm)	83.0	68.8	54.0	26.8	7.8	1.4	0.0	0.0	0.0T	4.0	21.5	54.7	322.1
Precipitation (mm)	147.8	133.6	126.7	110.4	100.9	96.9	77.9	121.8	125.0	151.7	144.7	144.2	1481.7



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WATER RESOURCES MANAGEMENT AGREEMENT

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GOULDS, PETTY HARBOUR AND FERRYLAND

FIGURE 4.1

BASIN DISCRETIZATION
FOR MODELLING

SCALE 1 : 60000

hydrologic soil group C. The land uses of the watershed were derived using the 1987 aerial photographs and field observations by Fenco MacLaren staff (August to October, 1994). Table 4.3 summarized the weighted CN numbers and drainage areas for the subcatchments. The QUALHYMO model schematic is shown on Figure 4-2.

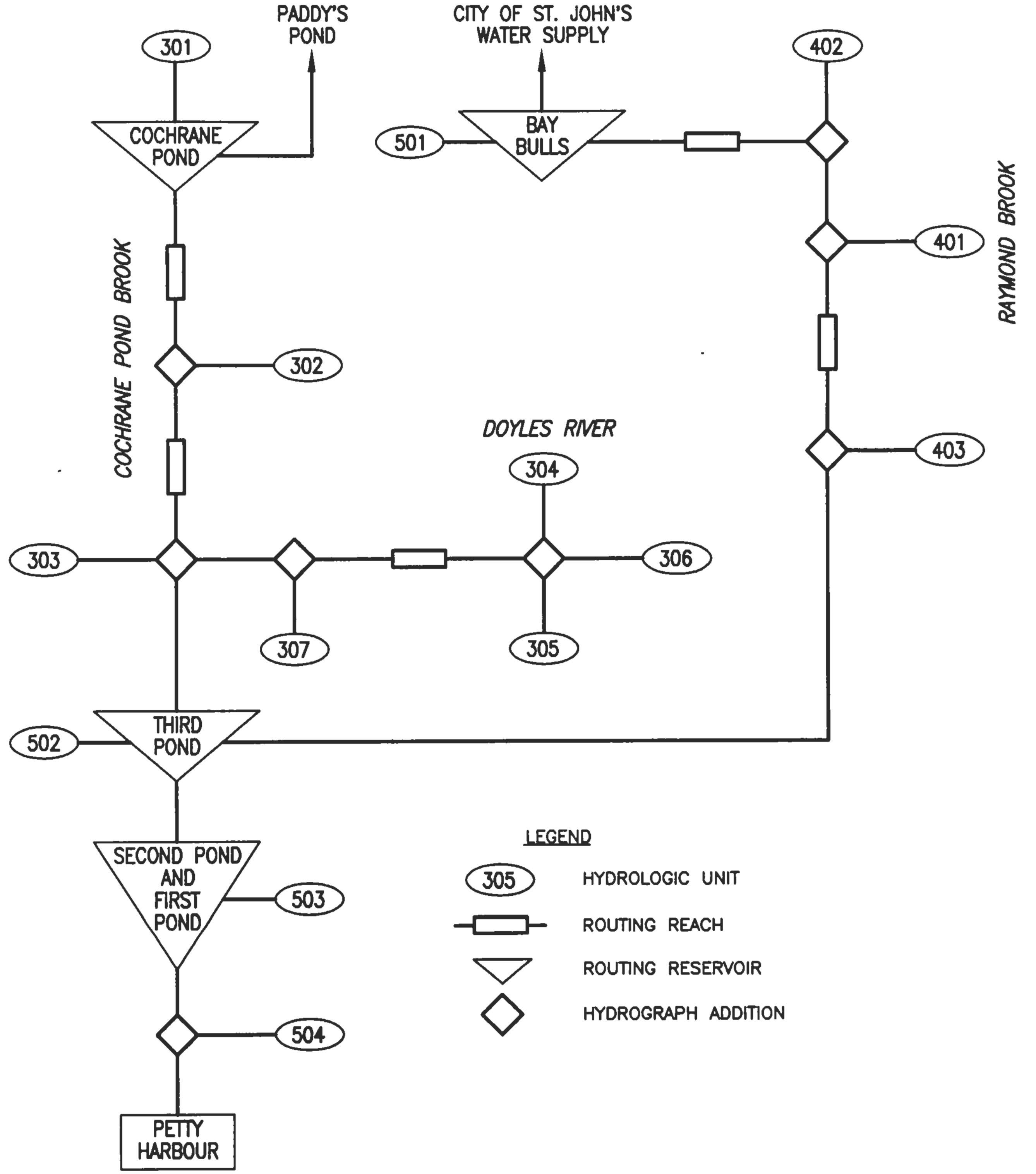
4.3 Hydrologic Model Calibration

The initial efforts of the calibration exercise started with a comparison of the model snow pack predictions with historical snowpack (i.e., accumulated snow on the ground). Depth of snow on the ground observations are available from Petty Harbour Station for the period 1963 to 1992, and from snow cover data at Cochrane Pond for the period 1963 to 1983 (Table 4.4). These were used to calibrate the timing and duration of the spring melt events. The snowpack available for melt in April is usually depleted by the middle or end of April. As noted in Table 4.4, the water equivalent of the snowpack reaches as much as 350 mm between late February and early March (prior to spring melt periods). From our simulations, a good correlation was noted (Figure 4-3) between observed snowpack and QUALHYMO simulated snowpack depths.

As mentioned early in the report, there are two Water Survey Canada (WSC) streamflow gauges operating within the study area. They are the Petty Harbour at Second Pond (Station No. 02ZM001) and Raymond Brook at outlet of Bay Bulls Big Pond (Station No. 02ZM022). The mean monthly streamflow distributions at these stations are summarized in Table 4.5. These gauged watersheds are jointly regulated by the Newfoundland Light and Power Co. Limited and the City of St. John's for hydro power and water supply purposes. Due to the regulation of flows exiting the Pond and withdrawal from the pond for water supply to St. John's, it is very difficult to compare the simulated and recorded runoffs on an individual monthly basis or even a peak event basis. However, the recorded annual runoff volumes can at least be used as an indicator to check and calibrate the noted simulated runoff volumes on an annual basis. For this investigation, recorded annual runoff volumes from Petty Harbour River at Second Pond Station were used to compare with the simulated runoff volumes due to a longer period of record (1963 to 1992, see Table 4.5). During this stage, the hydrologic model incorporated the stage, storage, outflow relationship at Bay Bulls Big Pond in the long-term analysis of downstream flows to Raymond Brook. Evaporative losses from the surface of the pond were also included, but water supply extractions to St. John's were not included. These latter losses from the pond have been variable and their exclusion results in a conservatively high runoff potential along Raymond Brook - the only watercourse affected by these water supply withdrawals. As shown later (Table 4.10), the return period flood flow estimates are not significantly affected.

TABLE 4.3
Summary of Weighted CN Number and Drainage Areas for the Subcatchments

Subcatchment No.	Drainage Area (ha)	Weighted CN (AMCII) (--)
301	1149	78
302	491	73
303	1339	74
304	348	81
305	494	80
306	345	74
307	40	92
401	1545	76
402	173	73
403	626	79
501	3900	82
502	745	79
503	2361	74
504	146	76



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WATER RESOURCES MANAGEMENT AGREEMENT

FLOOD RISK MAPPING STUDY OF
GOULDS, PETTY HARBOUR AND FERRYLAND

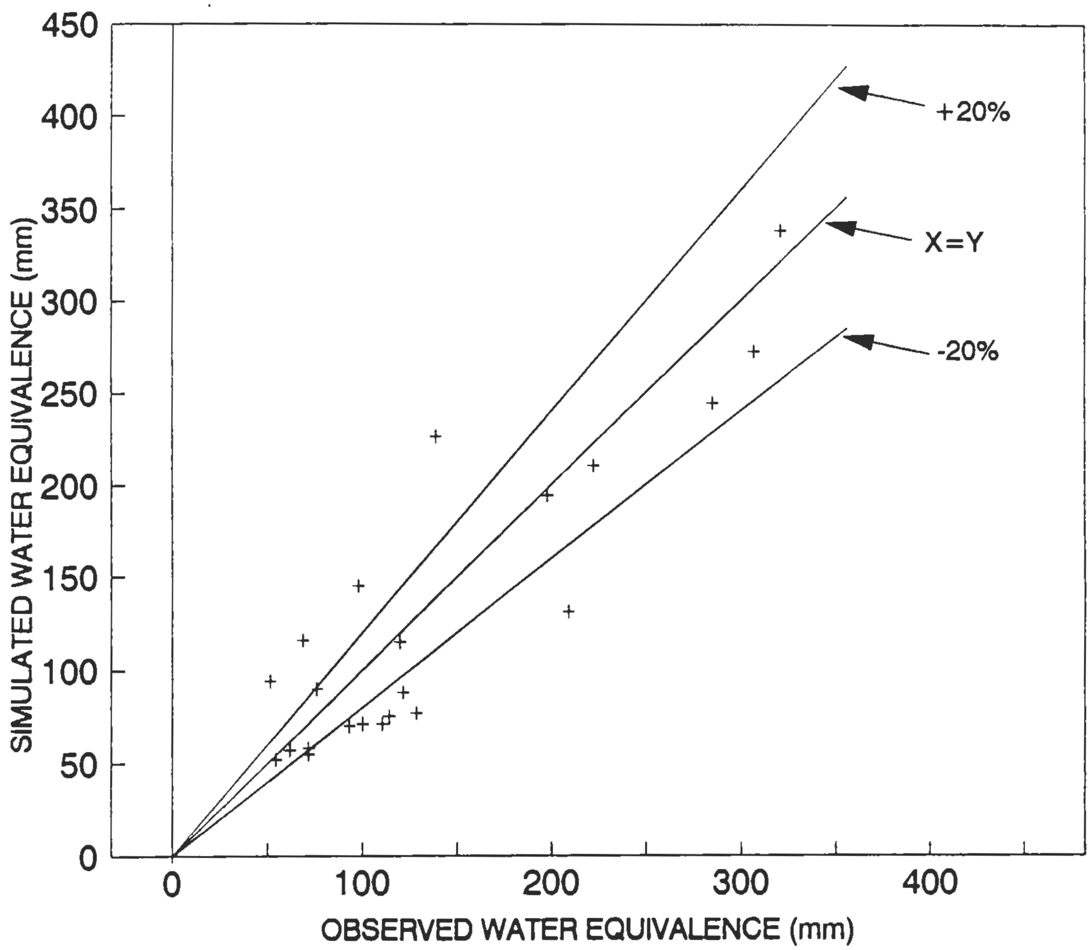
FIGURE 4.2

BASIN SCHEMATIC

TABLE 4.4
Observed Maximum Snow Water Equivalent at Petty Harbour Station and Cochrane Pond Station

Year	Max. Water Equivalent at Petty Harbour Station (mm)	Max. Water Equivalent at Cochrane Pond Station (mm)
61-62	106	--
62-63	98	38
63-64	356	295
64-65	72	145
65-66	139	302
66-67	188	107
67-68	71	51
68-69	51	58
69-70	110	71
70-71	177	--
71-72	154	127
72-73	68	--
73-74	122	120
74-75	222	243
75-76	109	--
76-77	135	81
77-78	259	--
78-79	54	49
79-80	198	--
80-81	76	--
81-82	285	325
82-83	62	47
83-84	128	--
84-85	209	--
85-86	93	--
86-87	321	--
87-88	114	--
88-89	120	--
89-90	71	--
90-91	100	--
91-92	307	--

Note: -- Not available
Petty Harbour Station (AES Station 8402925)
Cochrane Pond Station (Environment Canada, Snow Course Station)



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WATER RESOURCES MANAGEMENT AGREEMENT

FLOOD RISK MAPPING STUDY OF
GOULDS, PETTY HARBOUR AND FERRYLAND

FIGURE 4.3

COMPARISON BETWEEN SIMULATED AND OBSERVED MAXIMUM WATER EQUIVALENCE

TABLE 4.5
Recorded Mean Monthly Streamflows at Petty Harbour at Second Pond and Raymond Brook at Outlet of Bay Bulls Big Pond

Month	Petty Harbour at Second Pond [1] (m^3/s)	Raymond Brook at Outlet of Bay Bulls Big Pond [2] (m^3/s)
JAN	6.41	0.917
FEB	6.83	0.903
MAR	7.79	0.832
APR	9.29	0.780
MAY	7.11	1.40
JUN	4.14	1.11
JUL	2.56	0.752
AUG	2.47	0.426
SEP	3.13	0.160
OCT	5.76	0.494
NOV	6.72	0.919
DEC	8.84	1.24
MEAN	**5.73**	**0.827**

(1) Station No. 02ZM001 (1963-1992)
(2) Station No. 02ZM022 (1988-1992)

Results from this stage of calibration are summarized in Table 4.6 and Figure 4-4. As shown by the tabulations, the simulated annual runoff volumes compare quite well with the recorded annual volumes. In general, the simulated runoff volumes are within 10% of the recorded volumes. Notable exceptions include 1965, 1966, 1967, 1969, 1983 and 1987 which are within 20% of the recorded volumes.

The next stage of calibration included a comparison of the timing of the simulated annual maximum peak/flood events to the observed significant historical flooding events. These historical flooding events were identified in the 1991 DELCAN report (2) and are summarized in the following Table and Technical Appendix B.

TABLE 4-7
Comparison Between Timing of the Simulated Annual Maximum and Observed Historical Flood Events

Year	Observed	Simulated
1985	May 24 to 25	May 25
1991	Feb 15 to 16	Feb 17

As shown in Table 4.7, the simulated timing of the maximum flood events compares well with the above two reported historical flood events. Based on the results of the above discussed calibration exercises, it was concluded that the model could be used to determine runoff histories at selected sites throughout the watershed.

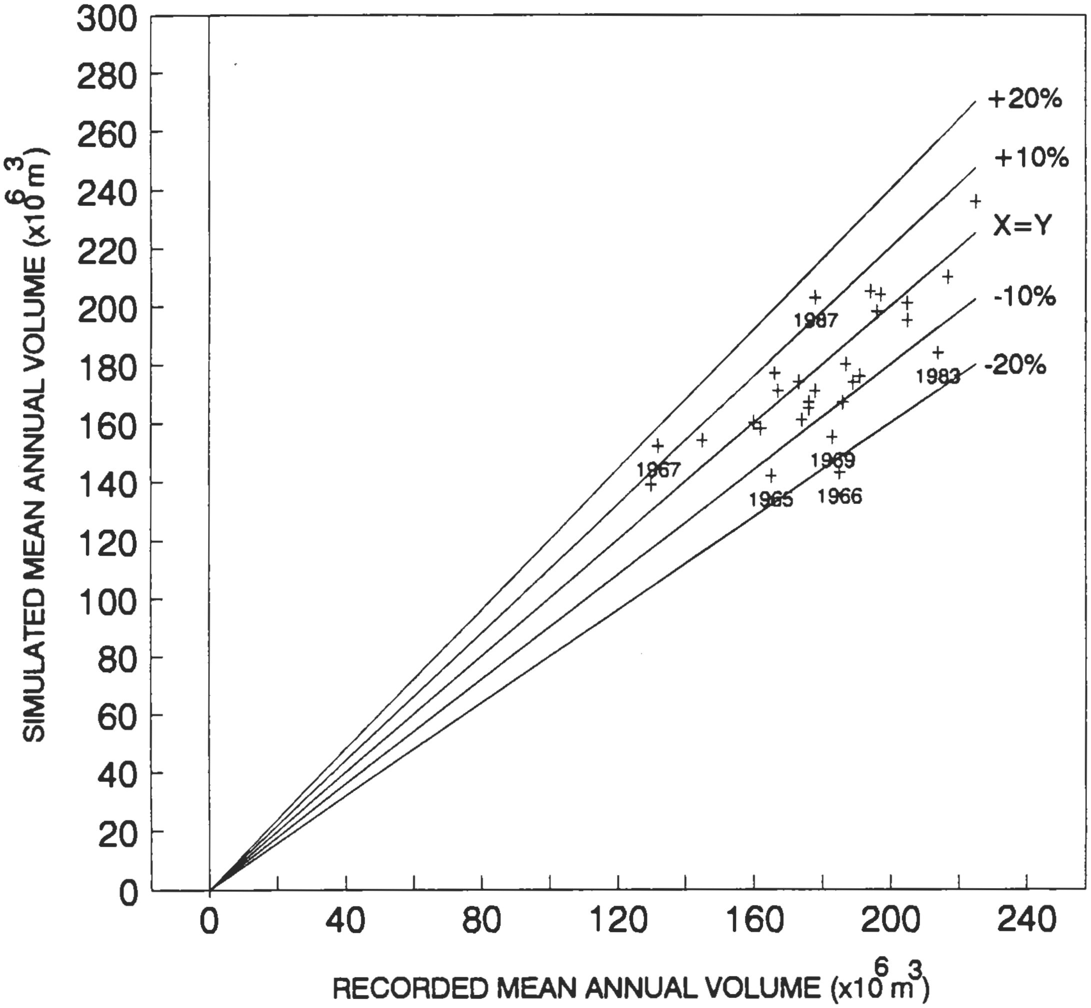
The calibrated model was then used to simulate the annual runoff events for the period between 1963 and 1992. The annual maximum flood peak for each year was selected by examining the annual simulations. These values/peaks are listed in Table 4.8 at selected locations/within the study basin area. Frequency analyses were then undertaken to determine the probability of occurrence of high or extreme runoff conditions. In this investigation, the 1:20 year and 1:100 year peak discharges are required for use in the subsequent hydraulic analyses.

As noted above, it was difficult to undertake a rigorous calibration and verification of the hydrologic model without the existence of more detailed observations of flow rates at sites throughout the basin combined with the regulation effects of Second Pond at Petty Harbour.

TABLE 4.6
Comparison Between Simulated and Recorded Mean Annual Runoff at Petty Harbour at Second Pond

Year	Recorded Mean Annual Runoff Volumes at Petty Harbour Station (million m^3)*	Model Simulated Mean Annual Runoff Volumes (million m^3)
1963	205	195
1964	214	184
1965	145	154
1966	166	177
1967	160	160
1968	173	174
1969	205	201
1970	217	210
1971	187	180
1972	185	143
1973	176	167
1974	191	176
1975	183	155
1976	189	174
1977	162	158
1978	178	171
1979	167	171
1980	225	236
1981	194	205
1982	196	198
1983	197	204
1984	178	203
1985	132	152
1986	186	167
1987	176	165
1988	----	----
1989	130	139
1990	174	161
1991	165	142
1992	184	----

Note: ---- Not available
* Includes effect of water supply extraction from Bay Bulls Big Bond (increasingly significant since 1987)



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WATER RESOURCES MANAGEMENT AGREEMENT

FLOOD RISK MAPPING STUDY OF
GOULDS, PETTY HARBOUR AND FERRYLAND

FIGURE 4.4

COMPARISON BETWEEN RECORDED AND SIMULATED
MEAN ANNUAL RUNOFF VOLUMES

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TABLE 4.8
Summary of Simulated Annual Peak Flows at Selected Locations

Year	Cochrane Brook Outlet (m^3/s)	Doyles River Outlet (m^3/s)	Raymond Brook Outlet (m^3/s)	Third Pond Inlet (m^3/s)
1961	23	17	23	70
1962	32	24	29	96
1963	62	49	51	179
1964	48	38	43	140
1965	33	24	30	98
1966	46	36	39	133
1967	48	36	44	141
1968	57	46	49	166
1969	28	20	29	85
1970	56	44	46	161
1971	41	33	35	121
1972	73	61	56	210
1973	69	61	48	196
1974	59	62	43	171
1975	51	37	47	148
1976	33	26	31	97
1977	63	56	45	181
1978	53	44	47	154
1979	28	21	27	83
1980	51	40	47	153
1981	77	62	62	223
1982	47	40	41	140
1983	34	27	31	100
1984	91	72	76	262
1985	46	36	42	136
1986	50	35	49	151
1987	58	48	49	169
1988	48	37	41	140
1989	70	54	61	203
1990	36	27	34	109
1991	67	24	52	194
1992	34	24	31	100

However, while this investigation was ongoing, a severe winter storm occurred in the study area. This mid-winter storm brought up to 50 mm of rainfall over a two-day period (January 7 to January 8, 1995). In the weeks preceding the storm, meteorological observations were described as seasonably mild with temperatures reaching above 0°C during the daytime, along with periods of both rain and snow being noted up to a week before the storm (Table 4.9). Snowpack observations identified a growing pack depth leading up to the storm, with a maximum of 36 cm of snow being observed at St. John's West CDA on January 6, 1995.

During the heavy rainfall on January 7 and January 8, snowpack depths within the study area deteriorated rapidly, as shown in Table 4.9, and near-flooding levels were reported throughout the region.

Streamflow data was also available from the Petty Harbour generating station. Examination of these flows shows that the rainfall event yielded significant runoff volumes that were ultimately spilled downstream from Second Pond. While the routing effects of the upstream ponds and control structures tended to mask the true response of the upstream watershed areas, these results show that at least 14.5 x 10^6 m^3 of water was released from the pond over a seven-day period (during and following the rainfall).

Following the retrieval of the January 1995 meteorological and streamflow data, efforts were directed to the verification of the hydrology model. Unfortunately, the data were limited to daily observations only (not hourly) and they were further complicated by missing observations; however, this information can still be used to approximate the response of the watershed model to the January 1995 event.

Initially, it was assumed that the daily rainfall depth was uniformly distributed throughout the day for this initial simulation. The QUALHYMO simulation of the event demonstrated that only 8 x 10 m^3 of runoff water were draining into Second Pond. However, it was apparent that the air temperatures and the daily rainfall distribution estimates used for the initial simulations were not allowing enough of the snow accumulations on the ground to melt to the degree it was observed at St. John's West CDA Climatological Station (Table 4.9).

Through successive additional simulations, it was determined that by simply redistributing the daily rainfall depth, the runoff volume for the same seven-day period could be increased up to 10 x 10^6 m^3. By further assuming an hourly temperature variation during the day, the runoff volumes were further increased up to 12 x 10^6 m^3.

TABLE 4.9
Meteorological Summary - January 1995

	St. John's East Climatological Station				St. John's West CDA Climatological Station				Petty Harbour Climatological Station			
	Avg. Temperature (°C)	Rain (mm)	Snow (cm)	Snow on Ground (cm)	Avg. Temperature (°C)	Rain (mm)	Snow (cm)	Snow on Ground (cm)	Avg. Temperature (°C)	Rain (mm)	Snow (cm)	Snow on Ground (cm)
Jan 1 - 6	-4.0	0	12	17.0	-3.7	3.8	29.7	36	N/A	0	23.0	N/A
Jan 7	N/A	N/A	N/A	17.0	1.8	20.6	0	32	N/A	37.2	0	N/A
Jan 8	N/A	N/A	N/A	0	4.8	27.8	0	12	N/A	16.0	0	N/A
Jan 9	N/A	N/A	N/A	0	-1.0	0	Tr	12	N/A	0	0	N/A
Jan 10	N/A	N/A	N/A	0	-2.0	0	0.4	12	N/A	0	0	N/A

While these efforts did not directly verify the calibrated hydrologic model, the correlation between the observed and calibrated model is reasonable - particularly because the input data for these simulations were not consistent with the hourly data from which the model was calibrated. Overall, the model did respond to the extreme nature of the event and given the variability of meteorological conditions with the study areas, as shown in Table 4.9, the model did in fact respond with reasonable results.

4.4 Frequency Analysis

The modelling results were subject to frequency analyses to determine the 1:20 year and 1:100 year flood flows in the Goulds area only. The Consolidated Frequency Analysis Package (CFA88, Environment Canada, 1991) was employed to develop probability distributions which included:

- Generalized Extreme Value (GEV);
- Three Parameter Log Normal (3PLN);
- Log Pearson Type III (LP3); and
- Wakeby.

The estimated return period flood flows (i.e., 1:20 and 1:100 year) were compared to results from the previous modelling study (Delcan, 1991) and estimates using the regional flood frequency analysis prepared by the Canada-Newfoundland Flood Damage Reduction Program (1984). Similar frequency analyses using gauged flow records were employed to determine the return period flood estimates for the Petty Harbour area. Results of the evaluations for both the Goulds area and Petty Harbour area are presented below.

4.4.1 Frequency Analysis - Goulds Area

Statistical estimates of the 1:20 year and 1:100 year events were derived from Table 4.8 and are presented in Table 4.10. Also shown in the Table are estimates of return period flows from the 1991 Delcan study and the Province's Regional Analysis equations (Technical Appendix B).

TABLE 4.10
Comparison of Estimated Flood Peaks to Earlier Investigations

Basin Name	Drainage Area (km^2)		Flood Flows (m^3/s)					
			1:20 year			1:100 year		
Cochrane Pond Brook	30.9 (A)	29.9	64 (B)	79 (C)*	72 (D)	82 (B)	109 (C)*	91 (D)
Doyles River	14.6 (A)	12.3	78 (B)	62 (C)*	68 (D)	99 (B)	90 (C)*	84 (D)
Raymond Brook	63.7 (A)	62.4	64 (B)	86 (C)	61 (D)	81 (B)	113 (C)	83 (D)
Third Pond	116 (A)	112.1	194 (B)	173 (C)*	171 (D)	248 (B)	235 (C)*	233 (D)

(A) Delcan Drainage Area (extracted from 1991 report)
(B) Flood Flows from Delcan Study (1991)
(C) Flood Flows using Regional Analysis equations (1990)
(D) Flood Flows estimated in this investigation
* FACLS and FLSAR out of recommended range for Regional Analysis

Overall, the frequency analysis of the 32 years of modelled results provided return period flood flow estimates which are within the range of the results from the previous study and the Province's Regional Analysis at selected locations within the basin. The frequency analyses prepared for this report are provided in Technical Appendix C.

4.4.2 Frequency Analysis - Petty Harbour Area

Runoff entering into Second and First Ponds makes its way downstream to Petty Harbour via the Powerhouse intake or by spilling over the dam located at the outlet of First Pond. Historically, the water draining to Petty Harbour has been monitored by Newfoundland Light and Power Company Ltd. since 1944. Streamflow records have also been reported by Water Survey of Canada since 1963.

For this investigation, the Powerhouse and WSC records were retrieved and examined. The objective of this part of the investigation was to estimate the 1:20 year and 1:100 year flood flows using historical records from the Water Survey Streamflow Station and/or the Power Company. After examining the data, the historical streamflow records from the WSC were employed in estimating the flood flows for Petty Harbour area, since only monthly runoff volumes were available from historical Powerhouse records.

Statistical analysis of the maximum daily peaks from WSC records using CFA88 was again undertaken, and the estimated flood flows were then compared to flood flows reported in an earlier study by Newfoundland Design Associates Limited (1987). Results of the analyses are presented in Table 4.11.

TABLE 4.11
Maximum Daily Flood Flows (m^3/s)

Return Period	WSC Data (1963-1993)	1987 Reported Flood Estimate (1954-1983)
20 Year	76	68
100 Year	101	--

-- Not Reported

As shown in the above table, the estimated 1:20 year flood flow from the present investigation is about 10% higher than the earlier reported flood flow. This difference results from different periods of record in each study.

4.5 Design Rainfall Event Simulation

Extreme rainfall events (such as the 1:20 and 1:100 year storms) will also cause high flows which may be higher than the composite rainfall-snowmelt events discussed earlier in the report. Development of runoff peak flow rates from these rainfall events is described in the following sections.

4.5.1 Design Rainfall Distributions

The rainfall distribution used in this investigation was extracted from the 1988 hydrotechnical study of the Waterford River Basin (approximately 6 km north of the present study area). Table 4.12 summarizes the distribution of total design rainfall determined for the St. John's area for the 12-hour storm event. It was noted in the Waterford River Basin Study that the longer duration storms (i.e., 12-hour durations) produced the maximum flow response and, consequently, due to proximity and similar hydrologic features, the storm distribution was assessed to be appropriate for this investigation. The total rainfall depths for the 20 and 1:100 year are approximately 80 mm and 97 mm, respectively.

TABLE 4.12
Design Rainfall Distribution [A]

Time (hr)	Percent of Total Rainfall	Intensity (mm/hr)	
		20 yr.	100 yr.
0-1	1.0	0.8	0.97
1-2	1.0	0.8	0.97
2-3	5.0	4	4.85
3-4	11.0	8.8	10.67
4-5	20.0	16	19.4
5-6	25.0	20	24.25
6-7	18.0	14.4	17.46
7-8	10.0	8	9.7
8-9	5.0	4	4.85
9-10	2.0	1.6	1.94
10-11	1.0	0.8	0.97
11-12	1.0	0 .8	0.97

Rainfall Totals*

12-hr 20 Year	80 mm
12-hr 100 Year	97 mm

*Based on IDF from St. John's Airport

[A] Hydrotechnical Study of the Waterford Area, 1988 (storm distribution is based on work by W.D. Hogg, Hydrometeorology Division, Atmospheric Environment Service, Downsview)

Initial conditions or the antecedent moisture conditions (API) in the watershed must reasonably reflect average or typical conditions which would likely be present just prior to the start of such storms. In this investigation, 32 major storms from 1961 to 1992 were examined for conditions just prior to high flow events. It was determined that the average soil moisture condition in the watershed before the annual peak flow events were wet, but not extremely wet (i.e., above AMCII condition). An average API of 39 mm was selected as typical for these rainfall events. This is slightly higher than an AMCII condition (33 mm), yet significantly lower than AMCIII (56 mm).

Table 4.13 summarizes the results of the design rainfall simulations.

TABLE 4.13
Design Rainfall Event Peak Flow Rates

Basin	1:20 year 12-Hour Rainfall Event (m^3/s)	1:100 year 12-Hour Rainfall Event (m^3/s)
Cochrane Pond Brook	55	68
Doyles River	41	51
Raymonds Brook	45	54
Third Pond	157	193

4.6 Summary of Results

It is appropriate to compare the modelled frequency based flow estimates and the results of the design storm simulations.

As noted in Table 4.14, it is apparent that the results of the frequency analyses yield higher runoff rates than the design rainfall simulations. This result clearly shows that snowmelt alone, snowmelt with rainfall, and rainfall alone all join in to play a role in defining flood peaks in the study basin. The design rainfall events provide reasonably good estimates of flood flows but they are somewhat lower than projections using a simulated time sequence (32 years) of flows. Hence, the frequency based flood peak estimates have been selected for determining backwater conditions within the study basin.

TABLE 4.14
Summary of Design Storm and Flood Frequency Peak Flow Estimates

Basin Name	1:20 year		1:100 year	
	Design Rainfall (m^3/s)	Frequency (m^3/s)	Design Rainfall (m^3/s)	Frequency (m^3/s)
Cochrane Pond Brook	55	72*	68	91*
Doyles River	41	68*	51	84*
Raymond Brook	45	61*	54	83*
Third Pond	157	171*	193	233*
Petty Harbour**	N/A	76**	N/A	101**

* Based on QUALHYMO simulated annual peaks

** Based on WSC records (1963-1993) at Petty Harbour River at Second Pond

5.0 HYDRAULIC ANALYSIS - GOULDS AND PETTY HARBOUR STUDY AREAS

5.1 Introduction

The purpose of the hydraulic investigation is to compute the design flood profiles for the 1:20 year and 1:100 year year events for the Petty Harbour River in the Town of Petty Harbour, and for Doyles River, Dirty Bridge River, Cochrane Pond Brook and Raymond Brook in the Town of Goulds. To undertake this investigation, the HEC-2 backwater model was used.

The HEC-2 model (US Army Corps of Engineers, 1990) was used because it represents the state-of-the-art for computation of water surface profiles. It has been successfully used in many parts of Canada; is well documented; is parameter efficient for calibration, and is flexible to use.

The HEC-2 model accounts for various hydraulic conditions. These are:

- channel and floodplain conveyance
- channel constrictions at bridges, weirs and other hydraulic structures
- various peak flow conditions
- various starting water conditions.

5.2 Brief Description of the HEC-2 Model

The HEC-2 model was developed by the U.S. Corps of Engineers, Hydrologic Engineering Centre to compute water surface profiles for gradually varied steady state flow in natural or man-made channels. The model estimates the change in water surface elevation between given river cross-sections with special computation methods for bridge structures and other flow obstructions in the flood plain. The basic computational procedure used in the model is the solution of the one-dimensional energy equation with energy loss due to friction evaluated with Manning's equation.

Full details of the HEC-2 model and its underlying theory are given in the user's manual.

5.3 Model Setup Goulds Study Area

Two HEC-2 models for the Town of Goulds were developed. The first model, for Doyles River, consists of the main branch of Doyles River, plus tributaries Dirty Bridge River, Cochrane Pond Brook, and Fourth Pond Brook. The second model was for Raymond Brook.

For both models, the river reaches downstream of Route No. 10 (the Southern Shore Highway) were modelled using modified cross-sections from the previous HEC-2 model (Delcan Corporation 1991). The river reaches upstream of the highway were set up based on field surveyed cross-sections undertaken in August and November of 1994 (with the exception of Doyles River upstream of the junction with Fourth Pond Brook, where Delcan Corporation information was used). All sections were coded adopting the convention of looking downstream from left to right.

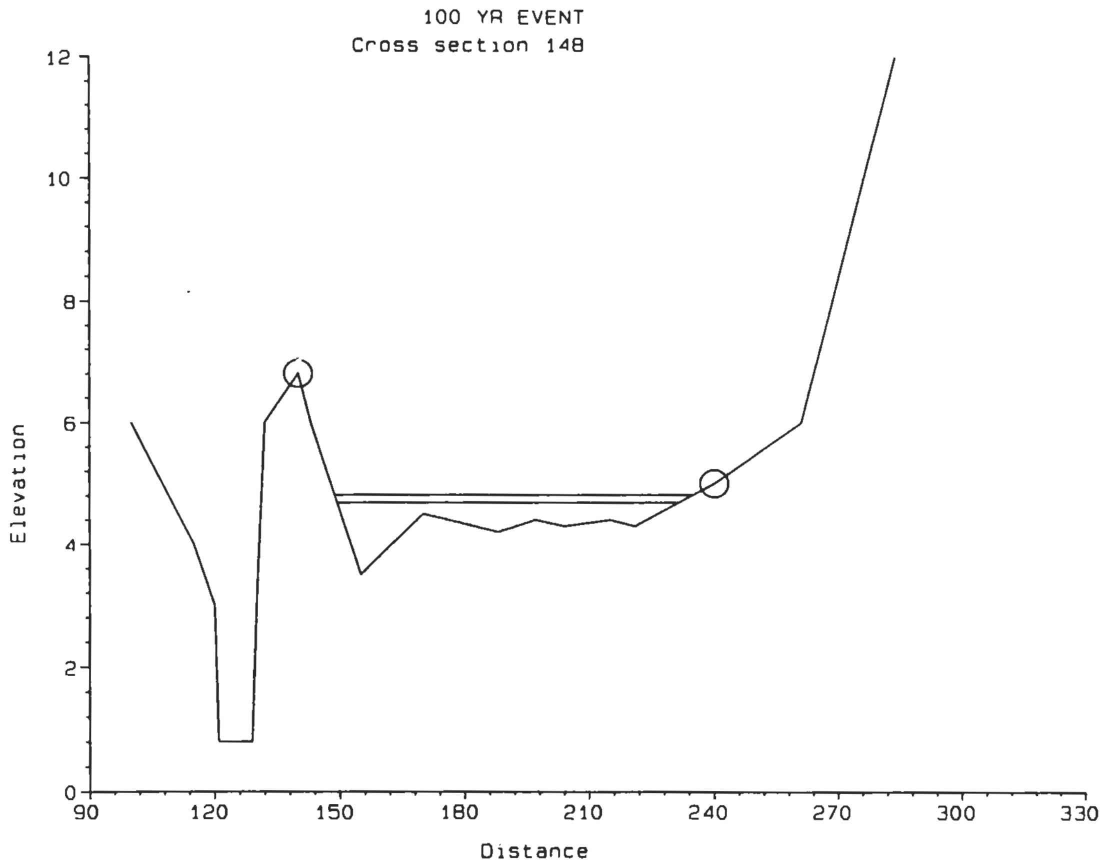
The Doyles River model was set up from Third Pond to Doolings Line, a distance of approximately 3.3 km and includes 78 cross-sections. The Dirty Bridge River model extends from Third Pond to Back Line Road, a distance of approximately 1.5 km and includes 20 cross-sections. The Cochrane Pond Brook model extends from Third Pond to 600 m upstream of the highway, a total distance of approximately 1.4 km and includes 16 cross-sections. The Raymond Brook model was set up from Third Pond to 450 m upstream of the highway, a total distance of approximately 1.3 km and includes 19 cross-sections. The Fourth Pond Brook model consists of approximately 180 m and seven cross-sections, extending from the junction with Doyles River to north of the Petty Harbour Road.

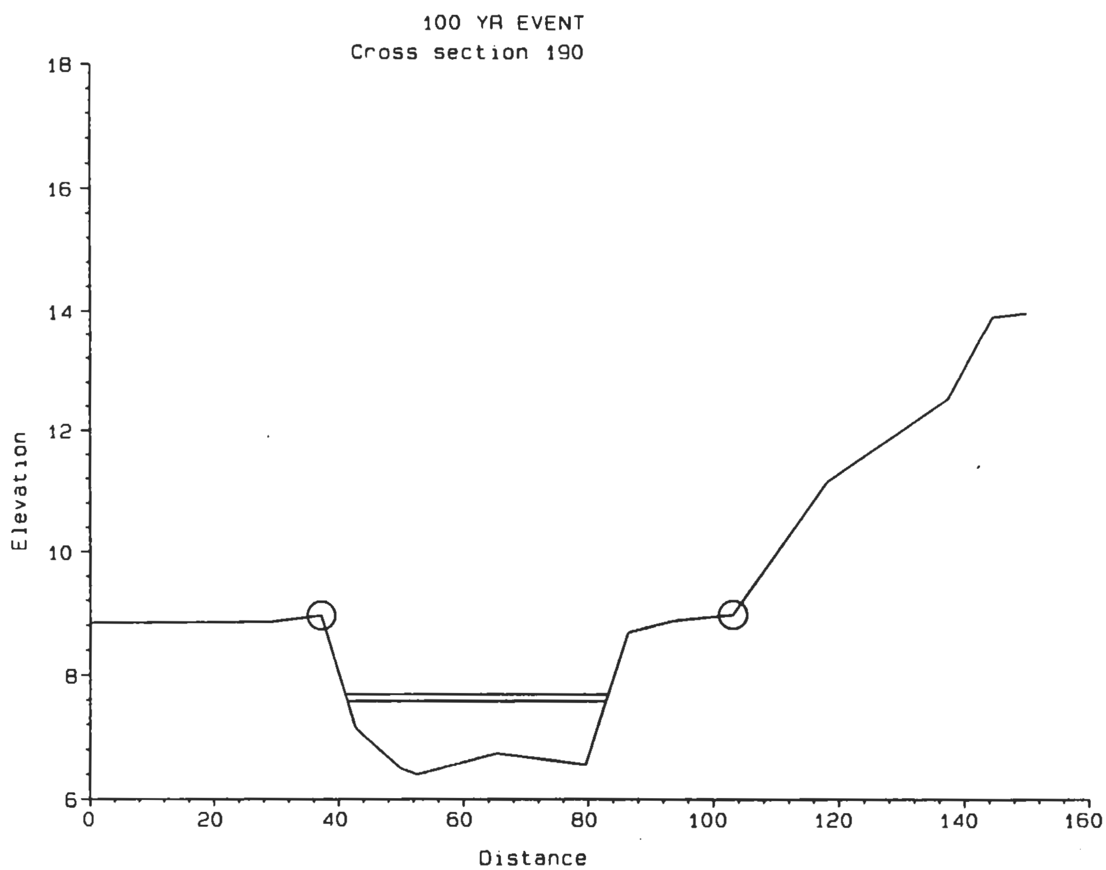
Revisions to the Delcan Corporation portion of the model included reversing all cross-sections to follow the convention of looking downstream from left to right, and extending several cross-sections based on the 1:2500 scale topographic mapping to avoid overtopping. As part of the terms of reference, several cross-sections were checked based on new survey information. A total of six cross-sections were checked, and in all cases the field surveys ultimately confirmed the adequacy of those cross-sections.

Figure 5-1 shows an example of two of the cross-sections employed in the study. Plots of each cross-section are provided in the Technical Appendix. The cross-section plots show the ground surface (and in the upper figure, the powerhouse tailrace on the left), the location of the tops of the banks (for modelling purposes), and the 1:20 year and 1:100 year water levels (the two horizontal lines).

Bridges

The HEC-2 models include 13 bridges: nine on Doyles River, and one each for Dirty Bridge River, Cochrane Pond Brook, Fourth Pond Brook and for Raymond Brook. The bridges at Cochrane Pond Brook and Raymond Brook were recoded based on the new survey information. The bridges were coded using the normal bridge method. As there was some discrepancy between the datums used in the Delcan Corporation model and the new survey information, a correction factor was added to match the two datums (using top of road on bridge sections as a reference elevation).





EXAMPLE CHANNEL CROSS-SECTIONS

FIGURE 5-1

A listing of the HEC-2 input data for the Doyles River model and Raymond Brook model is presented in Technical Appendix D.

5.4 Model Setup Petty Harbour

The Petty Harbour River model was set up from the mouth of the Petty Harbour River upstream 410 m, and includes nine cross-sections. The first two bridges were coded. The model was set up based on field surveyed cross-sections undertaken in August 1994.

5.5 Model Calibration and Verification

Before final backwater model simulation of the 1:100 year and 1:20 year flood levels can be confidently undertaken, the models must be calibrated. This exercise entails a process by which model parameters (i.e., roughness coefficients, bridge coefficients, etc.) are adjusted to make the model reproduce known (observed) historical water levels. When high quality water level and discharge data are available (i.e., at Water Survey of Canada streamflow gauges), this task is often easily undertaken. However, in the absence of precise water level and discharge data, calibration is less rigorous - yet it is still a valuable tool to confirm the adequacy of predicting the design flood profiles.

For this investigation, observations and photographs of historical flood events (gathered during site interviews) served as the principal source of data and were supported by site observations made during the study.

Precise streamflow estimates during the historical flood events are not available and, therefore, the historical model simulations described in the preceding chapters were used to supplement this information.

In the Goulds area, notable river-flooding events have almost always occurred when river ice has been a complicating factor. For those few occasions when ice effects were limited and flow observations were made, Table 5.1 gives observed water levels and approximate flows. One historical high flow event is included in this list (Doyles River 1984 flood). Table 5.4, given later, provides further information on ice effects.

TABLE 5.1
Calibration - Open Water Flooding Events*

Location	Estimated Flood Elevation (m)	Simulated Flood Elevation (m)	Estimated Flood Discharge (m^3/s)
Raymond Brook Cross-section 4400		79.2	15
Cochrane Brook Cross-section 40	70.5 70.0	70.4 70.0	19. 9.
Doyles River (upstream of 1610)	no observations	---	---
Doyles River Cross-section 500	76.6	76.8	72

* limited ice effects

Table 5.1 illustrates that there is reasonable agreement between the observed and simulated backwater levels. Given the somewhat limited information base for open water conditions and this agreement, the parameters selected for backwater conditions were set and the model verified.

5.6 Computation of Water Surface Profiles

The calibrated/verified HEC-2 model was used to compute the design water surface profiles for the 1:20 year and 1:100 year return period events. The peak flow rates computed as part of the hydrologic investigation were used as input to the HEC-2 model. Table 5.2 lists the peak flow rates used in the HEC-2 model.

TABLE 5.2
Peak Flow Rates for HEC-2 Model[1]

River	1:20 Year Flow (m^3/s)	1:100 Year Flow (m^3/s)
Cochrane Pond Brook	72	91
Doyles River	68	84
Raymond Brook	61	83
Dirty Bridge River	9.6	12.6
Fourth Pond Brook	16.7	13.8
Petty Harbour[2]	76	101

[1] Flood Frequency Peak Flow Estimates based on QUALHYMO simulated annual peaks
[2] WSC Records at Petty Harbour River

To determine the water level conditions for the various design flood profiles, a statistical analysis was performed on the annual peak outflows from Third Pond to derive the 1:20 year and 1:100 year return period outflow rates. These were 147 m^3/s and 168 m^3/s, respectively. Using the rating curve developed for Third Pond, these translate to water levels of 69.6 m and 69.78 m for the 1:20 year and 1:100 year return period events, respectively. An evaluation of the timing of peak annual inflows from Doyles River, Cochrane Pond Brook, Raymond Brook and the local catchment tributary to Third Pond, versus the timing of peak annual outflows from Third Pond was conducted. For 24 of the 32 years modelled (i.e., 75% of the time), the timing of peak outflows coincided with the timing of peak inflows from these four tributary sources. This validates the use of the annual peak outflows from Third Pond to derive the 1:20 year and 1:100 year return period water level conditions for Third Pond.

Similarly, the starting water level (2-year condition) which corresponds to mean annual discharges from the pond was determined to be 69.08 m.

5.7 Flood Level Determination

As indicated previously, the purpose of this study is to derive open water surface profiles for the 1:20 and 1:100 year return period flood flows. This includes water levels arising from flood flows on the river and those arising from high water levels from the sea. Typically, sea levels (1:20 year and 1:100 year) determine flood hazards at the downstream-most locations, and river flows determine the extent of flood risk upstream from the river mouth.

5.8 Starting Water Levels

The starting water level for backwater modelling at Petty Harbour was taken from water levels at St. John's Harbour. Water levels have been recorded there for many years, but it is only since 1962 that maximum instantaneous values have been taken by the Marine Environmental Data Service (MEDS).

The mean high tide level (mean of large tides and average tides) at the datum employed by MEDS is 1.295 m. This datum at St. John's is 0.677 m below the geodetic datum employed for this study. Hence, the mean high tide level is 0.62 m (GSCD). As it is quite possible that this water level would be present during the course of high river flows, 0.62 m (GSCD) was selected as the backwater starting level.

Starting water levels for the Goulds area were derived from the analysis of water levels at Third Pond. This analysis gave levels for initiating the backwater modelling of Raymond Brook, Cochrane Pond Brook, Dirty Bridge Brook and the Doyle's River (i.e., 69.08 m). It also gave 20 and 1:100 year levels at Third Pond for use in delineating the flood risk areas along the edge of the pond.

5.9 Backwater Sensitivity Analysis

A series of simulations was conducted with the hydraulic model to determine the sensitivity of flood level projections to changes in certain parameters. Included on this list are variations in channel and bank roughness parameters, the starting water level (sea level for Petty Harbour, and Third Pond for the other study areas), and flood flow. All sensitivity tests were conducted with the 1:20 and 1:100 year flow rates.

The tests were conducted independently using the base models prepared for the study area (i.e., channel roughness variations were conducted separately from other tests, then the starting water level variation tests were conducted separately, and so on).

In that there are over 150 cross-sections which have been modelled, the analysis of water level sensitivity to various factors is described below for only selected locations. These are:

Petty Harbour River

Section 40 - just upstream of the harbour bridge
Section 148 - at downstream end of the powerhouse
Section 315 - about 50 m downstream of the powerhouse road crossing

Raymond Brook

Section 4320 - midway from Southern Shore Highway to Third Pond
Section 4340 - just downstream of Southern Shore Highway
Section 60 - just upstream of Southern Shore Highway

Cochrane Pond Brook

Section 20 - midway from Southern Shore Highway to Third Pond
Section 90 - just upstream of the Southern Shore Highway
Section 510 - about 500 m upstream of the Southern Shore Highway

Dirty Bridge Brook

Section 65 - just downstream of the Southern Shore Highway
Section 9 - about 450 m upstream of the Southern Shore Highway and south of McConnell Place
Section 13 - about 650 m upstream of the Southern Shore Highway near Hannaford Place

Doyles River

Section 400 - at Riverside Drive trailer park
Section 1600 - at Petty Harbour Road gasoline station
Section 2800 - just upstream of Bishop's Line

Fourth Pond Brook

Section 23 - just upstream of Petty Harbour Road

Table 5.3 gives these section numbers and the selected water levels (1:100 year and 1:20 year) followed by the water levels resulting from the various sensitivity tests.

The table illustrates that Petty Harbour flood levels are: insensitive to modest changes in channel roughness; increase/decrease by 1% to 3% (maximum 0.14 m) with 15%± changes in flood flow; and are insensitive to changes in the starting water level in the harbour.

Raymond Brook flood levels are insensitive to channel roughness, and insensitive to flood flow changes downstream of the Southern Shore Highway. Levels are increased/decreased by about 0.3

TABLE 5.3
Flood Level Sensitivity Analysis

Location and Section No.	Water Level 1:100 year 1:20 year (m)	Channel Roughness		Flood Flow		Starting Water Level (m) Changes	
		+15%	-15%	+15%	-15%	+1.31 m	-0.40 m Petty Harbour
						+0.52 m	-0.52 m Third Pond
Petty Harbour							
40	**2.65**	2.64	2.65	2.72	2.57	2.66	2.63
	2.51	2.51	2.51	2.58	2.46	2.55	2.51
148	**4.81**	4.79	4.79	4.86	4.73	4.82	4.80
	4.69	4.69	4.69	4.74	4.70	4.69	4.69
315	**13.86**	13.85	13.85	13.98	13.72	13.86	13.86
	13.62	13.62	13.62	13.74	13.49	13.62	13.62
Raymond Brook							
4320	**74.15**	74.15	74.15	74.27	74.01	74.15	74.15
	73.88	73.88	73.88	74.00	73.75	73.88	73.88
4340	**79.11**	79.11	79.11	79.18	79.03	79.11	79.11
	78.96	78.96	78.96	79.03	78.89	78.96	78.96
60	**82.48**	82.50	82.47	82.85	82.12	82.48	82.48
	81.82	81.84	81.79	82.11	81.46	81.82	81.82
Cochrane Pond Brook							
20	**70.41**	70.41	70.41	70.44	70.35	70.42	70.42
	70.32	70.32	70.32	70.37	70.27	70.34	70.32
90	**72.18**	72.21	72.16	72.38	71.80	72.18	72.18
	71.65	71.68	71.63	71.92	71.35	71.65	71.65
510	**79.83**	79.83	79.83	79.91	79.73	79.83	79.83
	79.69	79.69	79.69	79.77	79.61	79.69	79.69
Dirty Bridge Brook							
65	**71.73**	71.86	71.58	71.81	71.63	71.73	71.53
	71.56	71.68	71.43	71.65	71.46	71.57	71.50
9	**83.56**	83.63	83.52	83.67	83.44	83.56	83.56
	83.36	83.43	83.32	83.46	82.28	83.36	83.36
13	**85.43**	85.44	85.43	85.47	85.97	85.43	85.43
	85.39	85.39	85.37	85.97	85.39	85.39	85.36
Doyles River							
400	**71.58**	71.58	71.58	71.61	71.50	71.56	71.55
	71.48	71.48	71.48	71.55	71.40	71.47	71.50
1600	**91.90**	91.96	91.86	92.15	91.65	91.90	91.90
	91.58	91.64	91.53	91.79	91.39	91.58	91.58
2800	**101.37**	101.37	101.37	101.43	101.77	101.37	101.37
	101.76	101.77	101.76	101.36	101.44	101.76	101.76
Fourth Pond Brook							
23	**91.87**	91.88	91.85	91.90	91.84	91.87	91.87
	91.83	91.84	91.81	91.86	91.80	91.83	91.83

m to 0.4 m upstream of the highway, however, with a 15%± change in flow. For this reason, it is recommended that the water levels associated with a flow increase of +15% be adopted in this area, and they have been plotted on the flood plain maps and appended (Technical Appendix F). Water levels are insensitive to change in the starting level at Third Pond.

Flood levels at Cochrane Pond Brook are relatively insensitive to variations in channel roughness, flood flows and the water level at Third Pond. There is some minor sensitivity (+.3 m and -.3 m) to change in flood flow in the area just upstream of the Southern Shore Highway.

Flood levels on Dirty Bridge Brook are relatively insensitive to variations in channel roughness, flood flows and starting water levels at Third Pond. Flood levels in the area around Hannaford Place, where the natural channel has been realigned, are sensitive to flow change and may be 0.5 to 0.6 m higher than shown on the flood plain maps - suggesting that the realigned channel is undersized and that development in the area should be set back from the channel to account for this sensitivity.

Flood levels on the Doyles River show some variability (0.4 m±) in response to flood flows at the Petty Harbour Road gasoline station. Otherwise, levels are relatively insensitive to change in flood flows, channel roughness and the backwater starting level at Third Pond.

Fourth Pond Brook flood levels are insensitive to changes in channel roughness, flood flows and starting water levels at Third Pond.

Sensitivity to ice conditions (ice jams/or snow-packed channels) was identified in the model calibration and verification stages and is well known to be a source of overbank flooding. This is particularly the case in the raceway area, at the highway bridges over Raymond Brook and Cochrane Pond Brook, and on Doyles River upstream of the gasoline station at Petty Harbour Road. Generally, the problems occur during normal annual runoff (about 1:2 year flows) and cause flood levels which are in the range of the 1:100 year level).

Several model simulations were conducted in an iterative process to simulate the configuration of the historical ice blockages and replicate the resulting flood levels. Reasonable results were obtained by raising the bottom of the channel until observed flood levels were obtained. The disadvantage of this approach is that it cannot be applied for flows in the 1:20 and 1:100 year range, because these flows would completely clear the ice from the channel.

As sophisticated ice jam modelling was beyond the scope of this analysis, the locally observed flood levels were plotted (as 1:20 year levels) if they exceeded the open water 1:20 or 1:100 year levels.

These observed levels are very good indicators of maximum ice-effect levels because they reflect both the configuration and strength of the local ice jams. The following levels were introduced.

TABLE 5.4
Ice Effect Levels

Location and Cross-Section Number		Ice Effect Level (m)
Raymond Brook	Section 4400	88.2
Raymond Brook	Section 15	71.5
Raymond Brook	Section 4200	70.0
Doyles River	Section 1600	92.5
Doyles River	Section 1610	94.0
Doyles River	Section 2810	101.0

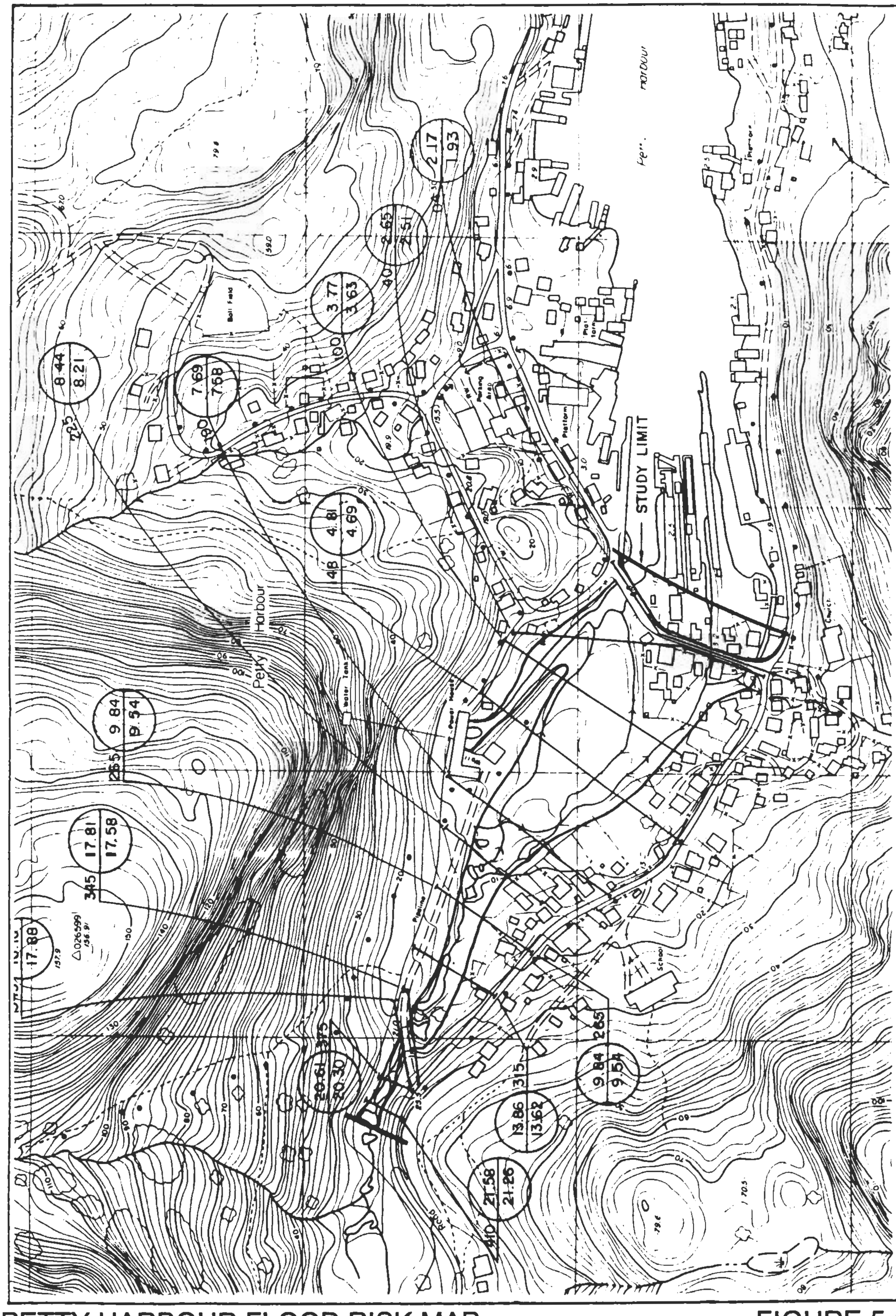
Overall, the water levels developed in the backwater analysis and plotted on the flood plain mapping sheets are good representations of the 1:20 year and 1:100 year open water flood levels and those with ice effects. As shown in the sensitivity analysis table, many factors can influence the flood levels and flood line locations and it is recommended that each municipality provide an offset (e.g., 10 m or 20 m) from the plotted floodlines to account for these individual variabilities and their possible combination.

5.10 Flood Level Delineation

5.10.1 Petty Harbour River

The complete calibrated/verified HEC-2 model was run with a starting level of 0.62 m and the 20 and 1:100 year peak flows. This provided water level data at all cross-sections in the model, and a summary printout of the results for both flood flows is presented in Technical Appendix E.

The cross-section locations in the model are plotted in Figure 5-2, which is a photo-reduction of a portion of the flood plain mapping. Each section is marked as a line on the map and the circle connected to the line gives the 1:100 year and 1:20 year flood water level (at the top and bottom of the circle, respectively). The dashed and solid lines connecting the cross-sections delineate the 1:100 year and 1:20 year flood hazard areas, respectively. There are many locations along the river where



PETTY HARBOUR FLOOD RISK MAP FIGURE 5-2

the 1:100 year flood levels are only slightly higher than the 1:20 year levels. In these locations, only the 1:20 year flood line has been plotted.

The historical series of monitored instantaneous water levels (tides and surges with no wave effects) also provides a data set for evaluating the 1:20 year and 1:100 year flood elevations near the river mouth where flood levels are dominated by the sea. MEDS records indicate that the highest annual levels occur in the winter months when streamflows are low. Frequency analyses of these levels provide the following results:

Petty Harbour River
Instantaneous Sea Water Levels at the Mouth

Return Period	Instantaneous Level (GSCD)
1:20 year	1.54 m
1:100 year	1.69 m

Concurrent analyses of storm surges at nearby Ferryland (Section 6.0) suggest that the storm surges themselves account for the majority of the instantaneous level observed above (i.e., 1.12 m at the 1:20 year return period and 1.36 m for the 1:100 year). In that it is also reasonable to expect that these surges could occur at a 2-year high tide (0.81 m GSCD) gives the following water levels which were selected for use at Petty Harbour.

TABLE 5.5
Petty Harbour River
Design Sea Water Levels at the Mouth

Return Period	High Tide and Surge Level (GSCD)
1:20 year	1.93 m
1:100 year	2.17 m

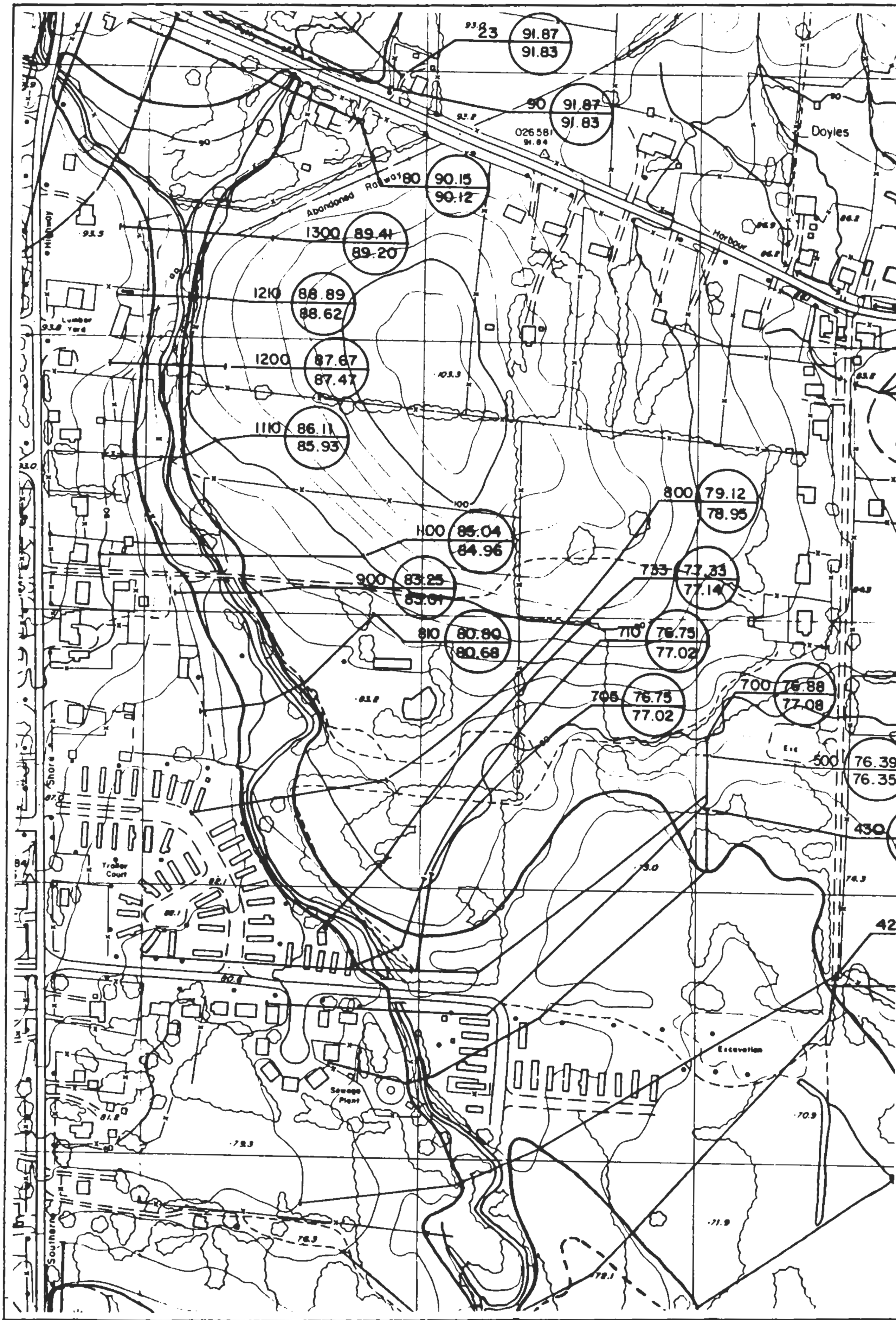
Hence, to show both possibilities of flooding (from high flows on Petty Harbour River and high sea levels at the river mouth) the highest of the sea levels or levels resulting from river flows must be plotted near the mouth to designate the overall flood hazard.

5.10.2 Goulds Area

The HEC-2 modelling was initiated for the brooks and river in this section of the study area at a starting water level of 69.08 m at Third Pond. From there, the backwater was carried upstream in two mathematical models. One model evaluated backwater conditions for Raymond Brook, while the other had subcomponents which gave levels for the Doyle's River and inflows from Cochrane Pond Brook, Dirty Bridge Brook and Fourth Pond Brook.

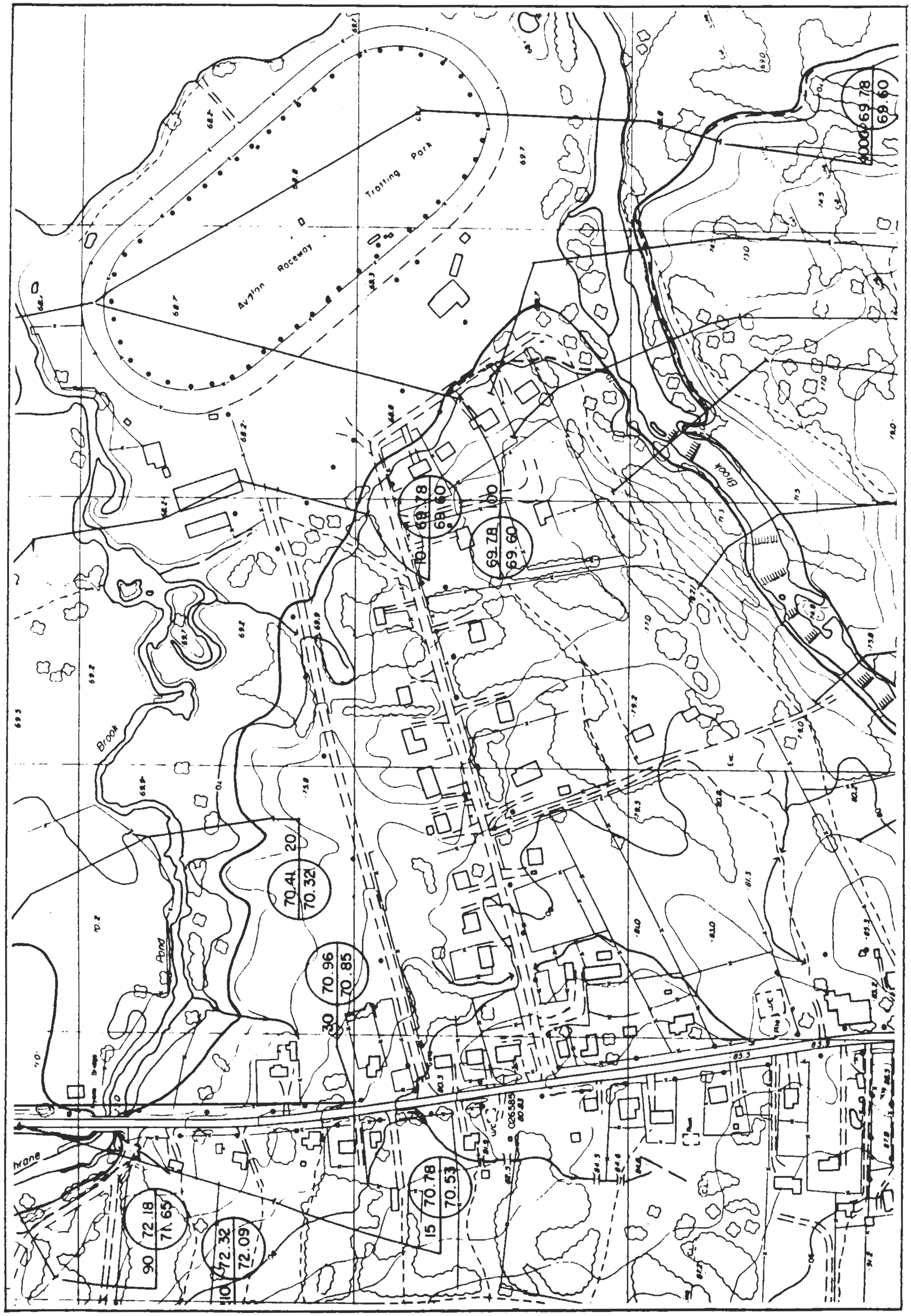
Typical cross-section locations in the model are shown in Figure 5-3 and Figure 5-4. These figures are photo-reductions of portions of the floodplain mapping and they show the 20 and 1:100 year flood levels in two ways. The circle connected to each cross-section gives the 1:100 year flood level at the top of the circle and the 1:20 year flood level at the bottom of the circle. The flood lines which connect each cross-section on the map show the 1:20 year flood hazard area (the solid line) and the 1:100 year flood hazard area (dashed line). The 1:20 year flood lines are the only ones plotted in areas where the 20 and 1:100 year levels are in close proximity.

The overall results of the hydraulic analysis are provided in Technical Appendix E which gives the computed water surface elevations and other information for the 1:20 year and 1:100 year year return period events for each cross-section within the mapping limits. Table 5.6, which follows, summarizes the return period flood elevations for each cross-section. These flood profiles were plotted on 1:2500 scale topographic mapping - photo-reduced examples of which are shown here. Specific sheets for a particular area can be obtained from the Department of Environment in St. John's.



GOULDS FLOOD RISK MAP - DOYLES RIVER

FIGURE 5-3



GOULDS FLOOD RISK MAP - COCHRANE POND BK. FIGURE 5-4

TABLE 5.6
SUMMARY OF 1:20 AND 1:100 YEAR FLOOD LEVELS
DOYLES RIVER, COCHRANE POND BROOK,
DIRTY BRIDGE RIVER, FOURTH POND BROOK

Cross Section	1:100 Year Level (m)	1:20 Year Level (m)
4000	69.78	69.60
100	69.78	69.60
300	69.78	69.60
400	71.58	71.48
410	72.06	71.99
420	74.01	74.00
430	74.00	73.99
500	76.39	76.35
600	76.88	77.08
700	76.88	77.08
705	76.75	77.02
710	76.75	77.02
733	77.33	77.14
800	79.12	78.95
810	80.80	80.68
900	83.25	83.01
1000	83.76	83.49
1000.1	84.73	84.64
1000.2	85.10	84.98
1000.3	85.21	85.09
1100	85.04	84.96
1110	86.11	85.93
1200	87.67	87.47
1210	88.89	88.62
1300	89.41	89.20
1310	90.63	90.53
1400	91.24	91.07
1500	91.32	91.16
1500.1	91.26	91.14
1500.2	91.34	91.20
1500.3	91.25	91.12
1600	91.90	91.58
1610	93.32	93.24
1700	94.30	94.24
1800	94.55	94.52
1800.1	94.75	94.58
1800.2	95.17	94.97
1800.3	95.78	95.51
1900	95.78	95.50
2000	95.79	95.51

Cross Section	1:100 Year Level (m)	1:20 Year Level (m)
2100	95.79	95.52
2100.1	95.64	95.41
2100.2	95.97	95.44
2100.3	96.08	95.45
2200	96.06	95.79
2300	96.43	96.34
2400	96.79	96.67
2400.1	96.79	96.67
2400.2	96.80	96.68
2400.3	96.80	96.68
2500	97.20	97.10
2510	99.67	99.59
2600	101.05	100.49
2700	101.12	101.75
2700.1	101.32	101.76
2800	101.37	101.76
2810	102.18	102.11
2820	104.28	104.19
2900	105.74	105.64
2910	108.39	108.33
2920	110.17	110.12
3000	111.91	111.86
3010	113.83	113.80
3020	115.85	115.80
3100	117.67	117.61
3200	117.81	117.75
3200.1	117.82	117.76
3200.2	118.01	117.89
3200.3	118.01	117.89
3300	118.57	118.47
3400	119.25	119.19
3500	119.34	119.26
3500.3	119.61	119.52
3600	119.63	119.55
-100	69.08	69.08
10	69.13	69.11
20	70.41	70.32
30	70.94	70.83
40	70.80	70.53
40.1	70.78	70.53

Cross Section	1:100 Year Level (m)	1:20 Year Level (m)
40.2	70.84	70.80
40.3	72.17	70.97
15	72.17	71.63
90	72.18	71.65
140	72.21	71.70
210	72.32	72.09
310	74.03	73.79
410	78.45	78.26
510	79.83	79.69
585	81.29	81.04
-300	69.15	69.12
60	69.96	69.87
65	71.73	71.56
70	71.79	71.62
70.3	71.79	71.62
75	72.64	72.51
5	75.83	75.66
6	77.94	77.78
7	81.95	81.88
8	83.27	83.11
9	83.56	83.36
10.1	84.64	84.57
11	85.21	85.14
12.1	85.43	85.35
12.3	85.43	85.44
13	85.43	85.39
14	85.64	85.58
15.1	87.12	86.92
16	88.50	88.39
17.1	89.83	89.77
17.3	89.91	89.86
18	89.93	89.88
-1300	89.41	89.20
80	90.15	90.12
85	90.97	90.80
85.1	91.66	91.64
85.2	91.84	91.81
85.3	91.87	91.83
90	91.87	91.83
23	91.87	91.83
231	92.36	92.30

TABLE 5.6
SUMMARY OF 1:20 AND 1:100 YEAR FLOOD LEVELS
RAYMOND BROOK

Cross Section	1:100 Year Level (m)	1:20 Year Level (m)
4100	69.78	69.60
4200	69.78	69.60
4300	69.78	69.60
4310	70.23	70.00
4320	74.15	73.88
4330	76.84	76.61
4340	79.11	78.96
4400	80.08	79.82
4500	81.24	80.82

Cross Section	1:100 Year Level (m)	1:20 Year Level (m)
4500.2	81.45	81.04
4500.3	81.46	81.04
10	82.82	82.08
60	82.85	82.11
140	82.77	82.35
210	83.07	82.94
320	83.84	83.55
440	84.48	84.26

PETTY HARBOUR RIVER

Cross Section	1:100 Year Level (m)	1:20 Year Level (m)
2	2.17	1.93
3	2.17	1.93
4	2.17	1.93
40	2.65	2.51
100	3.77	3.63
148	4.81	4.69
190	7.69	7.58
225	8.44	8.21

Cross Section	1:100 Year Level (m)	1:20 Year Level (m)
315	13.86	13.62
345	17.81	17.58
347	17.82	17.59
348	18.12	17.89
349	18.16	17.88
375	20.61	20.30
410	21.58	21.26

6.0 FLOOD RISK ASSESSMENT - FERRYLAND STUDY AREA

6.1 Introduction

The coastal communities on the east coast of Newfoundland have endured flooding due to storm surges, high waves and tides for centuries. The inherent nature of the fishing industry required most of the communities to be built near or on the shoreline. When the proximity of the residents is combined with the storm surge, high tides, and waves, there is a great potential for damage.

Generally, the flooding in Newfoundland coastal areas occurs in early winter and late fall due primarily to convectional storms and ice. However, there is occasional flooding due to extensive low pressure systems in the summer.

This study uses a simple analytical model to determine the 1:20 year, 1:50 year, and 1:100 year storm surge events for Ferryland. Also, the Bretschneider method was used to predict the design wave height/period. Site-specific wave prediction was done by using a simple shallow water wave propagation analysis.

6.2 Physiography and Climate

The region of interest in this study is the east coast of the Avalon Peninsula. This area is less rugged than the rest of the island with topography formed mainly by glacial retreat. The elevations are generally under 300 m with precipitation primarily due to frontal lifting.

Newfoundland generally has a temperate climate, however there is a great difference in the weather between the east and west coasts. On the west coast rain amounts tend to be smaller, due to orographic lifting, and snowfall amounts tends to be higher (Table 6.1). The main reason for the difference is due to the presence of the Labrador Current and the Gulf Stream.

The Labrador Current starts as a clockwise circulation over the Canadian Basin which then carries southward and mixes with the warm Gulf Stream off the Newfoundland Grand Banks. This mixing forms a region of high baroclinic instability which drastically increases the available potential energy (APE). Generally, low pressure systems track from the southwest with the gulf stream providing an excellent energy source. The Labrador Current causes the air mass to cool inducing precipitation and a large latent heat release. As a result the storms tend to be more severe than the rest of the island and the winter temperatures are higher, and the summer temperatures are cooler.

6.3 Methodology

Long term tide, water level, and meteorological data are not available for Ferryland. St. John's is the nearest community (60 km north) with adequate available data. Although St. John's is not in the study area, it can be shown that both places have approximately the same climate. Table 6.1 shows the precipitation values for Cape Broyle (10 km North of Ferryland), Petty Harbour, St. John's, and Corner Brook. The first three are located on the eastern Avalon and demonstrate little difference in precipitation, Corner Brook is situated on the west coast of the island and shows a significant difference. Considering the distance between Ferryland and St. John's, from a meteorological point of view, on a synoptic scale it can be assumed that there is no difference in weather.

TABLE 6.1
Precipitation Values for Various Sites

Site	Elevation	Coverage	Rain (mm)	Snow (cm)	Precipitation (mm)
Cape Broyle* (47°06'W, 52°56'N)	6 m	1955-1990	1346.8	226.8	1579.0
St. John's A.* (47°37'W, 52°44'N)	134 m	1942-1990	1163.1	322.1	1481.7
Petty Harbour* 47°28'W, 52°43'N)	6 m	1955-1990	1198.9	204.3	1403.8
Corner Brook** (48°57'W, 57°57'N)	5 m	1933-1990	771.0	414.4	1186.0

* East Coast location
** West Coast location

6.3.1 Storm Surge

The definition of storm surge is the increase in water level from the normal (mean sea level, MSL) due to the action of storms (wind stress and atmospheric pressure reduction) and its determination is a fairly convoluted process.

There are three main factors which contribute to storm surge. The first is the speed of the wind, generally the faster the wind the greater the surge. Secondly, the duration of the wind travelling over a water body in the same direction is important. If a constant velocity wind is blowing, it tends to

TABLE 6.2
Wave Hindcast for Ferryland (At Depth of 20 m Near Head of Peninsula)

Wind Speed (km/hr)	Wind Speed (m/s)	Equivalent Fetch (km)	Duration (hr)	Water Depth (m) HHW	Water Depth (m) HHW+SS	H_s (m)	T_p (s)	T_z (s)	L_o (m)
121	33.6	795	1	21.3		4.83	10.0	7.6	124.3
121	33.6	795	1		22.3	4.99	10.1	7.7	128.2
59	16.4	2310	12	21.3		3.20	7.8	5.9	86.7
59	16.4	2310	12		22.3	3.29	7.9	6.0	89.0

H_s = Significant Wave Height
T_p = Peak Period
T_z = Zero Crossing Period
L_o = Deep Water Wave Length
HHW = High Water Level
SS = Storm Surge

Note: The datum used in this report is the same found on the CHS chart, 0.6m.

increase setup. If there is a shift in wind direction (example, when a low centre passes), it tends to decrease the set-up. However, the duration is only important up to the point when water accumulation reaches a maximum (i.e., steady state). Finally, the fetch length is the distance wind travels over the water body. Since we are primarily concerned with winds directly off the Atlantic Ocean, the fetch length was assumed to be a maximum and did not enter into our calculations.

Thirty years of hourly wind speed and direction data were obtained for St. John's airport from the Atmospheric Environment Service (AES). The corresponding thirty years of water level data for St. John's Harbour were obtained from the Canadian Hydrographic Service (CHS) in Ottawa. The tide heights were generated from their constituents using our tidal prediction program.

All the data were put into a data base and programs were written to efficiently process the information. The AES hourly data were the first to be sorted; only the winds between 30 and 140 degrees true (directly off the water) were of interest. It was assumed that the maximum storm surges occurred only with onshore winds. This point was confirmed from local flooding and storm records. The associated times were then collected and cross-referenced with the CHS data to get the corresponding water level values. These were again cross-referenced with thirty years of tide data (generated from constituents). The tide data were then subtracted from the water level data and the resulting difference was the storm surge due to winds.

The problem with duration was overcome by selecting only those winds greater than 25 km/hr which originated from a constant direction for a minimum of twelve hours with a three hour gap allowed. The corresponding storm surges and winds were then used as input for an extreme analysis (using Gumbel Distribution) to determine the 2, 5, 20, 50, and 100 year return period.

6.3.2 Wave Prediction

It is important to evaluate all sources of wave data which are applicable to the study area. Unfortunately, there was no record of visual observations and no instrumentation in place to record wave height in the vicinity of Ferryland. However, there are several methods available to estimate the wave heights and peak periods from meteorological and hydrographic data. The method of choice was the Bretschneider Procedure (SMB) detailed in the U.S. Army Corps of Engineer's Shore Protection Manual (1984).

The wave hindcast for Ferryland was only done for the 20 m depth at the head of the peninsula. The associated results of the analysis can be seen in Table 6.2.

6.4 Analysis and Results

6.4.1 High Risk Areas

The study area can be delineated into six zones of high risk to potential flooding (Figure 6.1). The zones of high risk were determined from photos and video tape in conjunction with topographic and hydrographic maps for the area. The rear boundary for declination zones is the 10 m contour.

Zone 1 is the most vulnerable due to low elevations (maximum = 2.1 m) and presence of construction. It is bounded on both sides by the ocean and beach with and average slope of approximately 3.0% so runup can cause problems.

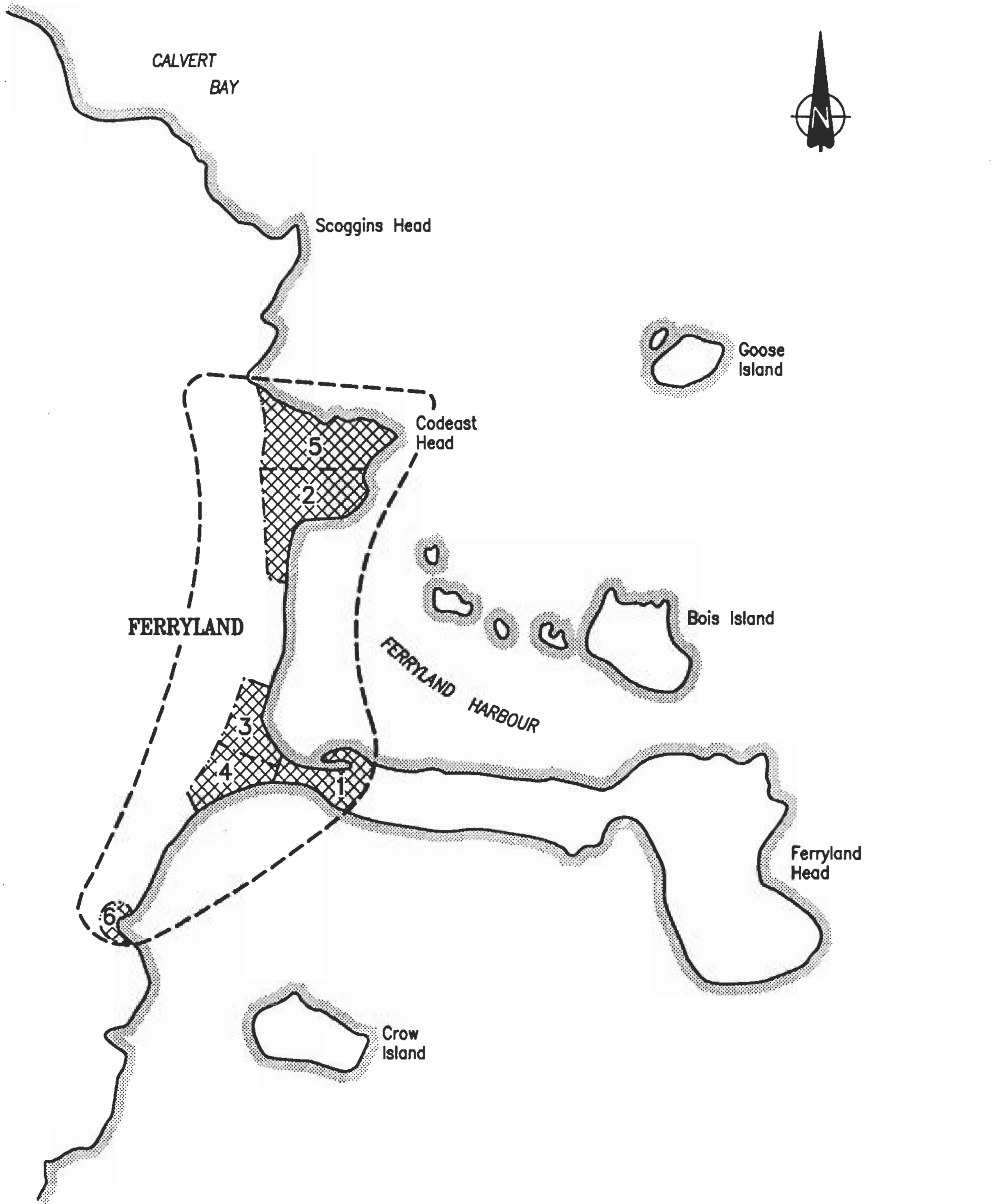
The second region of concern is Zone 2 which is relatively flat with an average elevation of 0.5 m. This zone has several buildings and flakes which are used in the fishing industry, and the replacement value could be very expensive. This zone is bounded by a beach with little, if any, wind brakes beside the buildings.

Zones 3 and 4 are similar in that the water-land interface is a beach. Zone 4 has a maximum elevation of 3.3 m at a retaining wall 20 m from mean sea level. It then slopes toward a minimum elevation of 0.7 m at the local softball field. This zone is very susceptible to surge generated from south-westerly winds. Zone 3 has a mean elevation of 0.7 m, which makes it very susceptible to storm induced waves and surge from the northeast. Both zones have construction and very little protection against onshore winds.

Zone 5, the most rugged of the six zones, has maximum elevation of 6 m with no construction under 4 m. The region is bounded on the north by a fairly rocky coastline and to the east with a relatively flat beach (slope = 1-2%).

In Zone 6, there are no buildings (only one 5 ft diameter culvert); however, it is included because a local access road crosses the area (with an elevation of 1.7 m) and it may be prone to washing out in a severe event.

The rest of the coast line included in the study area has fairly steep cliffs (approximately 10 m in height) with no construction on the shoreline.



LEGEND

- - - - - - STUDY AREA BOUNDARY

—·—·— ZONE BOUNDARY

3 ZONE NUMBER

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FLOOD RISK MAPPING STUDY OF
GOULDS, PETTY HARBOUR AND FERRYLAND

FIGURE 6.1

FERRYLAND ZONES OF
FLOOD RISK

SCALE APPROX. 1 : 25000

6.4.2 Surge, Waves and Tide Analysis Results

The results of the analysis can be seen in Tables 6.3 and 6.4. Table 6.3 shows the maximum wind values and their associated return period for St. John's Airport. Table 6.4 demonstrates the average wind in a sustained 12-hr event. These values are of paramount importance because it is the sustained wind which sets up the surge. It should be noted here that the top 214 events, with their associated values, are found in Appendix H. Tables 6.5 and 6.6 demonstrate the maximum surge values with associated return periods. Table 6.5 gives the maximum surge value for the past thirty years of data, while Table 6.6 gives the maximum surge for 31 of the top storm events as classified by highest average wind speed. There is little difference between the values of both tables; however, since we are only interested in maximum surge, the values in Table 6.5 should be used for design purposes.

The true test of any model is in the verification phase. In January of 1989 severe flooding occurred in Ferryland (the result of high tides combined with strong onshore winds). Although the exact date of the event is unclear, the data indicates winds reached velocities of 65 km/hr and a speed of 50 km/hr or more was maintained for 13 hours previously on the 9th of January. Within the three peak hours, tides reached values from 0.8 to 1.1 m. From the measured water level (less tides), the average surge value was found to be 0.92 m, with a maximum value of 1.05 m (which is near the best fit for the two to five year return period). It is understood that this magnitude of flooding is fairly common as indicated by the low return period. When combined with high tides and high significant wave heights it would be possible to have an increase in the water level by as much as 6 m above low tide (or above the chart datum).

On April 11, 1986, 71 mm of rain fell within 11 hours. This, combined with onshore winds, caused heavy flooding in the area. Winds maintained a velocity over 30 km/hr for 13 hours with a peak wind of 48 km/hr, coming from 130°(southeast direction) for 30 hours. The associated maximum tide for that time period was 1.3 m, with a predicted maximum surge of 0.71 m. These values are consistent with an event having a return period of less that two years.

The data shows the 2, 5, 20, 50, and 100 year return periods with maximum and average values for wind speed and surge. These values were obtained for an event which has a minimum 12- hour duration, this was done in order for maximum setup to occur. It should be noted here that there are higher winds recorded than the distribution indicates. This is because the analysis only deals with onshore winds; the majority of the wind in this region comes from the southwest. Only onshore winds can produce storm surge and wave heights of the given magnitudes.

TABLE 6.3
Maximum Wind Readings, 30 years of data, St. John's Airport, (km/hr)
Threshold = 76.00 km/hr, Correlation = 0.9746321

Return Period	Wind Speed	90% U.L.	95% U.L.
2 year	86.71	89.90	90.54
5 year	96.77	102.15	103.23
20 year	109.83	119.00	120.84
50 year	118.11	129.82	132.16
100 year	124.31	137.95	140.68

TABLE 6.4
Average Wind Speed in a 12hr Sustained Event, St. John's Airport, (km/hr)
Threshold = 35.60 km/hr, Correlation = 0.9576114, 31 Maximum Events

Return Period	Wind Speed	90% U.L.	95% U.L.
2 year	46.53	48.04	48.35
5 year	51.30	53.85	54.36
20 year	57.48	61.83	62.70
50 year	61.40	66.95	68.06
100 year	64.34	70.80	72.10

TABLE 6.5
Maximum Surge, 30 years of data, St. John's Harbour
Threshold = 0.42m, Correlation = 0.9614879, 31 Maximum Events

Return Period	Surge Height (m)	90% U.L. (m)	95% U.L. (m)
2 year	0.69	0.73	0.74
5 year	0.82	0.89	0.90
20 year	0.98	1.09	1.12
50 year	1.08	1.23	1.26
100 year	1.16	1.33	1.36

TABLE 6.6
Maximum Surge in a Sustained 12hr Event, St. John's Harbour
Threshold = 0.55m, Correlation = 0.9409861, 31 Maximum Events

Return Period	Surge Height (m)	90% U.L. (m)	95% U.L. (m)
2 year	0.73	0.75	0.75
5 year	0.79	0.83	0.83
20 year	0.87	0.94	0.95
50 year	0.93	1.00	1.02
100 year	0.97	1.06	1.08

Note: Surge heights are in metres above mean sea level.

When we consider the worst possible case (i.e., a 1:100 year surge (1.55 m) combined with highest tide and the corresponding 1:100 year significant wave height), the water can reach significant heights, as Table 6.7 demonstrates. The worst case would correspond to an onshore wind with a sustained 12-hour speed of 72 km/hr (at 95% U.L.) with gust up to 140 km/hr. It is reasonable to assume that the tide and surge heights can be added directly because they are relatively steady state when compared to the wave duration. The greatest damage caused by waves is when runup occurs on beaches due to breaking. From observation, the areas which are most prone to flooding would be around the neck of the peninsula (see Figure 6.1). When dealing with such severe events, it is obvious that what little protection is now in place becomes painfully inadequate.

Table 6.7
Summary of Study Results

RETURN PERIOD -->	2 Year	5 Year	20 Year	50 Year	100 Year
Wind (km/hr) (Average)	48.35	54.36	62.70	68.06	72.10
Surge (m)	0.74	0.90	1.12	1.26	1.36
Tide (m) CHS datum	1.49	1.51	1.53	1.54	1.55
Tide (m) GSCD datum*	0.81	0.83	0.85	0.86	0.87
Waves (m)	3.20	3.20	3.29	4.83	4.99

* datum (±0.1 m) used to prepare topographic mapping of Ferryland

6.5 Flood Level Delineation

Table 6.7 provides a summary of the design water level prediction due to tide and storm surges. It also provides wave height estimates for various return periods (or risk levels).

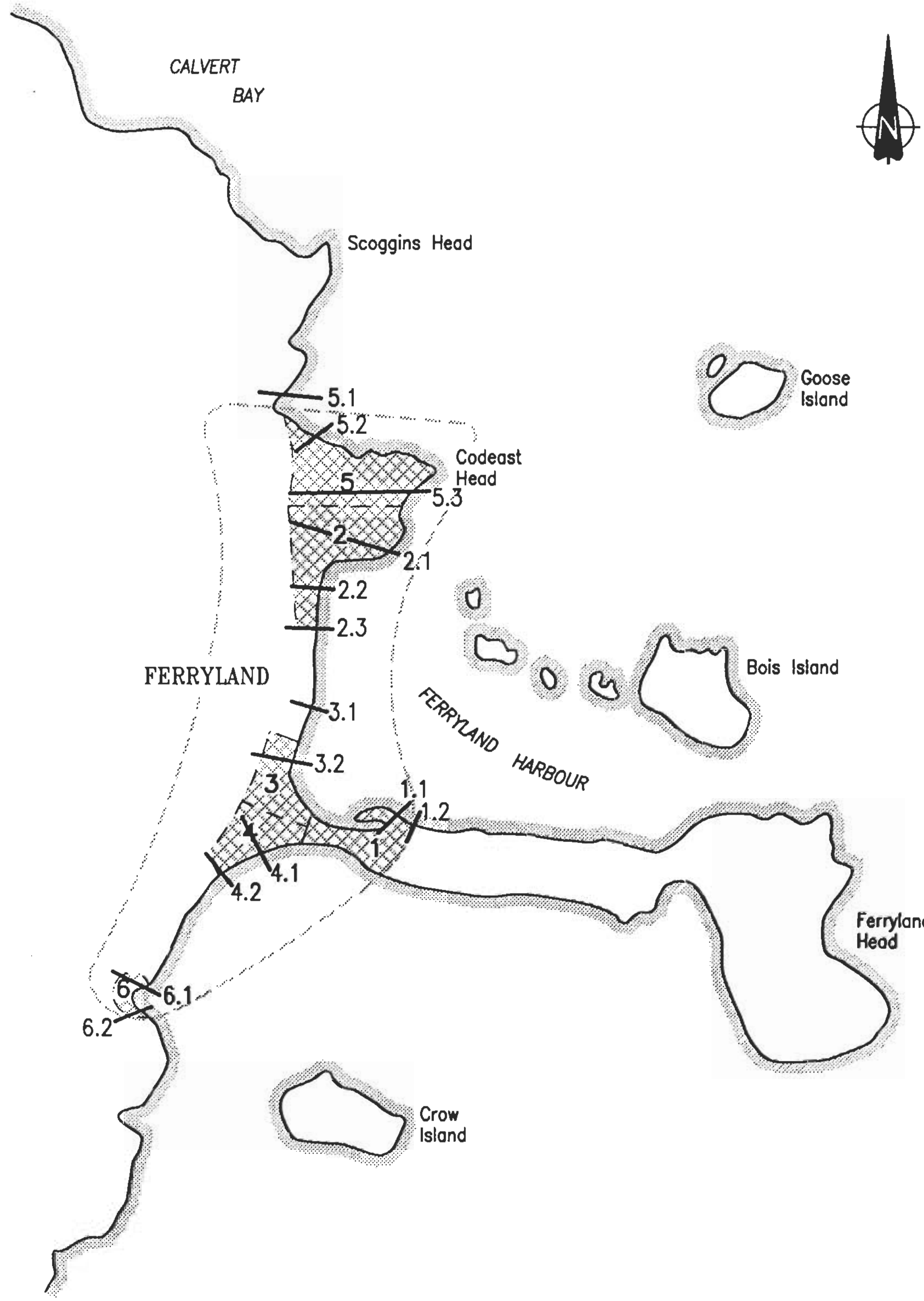
As discussed earlier, the storm surge and high tides can be combined to provide the worst case scenario (as the tides are periodical events that occur quite often). Therefore, the 1:100 year return period water level may reach 2.91 m above the chart datum (or 0.87 + 1.36 = 2.23 m GSCD). Similarly, the 1:20 year surge and tide level may reach 2.65 m above chart datum (1.97 GSCD). This only will result in flooding the high risk areas shown in Figure 6.1. In addition, large storm surge events will result from strong on-shore winds (blowing from off-shore) which also generate high waves at the site. Therefore, wave effects must be added to the above values. However, most of such high waves (3 to 5 m) will break before reaching the beach areas and most of the damage will occur due to wave run-up and breaking waves.

The elevation of wave run-up is affected by water depth, shoreline slopes, shoreline roughness, coastal structures, the characteristics of the wave, and other factors. The US Army Corps of Engineers (1984) has examined these various factors and their shore protection manual was used for determining the extent of wave run-up. Noteworthy is that local flooding and damage caused by spray, wave-tossed debris and erosion could not be considered at this level of study.

Figure 6-2 maps the locations where wave run-up was computed in the study area, and the following table summarizes the results of the analysis for 1:100 year conditions. Flood risk maps for the study area plot only the 1:100 year run-up level, as this level and the 1:20 year tide and surge level bracket the flood risk areas at Ferryland.

TABLE 6.8
Ferryland Flood and Wave Run-up Levels

Section No.	Wave Run-up Elevation (m)	1:100 year Surge, Tide and Wave Run-up Level (m GSCD)
5.1	1.12	3.35
5.2	4.90	7.13
5.3	1.35	3.58
2.1	0.50	2.73
2.2	3.40	5.63
2.3	3.10	5.33
3.1	4.5	6.73
3.2	3.4	5.63
1.1	0.5	2.73
1.2	3.25	5.48
4.1	0.5	2.73
4.2	0.5	2.73
6.1	2.8	5.03
6.2	3.4	5.63



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FLOOD RISK MAPPING STUDY OF
GOULDS, PETTY HARBOUR AND FERRYLAND

LEGEND

······	STUDY AREA BOUNDARY
— · — · —	ZONE BOUNDARY
3	ZONE NUMBER
——2.3	CROSS SECTION

FIGURE 6.2

FERRYLAND WAVE RUN–UP
CROSS SECTIONS

SCALE APPROX. 1 : 25000

Except for the steep slopes at Sections 5.2 and 3.1, the flood risk lies below elevation 6.0 m - an elevation which may serve as a general guideline, below which structures may be at risk from flooding.

7.0 FLOOD DAMAGE REDUCTION MEASURES

The final step in this study calls for preliminary identification of flood damage reduction alternatives which could be employed within the flood hazard areas identified earlier in Figures 5-1, 5-2, 5-3 (and in other locations). Noteworthy in considering these alternatives is that the flood-prone areas shown in the figures and attached floodplain mapping identify a "floodway" and a "flood fringe".

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

The floodway "fringe" is that portion of the flood risk area lying between the floodway and the outer limit of the area which is flooded on an average of once in 100 years. This zone generally receives less damage from flooding than the floodway because velocities and depths are less. Its limits are shown by the dashed line on the enclosed map sheets.

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

- floodplain regulations,
- acquisition, and
- flood proofing.

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

- flood control dams,
- channelization or dyking, and
- bridge opening expansions.

A benefit-cost analysis (which is beyond the scope of this project) is generally employed to give guidance in weighing all the possible alternatives. However, from experience on similar projects where damage-prone structures are often distributed along the full length of the river, the second group of alternatives often presents highly unattractive benefit-cost ratios. Hence, the focus of the following is on the first group of alternatives.

7.1 Flood Damage Prevention

A number of residential dwellings and commercial buildings currently fall within flood-prone areas of this study. Some of the river banks are now developed (particularly in Goulds), but there still remain flood-prone areas which appear to be considered as desirable river-side locations for future subdivisions.

7.1.1 Floodplain Regulations

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

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

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

7.1.2 Acquisition

Another alternative to reduce the potential for future damage in flood-prone areas is to acquire the undeveloped lands and properties and damage-prone structures. The former is often undertaken as part of subdivision agreements as it gives the municipality a desirable undeveloped corridor (or greenspace) through developed lands. The option of acquisition of all existing properties in the flood hazard zone does not appear immediately feasible because of cost and social disruption. It may be advantageous, however, to gradually acquire some of the most damage-prone structures as they come on the market or when they have sustained severe flood damages. Several of these are identified later in this Chapter.

7.1.3 Flood Proofing

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

- installation of permanent or temporary closures at low level openings in structures;
- raising structures on fill, columns or piers; and
- construction of floodwalls or low berms around structures.

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

The elevation of buildings or building additions above flood levels is suited for areas where permanent closure is difficult or impossible. As with permanent closure, no human intervention or flood warning is required to make the flood proofing effective.

Flood proofing also entails combinations of closure and/or elevation of certain structures, with berming around groups of other structures.

7.2 Recommended Options

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

1) It is recommended that the flood elevations advanced herein be adopted by the river and shoreline communities so that developable areas which are prone to flooding can be zoned in the near future for special flood risk restrictions and design consideration (e.g., elevation flood proofing on fill, extended and reinforced foundation walls or piles). Consideration should be given to gradual acquisition (by the municipality) of the most damage-prone buildings during this process.

2) It is recommended that the following should specifically be considered if it is desirable to provide physical flood protection for existing buildings along Raymond Brook.

Potential flood damages at Raymond Brook are limited to a small structure just west of the Southern Shore Highway, flooding of the road and a basement on the Lakeview Drive Extension (just west of the Avalon Raceway) and the Avalon Raceway property.

Flood damage reduction options which appear appropriate for Raymond Brook include:

- elevation flood proofing on fill of the outbuilding just west of the Southern Shore Highway (on the north bank of the river);

- a short berm parallel to the brook (~70 m long with top elevation of 70 m) at the end of the Lakeview Drive Extension. The objective would be to prevent overbank flows from flowing directly down the road and roadside ditches to the north;

- elevation flood proofing (~1.5 m fill) of the Avalon Raceway structures.

A more extensive berm surrounding Avalon Raceway might also be considered and was tested in the backwater model (Technical Appendix E - file RAYMDYKE). The dyking was found to have no effect on the upstream 20 or 1:100 year flood elevation and hence, would not increase flood damages elsewhere.

3) Potential flood damages at Cochrane Pond Brook include:

- Avalon Raceway barns at the mouth;
- a small structure at the northeast corner of Ryans Bridge (shallow flooding).

Recommended flood damage reduction can be completed northeast of Ryans Bridge by a shallow berm around the structure at that location or by elevation flood proofing on fill or piles (the flood depth here is shallow).

The Avalon Raceway barns may be flood proofed by fill (~1.5 m) or by a low berm surrounding the structures. In order to ensure safe access and egress during flooding, however, these berms must be connected to Race Track Road which must also be elevated to about 70 m.

The effect of this option was tested by modelling a full berm around the raceway. The results (Technical Appendix E) indicate that the full/partial berming raises water levels by about 5 cm at the horse barn area. There is no effect on 20 or 1:100 year flood levels further upstream.

4) Dirty Bridge Brook has been channelized between McGrath Place and Cleary Drive at the Southern Shore Highway. About 600 m west of the highway, the channelization turns to the north and then to the west to cross Back Line Road at its original culvert.

There is potential for flood damage in the western portion of the channel, including:

- the backlots of Sunset Street west of McConnell Place;
- much of lot 5 and two unnumbered lots along the channel north and south of the Hannaford Place crossing;
- the backlots of three or four homes north of Beaver Brook Drive near the channel bend.

It is recommended that the residents or prospective owners of these lands should be made aware of the hazard, and that floodplain regulations (e.g., zoning by-laws, subdivision regulations) be put into effect to reduce the risk associated with future development.

Lot 5 on Hannaford Place and the two unnamed lots (?7 and ?23) have significant portions in the floodway. Hence, it is recommended that these lots be designated as flood hazard zones and not be developed.

Alternatively, and only because the channel has been already altered from its natural state, the channel near Hannaford Place could be widened to improve its conveyance capacity.

5) Flood damage potential is relatively significant on the Doyles River because of proposed and existing development. There is concern related to:

- Riverside Drive and Eden Street trailer park developments (existing and proposed);
- proposed development north and east of Meadowbrook Drive (Bonnie Drive, Everard Avenue, Kieley Drive);

- the gasoline station at Petty Harbour Road and the Southern Shore Highway;

- an outbuilding and house in the flood fringe north of Petty Harbour Road.

It is strongly recommended that floodplain regulations be enacted to prevent future development in the floodplain at Riverside Drive and the southern end of proposed Everard and Kieley Drive. These locations lie directly within the flow path of waters which spill over the banks north of Eden Street and depths may exceed 2 m at some locations.

It is also recommended that the mobile homes in the area be elevated (preferably on fill) to safe levels.

The proposal for the development of Bonnie Drive, which appears to have been conceived without reference to existing flood mapping, must involve channelization of the river and significant infilling of the existing floodplain. Unless it can be demonstrated that flood and erosion conditions will not be altered by this development, it is recommended that the proposed subdivision be reconfigured to avoid the floodplain.

It is also recommended that a culvert be placed at proposed Gary Drive about 25 m north of the intersection of Meadowbrook Drive. This should allow for flows to follow existing paths without disrupting access/egress to and from Gary Drive during flood conditions.

The gasoline station at Petty Harbour Road is periodically flooded and projected to be so in the future for open water slows exceeding the 1:20 year flow. In that there are not residents (overnight) of this commercial establishment, the principal concern is the flammable contents of the underground storage tanks at the pumps. For this reason, it is recommended that a curb or post fence be located along the upstream edge of the current fill area to deflect/block debris or ice which could damage the pumps.

The structures in the flood fringe north of Petty Harbour Road (Section 2200) can be flood proofed by elevation without affecting upstream or downstream levels and flows.

6) At Petty Harbour, there are eight structures which are prone to flooding upstream of the bridge. Six of these structures lie within a relatively compact group which could benefit from flood proofing through a low berm around the group. Two others lie outside this group and may be flood proofed by elevation (e.g, fill or piles) or by low berms around each structure.

There are other buildings in Petty Harbour which are prone to flooding from local watercourses or from direct runoff down the hillsides. These situations are not part of this study of the Petty Harbour River floodplain, but can be addressed by local residents through judicious use of flood proofing and minor berming.

7) At Ferryland, there are a number of buildings and several flakes which are at risk along the southern shore of Codeast Point; and several structures, the archaeological dig and the provincial museum (near the ball diamond) at risk in the neck between the Downs and the mainland.

The buildings at Codeast Point will be difficult and expensive to protect by a shorewall or similar communal work. Flood-proofing by elevation on piles or gradual acquisition of the flood risk area following future flood damage/destruction are the most appropriate alternatives.

The flood-prone structures near the Downs should gradually be acquired by the Province or municipality following significant future flood damage, and immediate zoning by-laws should be enacted to limit future development in the area. The provincial museum lies within the 1:20 and 1:100 year flood risk area, as does a good portion of the archaeological dig. It is recommended that the artifacts from the dig be stored above flood levels (upper floor of the museum) and protected at the dig by covering during the off season.

8) In conclusion, there are damage reduction options which may be applied to reduce future and existing flood problems in the study area. Most may be carried out by the individual owners (e.g., flood proofing), but regulations to minimize future developing problems and to facilitate correction of existing ones requires the helping hands of the local government.

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Newfoundland and Labrador Department of Environment and Lands: Series 86005:
Photos 106 - 113 (from Second Pond west to south of Town of Goulds).
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CBC Film Footage:

March 1987 flooding event at Goulds was reviewed.

Drawings:

Cochrane Pond Diversion Dam - Plan, Sections and Elevations - 1986 - Scale 1:100 - Newfoundland Light and Power Co. Limited.

Bay Bulls Big Pond - Contour Map - 1974 - Scale 1":400' - St. John's Regional Water System, Department of Municipal Affairs and Housing, Government of Newfoundland and Labrador.

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Bay Bulls Big Pond Dam - Plan Existing Structure - 1992 - Scale 1:500 - Newfoundland Light and Power Co. Limited.

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Flow Data:

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WSC Station 02ZM001 Petty Harbour at Second Pond. October 1962 - December 1993.

WSC Station 02ZM022 Raymond Brook at Outlet of Bay Bulls Big Pond. January 1988 - August 1994.

HEC-2 Model:

River Flooding Study Town of Goulds - 1991 - Delcan Corporation.

Interviews With Local Residents:

Details previously submitted.

Meteorological Data:

AES Station 8402568 Logy Bay:

- Daily Total Rainfall (1969/12 - 1993/12)
- Daily Total Snowfall (1969/12 - 1993/12)
- Daily Total Precipitation (1969/12 - 1993/12)
- Daily Snow on Ground (1981/02 - 1993/12)
- Daily Maximum Temperature (1969/12 - 1993/12)
- Daily Minimum Temperature (1969/12 - 1993/12)
- Daily Mean Temperature (1969/12 - 1993/12)
- Monthly Snow on Ground Month End (1970 - 1993)

AES Station 8402925 Petty Harbour:

- Daily Total Rainfall (1955/10 - 1993/12)
- Daily Total Snowfall (1955/10 - 1993/12)

- Daily Total Precipitation (1955/10 - 1993/12)
- Daily Snow on Ground (1961/01 - 1993/12)

AES Station 8402956 Placentia:

- Hourly Dry Bulb Temperature (1970/11/01 - 1975/12/31)
- Hourly Precipitation (1970/11/01 - 1975/12/31)
- Daily 6-Hour Precipitation (1970/11 - 1975/12)
- Daily Total Rainfall (1970/11 - 1975/12)
- Daily Total Snowfall (1970/11- 1975/12)
- Daily Total Precipitation (1970/11 - 1975/12)
- Daily Snow on Ground (1970/11 - 1975/12)
- Daily Maximum Temperature (1970/11 - 1975/12)
- Daily Minimum Temperature (1970/11 - 1975/12)
- Daily Mean Temperature (1970/11 - 1975/12)
- Monthly Snow on Ground Month End (1970 - 1975)

AES Station 8403500 St. John's:

- Daily Total Rainfall (1874/01 - 1956/03)
- Daily Total Snowfall (1874/01 - 1956/03)
- Daily Total Precipitation (1874/01 - 1956/03)
- Daily Maximum Temperature (1950/01 - 1956/03)
- Daily Minimum Temperature (1950/01 - 1956/03)
- Daily Mean Temperature (1950/01 - 1956/03)
- Monthly Snow on Ground Month End (1956 - 1956)

AES Station 8403506 St. John's Airport:

- Hourly Dry Bulb Temperature (1949/01/01 - 1993/12/31)
- Hourly Precipitation (1961/06/01 - 1992/11/30)
- 6-Hour Precipitation (1947/01 - 1993/12)
- Daily Total Rainfall (1942/01 - 1993/12)
- Daily Total Snowfall (1942/01 - 1993/12)
- Daily Total Precipitation (1942/01 - 1993/12)
- Daily Snow on Ground (1955/11 - 1993/12)
- Daily Pan Evaporation (1971/05 - 1979/10)
- Daily Lake Evaporation (1971/05 - 1979/10)

- ▸Daily Maximum Temperature (1950/01 - 1993/12)
- ▸Daily Minimum Temperature (1950/01 - 1993/12)
- ▸Daily Mean Temperature (1950/01 - 1993/12)
- ▸Monthly Snow on Ground Month End (1947 - 1993)

AES Station 8403600 St. John's West CDA:

- ▸Daily Total Rainfall (1950/11 - 1993/12)
- ▸Daily Total Snowfall (1950/11 - 1993/12)
- ▸Daily Total Precipitation (1950/11 - 1993/12)
- ▸Daily Snow on Ground (1960/10 - 1993/12)
- ▸Daily Pan Evaporation (1980/05 - 1993/10)
- ▸Daily Lake Evaporation (1980/05 - 1993/10)
- ▸Daily Maximum Temperature (1950/11 - 1993/12)
- ▸Daily Minimum Temperature (1950/11 - 1993/12)
- ▸Daily Mean Temperature (1950/11 - 1993/12)
- ▸Monthly Snow on Ground Month End (1950 - 1993)

<u>Newspaper Articles (St. John's Evening Telegram):</u>

Articles documenting the following were reviewed: Extreme rainfall depth, extreme snowfall depth, extreme low and high temperatures, extreme winds, extreme tides, flooding damage, flooding evacuations and flood damage costs, ice jams and blasting, heavy storms, and meteorological summaries of weather extremes.

<u>Topographic and Hydraulic Data Mapping:</u>

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