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and Labrador

Department of Environment
Water Resources Division
St. John's, Newfoundland

Government
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Environment Canada
Inland Waters Directorate
Dartmouth, Nova Scotia

National Water Research Institute
Burlington, Ontario

FLOOD STUDY
VOL. 1

Urban Hydrology Study of the Waterford River Basin

TECHNICAL REPORT No.
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<u>WATERFORD RIVER BASIN FLOOD STUDY</u>

by

WATER PLANNING AND MANAGEMENT BRANCH
INLAND WATERS DIRECTORATE
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and

WATER RESOURCES DIVISION
NEWFOUNDLAND DEPARTMENT OF THE ENVIRONMENT

VOLUME 1 OF 2 - MAIN REPORT

Prepared for the
Waterford River Basin Urban Hydrology Study

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Your file Votre référence

Our file Notre référence

Dr. Wasi Ullah, Chairman
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Waterford River Basin Urban Hydrology Study
Newfoundland Department of the Environment
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St. John's, Newfoundland
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Dear Dr. Ullah:

On behalf of those investigating flooding along the Waterford River, I am pleased to submit herewith the final report entitled "Waterford River Basin Flood Study", as our contribution to the Waterford River Basin Urban Hydrology Study.

Yours truly,

E.R. Langley
Flood Studies Engineer
Water Planning & Management Branch

ABSTRACT

The Governments of Canada and the Province of Newfoundland had agreed to undertake a five-year urban hydrology study of the Waterford River on a work shared basis starting April 1, 1980. This component of the overall study determined the 1:20 and 1:100 year return period open water flood profiles using the HEC-II hydraulic model and identified flood prone areas along the Waterford River. The design flows used as input to this study were determined using the HYMO hydrologic model in another component of the Waterford River Study and the subject of a separate report. The flood prone areas are shown on the maps inserted at the end of the report.

RÉSUMÉ

Le gouvernement fédéral de même que la province de Terre-Neuve avaient décidé d'entreprendre une étude d'hydrologie urbaine de cinq ans pour la rivère Waterford à partir d'une entente de partage du travail débutant le 1er avril 1980. Cette composante de l'étude globale à déterminée les profiles des crues d'eau libre de 20 ans et de 100 ans utilisant le modèle hydraulique HEC-2, de même que l'identification des zones propices aux inondations le long de la rivière Waterford. Les débits utilisés pour cette étude furent déterminés précédemment par le modèle hydrologique HYMO, qui est une autre composante de l'étude de la rivière Waterford et est sujet d'un rapport séparé. Les zones propices aux inondations sont indiquées sur les cartes insérées à la fin du rapport.

PREFACE

The Waterford River Basin Urban Hydrology Study, developed as a co-operative effort between the Governments of Canada and the Province of Newfoundland, was proposed by the Newfoundland Department of Environment in response to watershed management problems that had resulted from urbanization of the Waterford River Basin. Among such problems, negative effects of urbanization on both water quality and quantity were found to be so serious that the Newfoundland Department of Environment identified the Waterford River Basin as a high priority area.

The five-year study began in 1980 was completed in March, 1985. The primary objectives of the study were to develop environmentally acceptable criteria for urban development in Newfoundland and to utilize the study results directly in the urban planning process in the Province. The specific objectives of the study, as outlined in the report "Waterford River Basin - Urban Hydrology Study Plan" were as follows:

(1) To examine the processes leading to changes in the hydrologic regime of the Waterford River watershed. This should include evaluation and monitoring of major hydrologic changes caused by urbanization, the study of precipitation-runoff processes, and the study of various forms of pollution originating in the urban areas of the watershed.

(2) To provide a hierarchy of mathematical models describing hydrologic processes in the watershed. Such models should deal with both water quality and quantity, and should be capable of simulating the impact of urbanization on the water resources in the studied basin.

(3) To recommend solutions to specific water management problems in the studied basin and to develop guidelines for implementation of similar solutions elsewhere in Newfoundland. Furthermore, planning and management criteria should be developed for those aspects of the urban development which related to the environmental protection of the affected water resources.

The complexity of the study called for a comprehensive approach which included hydrometric surveys, hydrological modelling, groundwater studies, biological surveys, water quality assessment, investigations of flooding, and land use and socio-economic analyses.

The study was administered by a Steering Committee appointed by the governments of Canada and Newfoundland. To implement the study plan, a Technical Committee, consisting of representatives of each participating agency, was established. Subsequently, the Technical Committee appointed sub-committees and working groups to prepare and carry out the workplans for the various components of the study.

The report that follows deals with one such component.

ACKNOWLEDGEMENTS

This report would not have been possible if it were not for the contributions of the following individuals:

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WRD - Water Resources Division, Environment Newfoundland

IWD - Inland Waters Directorate, Environment Canada

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1.0 INTRODUCTION

1.1 General

The growth of cities and towns, which is often rapid and largely unpredictible in nature, can cause a great variety of problems. The severity of these can usually be diminished by effective planning and control. One of these problems is increased rates of runoff which need to pass through the urban drainage system without incurring loss of life or significant damage to property, disruption of businesses, and inconvenience due to flooding.

Urban storm drainage systems are usually a combination of natural or altered stream channels, storage areas, man-made conveyances and storage areas such as ditching, pipes and reservoirs. Dyking is another technique which has been employed in order to contain or redirect streamflow. These systems usually follow the natural drainage patterns, at least when first constructed. However, after further urbanization due to constraints in the capacity of existing systems and increased runoff rates, man-made diversions , employing pumped or gravity flow, are sometimes constructed either to by-pass the constricting component of the system, or to transfer runoff to another drainage system.

Flooding occurs when the capacity of the drainage system to convey or store water is insufficient, and the surcharge of flow spills out onto the flood plain. Whether or not loss of life or significant damage to property, disruption of business, or inconvenience accompanies flooding, depends on the use to which floodplain lands have been put, as well as the severity of the storm, and the amount of advance warning which can be provided.

Severe flood damages in urban areas are largely the result of a lack of appreciation of the frequency of occurrence and magnitude of flooding, as well as inadequate planning or control by developers and government agencies. Some flooding damages are caused by inadequate design of drainage systems, possibly resulting from poor projections of future development, a lack of hydrologic data on which to base the design, or errors in hydrologic or hydraulic computations.

The governments of Canada and Newfoundland had agreed to undertake a five year urban hydrology study in the Waterford River Basin, which is located in the Avalon Peninsula in Newfoundland, on a work shared basis starting April 1, 1980. This study was designed to address existing and potential water resources problems such as increased flow rates and incidents of flooding, impacts on groundwater resources, and deterioration of surface water quality. The report, "Waterford River Urban Basin Hydrology Study Plan" (7), identified urban development as the cause of the impact on the water resources. Since urban development in the basin was expected to substantially expand in the 1980's due to the increased economic activities, these problems were anticipated to become much more serious and widespread.

This study examines the flooding component of the Waterford River Urban Hydrology Study. Subsequent to the findings of this investigation, further work may be undertaken towards the designation of flood risk zones whereby the Federal and Provincial Governments agree to restrict their funding of new flood vulnerable development, and promote wise use of floodplain lands.

The primary purpose of this study was to determine 1:20 and 1:100 year return period open water flood profile and identify flood prone areas along the Waterford River. The general location of the study area is shown on Figure 1.

1.2 Scope of Study

The following points summarize the overall scope of the hydraulic investigations:

1. Review of background information that characterizes the flooding problem.
2. Identify the significant flood prone areas along the Waterford River.

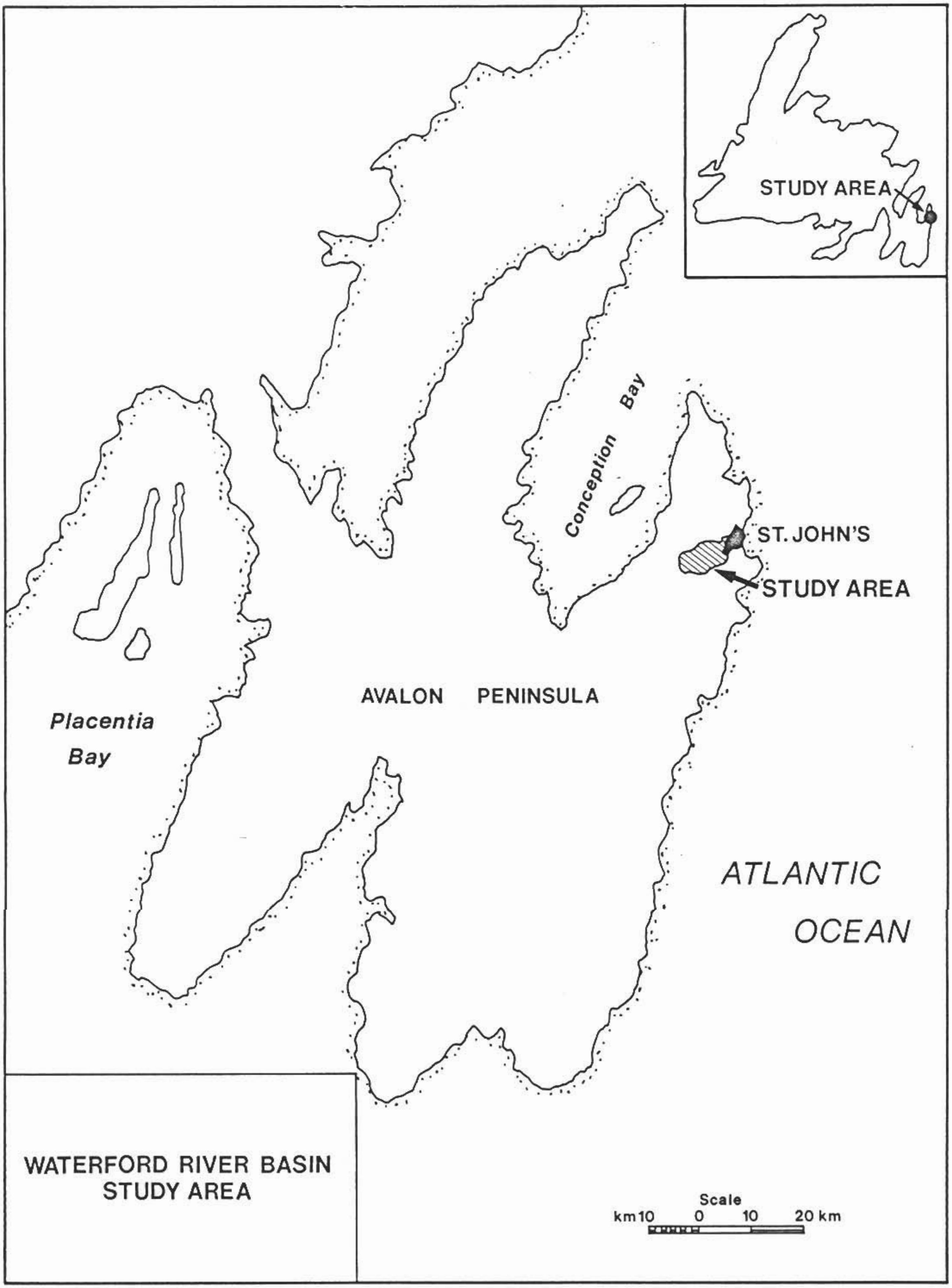
STUDY AREA
Conception Bay
ST. JOHN'S
STUDY AREA
Placentia Bay
AVALON PENINSULA
ATLANTIC OCEAN
WATERFORD RIVER BASIN
STUDY AREA
Scale
km 10 0 10 20 km

Figure 1

3. Evaluate the significance of various factors affecting flooding in the Waterford River.
4. Design, co-ordinate and manage a field program for the purpose of collecting hydraulic data for model calibration and validation.
5. Determine 1:20 and 1:100 year recurrence interval, open water flood profiles.
6. Undertake sensitivity analysis of backwater profiles.
7. Produce the 1:20 and 1:100 year flood profiles and plot the 1:20 and 1:100 year return period flood lines on existing topographic maps to determine the areal extent of flood prone areas.
8. Identify possible remedial measures for flood management and flood damage reduction to be analysed in possible future flood hazard investigations.

1.3 Overview of Study Methodology

The main items under consideration in the determination of the 1:20 and 1:100 year return period open water flood profiles, that is the levels which water has a 5 and 1 percent chance of reaching each year, and the subsequent identification of flood risk areas are summarized as follows:

1. Review: This consisted of a review of available information including the existing data, previous reports and a site reconnaissance.
2. Field Program: A field program was designed to collect the relevant hydraulic and physical data required to develop a hydraulic computer model. This included the collection of representative channel cross-sections and the measurement of water surface profiles and flows at selected points for model calibration/verification.
3. Hydrology: The designed flows used for this study were derived using HYMO model. This work is described in "Waterford River Basin Urban Hydrology Study, Watershed Modelling - HYMO" (5), another portion of the "Waterford River Basin Urban Hydrology" Study.

4. Hydraulics: This portion of the study consisted of assembling and calibrating a mathematical model using information generated in Steps 1 and 2. In so doing, complex interrelationships between several hydrologic and hydraulic factors were considered. These factors included: (a) historical flood conditions; (b) peak discharge rates; (c) effects of ice/debris jams; (d) existing stream channels and flood plain hydraulic characteristics; (e) man-made changes such as bridge and channel constrictions, dyking, etc.; and (f) natural and artificial flood storage.

   A sensitivity analysis was undertaken to determine the error associated with the use of the model. The calibrated/verified model then used the design floods developed in Step 3 to develop the required flood profile.
5. Floodline Plotting: Using the cross-sectional information and the derived flood profiles, the areal extent of flooding was plotted on existing topographical maps.
6. Remedial Measures: With reference to the flood risk areas delineated on the maps, potential remedial measures were identified. These remedial measures might be considered in more detail for alleviating future flood losses. No attempt was made to recommend a particular remedial measure.

The "Hydrologic and Hydraulic Procedures for Floodplain Delineation" (3) were used as basic guidelines throughout the course of these investigations.

2.0 BACKGROUND INFORMATION

2.1 General

The Waterford River is located in the Avalon Peninsula on the east coast of Newfoundland with its outlet in St. John's Harbour. The study area is shown in Figure 2. The main channel originates above Bremigens Pond at an elevation of about 168 metres, and flows north-easterly over a distance of about 14 kilometres. South Brook, which originates from a swampy area in the upper region, is a major tributary

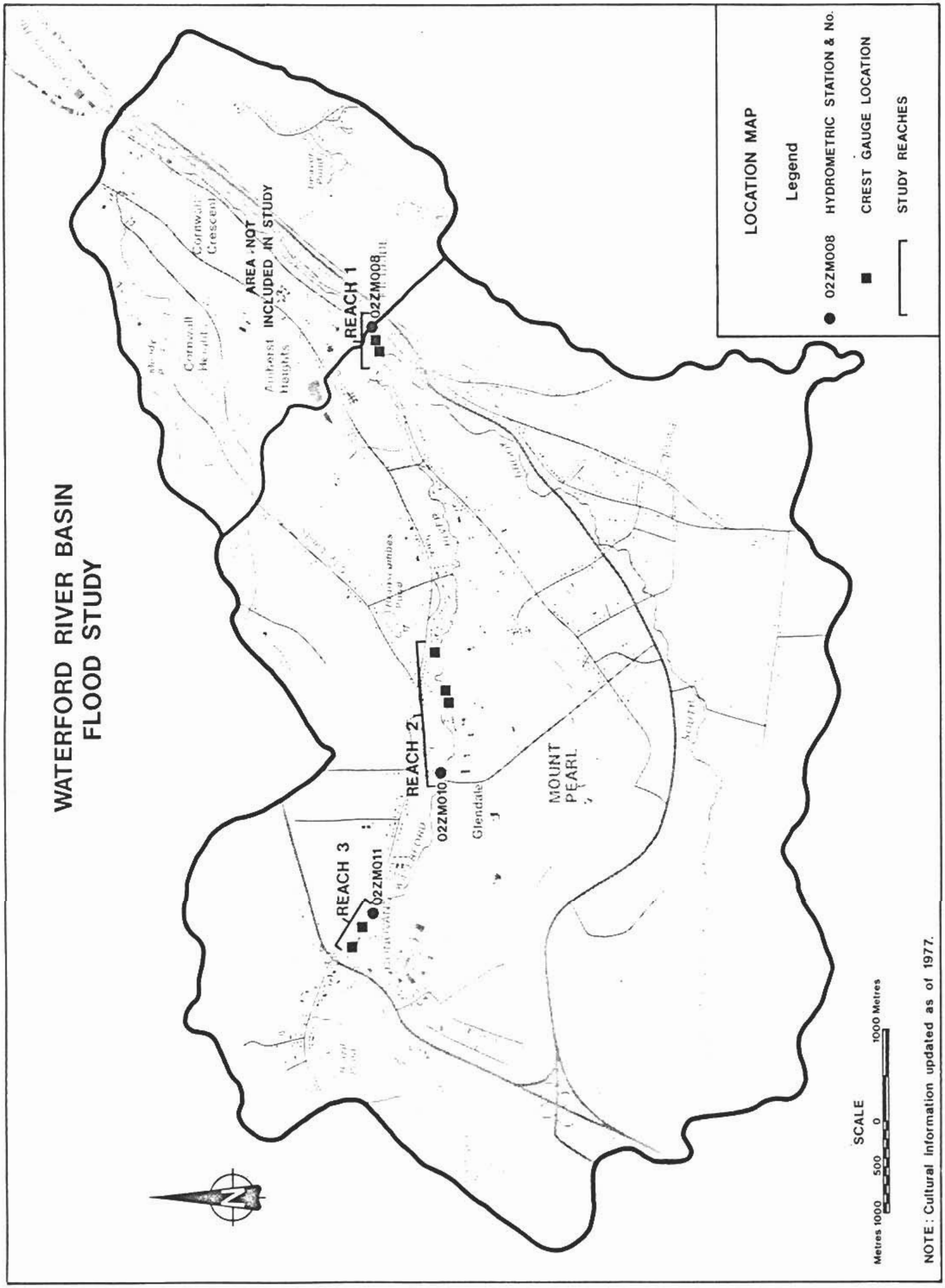
WATERFORD RIVER BASIN
FLOOD STUDY
LOCATION MAP
Legend
02ZM008 HYDROMETRIC STATION & No.
CREST GAUGE LOCATION
STUDY REACHES
AREA NOT
INCLUDED IN STUDY
Cornwall Crescent
Cornwall Heights
Amherst Heights
REACH 1
02ZM008
REACH 2
02ZM010
REACH 3
02ZM011
Glendale
MOUNT PEARL
SCALE
Metres 1000 500 0 1000 Metres
NOTE: Cultural information updated as of 1977.

Figure 2

of the Waterford River. Smaller tributaries, some of which have intermittent flows, also flow into the Waterford River and South Brook. The portion of the Waterford River Basin which falls within the scope of the overall "Waterford River Basin Urban Hydrology Study" corresponds to the watershed area to the hydrometric station at Kilbride.

Most of the area is covered with materials of glacial origin which may vary in depth from about 1 to 5 metres. The soils are coarse textured and contain stones, gravel, and boulders because there has been little breaking down of parent materials in the formation of the soil profile. There is also a tight till layer with clay and silt constitutants.

The modifying influence of the sea results in the absence of extreme temperatures, generally causing the winters to be mild and the summers to be cool. The maximum flow can occur at any time of year. The minimum flow normally occurs in June or July, but winter flows can occasionally be as low as the summer flows after a persistent spell of cold weather.

The land uses of the area include forest, agriculture, urban, recreation and ponds, bogs, barren river channels and gravel pits, etc. The area under forests is still more than 50% despite the urban development taking place. Because of unfavourable soil conditions, trees are small. Relatively small areas of land are suitable for agricultural development and there are many individually owned farms in the study area. The principle farming activities consist of dairy and conventional farming - especially root crops. There is indication that urban developments in the area will generally extend to these agricultural lands. The urban areas include residential, commercial, institutional and industrial areas, roads, transportation lines, etc. Recreation areas include Bowring Park and a few small municipal parks in Mount Pearl.

The study area includes a small portion of St. John's, Mount Pearl, Newtown, and a portion of Paradise, Kilbride and Goulds.

2.2 Summary of Historical Flooding

No severe flooding involving extensive damage to property or loss of life has been experienced in the area, but certain segments of the Waterford River have problems with frequent localized flooding. Some of the flooding incidents are as follows:

- In 1966, homes were flooded on both sides of the river near Wilson Avenue. A footpath bridge was washed out near Queen Mary of the World School. Homes, roads and rail lines were flooded in this area.
- The Waterford River overflows its banks almost every year (sometimes more than once) and floods the road near Newfoundland Hardwoods plant.
- In 1962, the concrete spillway of Bremigans Pond was washed out due to heavy rain which caused flooding in the Donovans area. Newfoundland Hardwoods boiler room had over one metre of water over the floor, and homes and the CNR track downstream in Mount Pearl were flooded.
- In 1951, a retaining wall protecting the CNR track was damaged by flood water.
- There were several other incidents of flood damages caused to CNR track and culverts in the Basin (1970-1981).

Additional information is presented in Table 1.

TABLE 1

SUMMARY OF HISTORICAL FLOODING ON THE WATERFORD RIVER*

Date	Cause	Description/Location
October 1934	Heavy Rain (3.69 inches in 3 days)	Southside Hills was flooded. Several families near the intersection of Southside Road and Blackhead Road left their homes. Train movements were held up for several hours due to material deposited on the railway tracks near Browning's Bridge.
December 1944	Heavy Rainfall	The Waterford River was very high. Extensive damages were reported.
December 1946	Heavy rainstorm	At Bowring Park, a 30 metre section of concrete retaining wall was torn down. Water was over the road at Brookfield Bridge. Syme's Bridge as well as another near Waterford Hall was closed to traffic. Fields near Road Deluxe were inundated. At Waterford Bridge, water swept over the road and rose to a height of about two feet near the Parish Church. Tons of gravel and silt were washed down from Southside Hills. At Victoria Park, water from Mundy Pond flooded the area.
September 1948	Torrential rain	Three houses on the Southside collapsed when hit by mud and gravel slides. In an area near Water Street West, which was flooded by overflow from the Waterford River, it was necessary to use boats to rescue residents from their homes.
April 1951	Heavy rainfall	The Waterford River was very high. Mundy Pond had risen high enough to flood the ground floor of a house on Pierce Avenue Place.
November 1951	Heavy rains	The Waterford River overflowed the St. John's Bridge and several houses near the beginning of Gould's Road were flooded.

TABLE 1 (Cont'd)

Date	Cause	Description/Location
December 1953	Heavy rainfall; melting snow	Along the Waterford River, a foot bridge at Steady Waters was partially destroyed. Near Mill Bridge, water swirled around several homes, damaging furniture. Mundy Pond Brook overflowed, flooding Victoria Park.
February 1955	Rain	Several people living at Mill Bridge were forced to leave their homes because of the flood waters. The Waterford River overflowed its banks.
November 1962	Heavy rain and melting snow	Flooding occurred due to blocked culverts along Bennett's Brook which flows from Mundy Pond to the harbour.
January 1963	Rain, melting snow and ice jams	An ice jam at Topsail Road Bridge tore the structure away from its abutments. Marshy land, just east of Topsail Road, was under an estimated five feet of water. Dunn's Bridge was reported to be unsafe and impassable because of the floodwaters. The Steady Waters pedestrian bridge was destroyed and the bridge on Commonwealth Avenue was damaged. Newfoundland Fiberply and Newfoundland Hardwoods plants were forced to close.
August 1964	Heavy rainfall	Flooding occurred along the banks of the Waterford River and in Bowring Park.
December 1966	Heavy rain	In the Blackhead area, a section of highway was washed out. Kenmount Road was washed out at one place. The Waterford River overflowed its banks and flooded basements in Mount Pearl.
February-March 1970	Rain (4 days)	At Littledale, the Waterford River overflowed its banks and flooded a classroom.

TABLE 1 (Cont'd)

Date	Cause	Description/Location
January and February 1971	Mild weather and rain	Waterford River overflowed its banks in numerous places. Floodwaters surrounded the portable classrooms at Littledale.
January 1973	Mild temperatures and rain	Gravel and grading were required on several roads in the Mundy Pond area.
February 1973	Heavy rainfall	Mundy Pond Road and Southside Road experienced flooding but no damage was reported.
August 1974	Rain and blocked	In Mount Pearl, several basements were flooded, storm sewers in the Donovan's Street area because of blocked storm sewers.
January 1976	Heavy rainfall	Debris blocked the outlet from Mundy Pond. The rush of water when cleared flooded the reservoir at the bottom of Victoria Park.
December 1976	Heavy rain	Southside Hills had a problem with blocked storm sewers. The intersection of Job's Bridge with Southside Road was inundated.
December 1977	Rain, high temperatures and melting snow	The Waterford River overflowed its banks, inundating sections of Bowring Park and Kinsman Park on Squires Avenue.
December 1978	Heavy rain and high winds, melting snow	Southside Road was closed to traffic.
January 1979	Heavy rain, temperatures and ice jams	Waterford River was high.
November 1980	Wind and rainstorm	Bowring Park was flooded.

TABLE 1 (Cont'd)

Date	Cause	Description/Location
October 1981	Heavy rain	A state of emergency was declared in the city of St. John's. The area of Southside Road was particularly hard hit by flooding and part of the road was blocked when rock and silt fell from the Southside Hills onto the road. Over 121 millimetres of rain fell over a two and one-half day period. The Harbour Arterial Road had to be closed for awhile during the weekend. Emergency Measures Organization (EMO) was contacted for assistance. Other areas of the city which were hard hit by the rain and flooding include Mundy Pond, Paird Place, Black Marsh Road and Topsail Road.

Source: Reference 2

*Many of these events did not affect the selected study reaches.

3.0 HYDROLOGIC ANALYSIS

As a part of the overall "Waterford River Basin Urban Hydrology Study", a separate investigation (5) was undertaken to identify the past and possible future effects of urbanization on storm water runoff along the Waterford River using the HYMO hydrologic model. The 1:20 and 1:100 year recurrence interval flood flows were determined during that investigation. These flood flows, presented in Section 4, were used as the design flood flows in the determination of the surface water profile described in Section 4.6.

Two of the conclusions of the above mentioned investigation were that the small amount of urbanization between 1973 and 1981 does not appear to have had a significant effect on streamflows for the Waterford River, and that significant increases in streamflows as a result of future urbanization are unlikely unless a much higher level of development involving high density residential and/or industrial complexes occurs. Under that extreme development scenario, little increases in the volume of storm runoff would be anticipated; however, potential paving and smoothing of land surfaces and the construction of storm drainage facilities may change the time of concentration and hence peak flows.

For the purposes of this study, the design flows are based on existing conditions.

4.0 HYDRAULIC ANALYSIS

4.1 General

The purpose of the hydraulic investigation is to provide reliable estimates of the 1:20 and 1:100 year return period open water flood profiles along the study reaches of the Waterford River. This was undertaken by utilizing a mathematical model to simulate water surface profiles for the corresponding instantaneous peak discharges of the design flood hydrograph. The flood water profiles are a function of the flow in each reach. Steady, gradually varied one dimensional flow conditions are assumed and river channels are assumed to have "small" slopes, say less than 1:10. The Donovan's, Mount Pearl and Kilbride reaches have average slopes of 0.0044, 0.0022 and 0.0052 respectively.

Computation of the steady state profile for each study reach can be achieved by applying the Bernoulli equation at each of a number of cross-sections along a reach in the following form (1):

$$\frac{dh}{dx} = \frac{(S_o - S_f)}{(1 - \frac{v^2}{gh})} \qquad 1$$

where
- h = Depth of flow
- x = Distance in the direction of flow
- S_o = Bottom slope
- S_f = Boundary frictional effect
- v = Velocity in direction of flow
- g = Acceleration due to gravity

Energy losses due to friction are computed and applied between cross-sections for each associated reach using (1):

$$S_f = \frac{nv}{R^{2/3}}^{2} \qquad 2$$

where
- n = Manning's roughness coefficient
- v = Velocity (m^3/s)
- R = Hydraulic radius (m)

In order to simulate the flood levels associated with each of the 1:20 and 1:100 year peak flood flows, a hydraulic computer model based on a standard computer program was developed for the Waterford River. The program selected for application in the project is discussed in Section 4.2.

The model was developed to simulate the existing hydraulic characteristics of the watercourse based on the channel and floodplain conditions as interpreted from the existing topographic mapping and the results of comprehensive field topographic and reconnaissance surveys. Subsequent to the model calibration and verification, sensitivity testing was carried out by varying model parameters. This included variations in the elevation of the bottom of the channel, peak flow rates, downstream water levels, Manning's 'n', and contraction and expansion coefficients. The backwater model was then utilized to establish the flood profiles associated with the peak 1:20 and 1:100 year discharge rates.

4.2 Model Selection

As in most studies of this kind, there are no observed flood profiles for all or any of the streamflows with magnitudes equal to the 1:20 and 1:100 year flows. For this reason, a mathematical model is often employed. The model, however, must be calibrated to simulate, with a reasonable degree of accuracy, profiles measured under a range of streamflow conditions. Once the calibration has been performed, it is accepted practice to verify the model by comparing the profiles simulated for a number of surveyed events which were not employed in the calibration.

There is no tidal influence and little channel storage relative to runoff volume, therefore, this study makes the assumption that it is reasonable to apply a steady-state model to the simulation of flood profiles in the study reaches.

The HEC-2 computer program (8) was selected to simulate the 1:20 and 1:100 year flood profiles. This program has been successfully used in many similar practical applications. It requires input of channel and floodplain cross-sections and associated hydraulic parameters (channel and floodplain roughness coefficients as well as channel and floodplain expansion and contraction coefficients) at locations where there is an appreciable change in cross-sectional area, roughness, slope or direction, and also at all culverts or bridges.

The HEC-2 program can be applied to evaluate the effects of hydraulic improvements and proposed channelization along the study reaches, etc. In addition, the program calculates the critical depth at each cross-section and can calculate profiles for supercritical flow, where required. Backwater profiles can be run for subcritical flow conditions by specifying the water level at the downstream end of the stream reach being simulated. For supercritical flow conditions, flood profiles can be computed by starting the computation at a known water level at the upstream end of the study reach. Some of the features of the steady state HEC-2 program that makes it particularly useful for this study is that it takes into account channel roughness, floodplain roughness, islands or flow divisions, bends in the stream or flood plain, crosssectional area of the stream channel and floodplain, slope of the channel and flood plain, energy losses at hydraulic structures (including bridges, culverts, weirs, dams, etc.), channel and floodplain expansion and contraction losses, variation in discharge along the study reach (i.e. due to tributary inflows), and if necessary, the effect of ice cover on the stream of flood plain. A major advantage of the HEC-2 model is that the channel and floodplain roughness (Manning's 'n') can be varied for each cross-section in the model. *This allows a description of the various* factors on which the roughness coefficient depends such as channel morphology, type and extent of vegetation, etc. For a more detailed description of the HEC-2 program, reference is given to the HEC-2 water program user's manual (8).

4.3 Development of Hydraulic Model

4.3.1 Channel and Floodplain Characteristics

In order to assess the hydraulic condition of the Waterford River and its flood plain, a review of the channel and floodplain characteristics of the Waterford River was undertaken. This included a detailed examination of the study area during field reconnaissance surveys and the interpretation of available mapping and background information such as reports of flooding and newspaper articles. As a result, three reaches were selected for investigation - Kilbride, Mount Pearl and Donovans.

As there are a number of waterfalls and rapids along the river's length, it was postulated that backwater from one reach would not significantly affect upstream reaches separated by such controls. Consequently, it was decided to identify the areas most prone to flooding and model these separately rather than studying the entire river. Studying the entire river length of the overall study area would have greatly increased the cost without necesarily showing additional significant flood plain areas.

The flooding problems in the two intermediate reaches that were not considered were thought to be insignificant. In the reach between Kilbride and Mount Pearl, much of the channel is steep with rapids; hence, it was unlikely flooding would occur there. Within the flatter area of this reach, Bowring Park has been flooded in the past, but there is no pressure for development there. The reach between Mount Pearl and Donovans contains a well defined channel bordered by a high railway embankment on one side which would act as a dyke if flooding should occur. On the other side, the channel slopes quickly to the river. There have been no historical reports of flooding in this area. In addition, no flooding problems were identified for South Brook, the major tributary. The three reaches studied are described below (see Figure 2):

(a) Donovans: This reach starts at the Water Survey of Canada hydrometric gauge site (Waterford River near Donovans, 02ZM011 - now abandoned) on the abandoned bridge abutment and continues upstream to the merging of two tributaries above the Newfoundland Hardwoods Ltd. buildings. Two bridges are encountered in this reach. The downstream bridge is a concrete box culvert (see details on Figure 3) owned by Fibrply Ltd., installed below the present channel

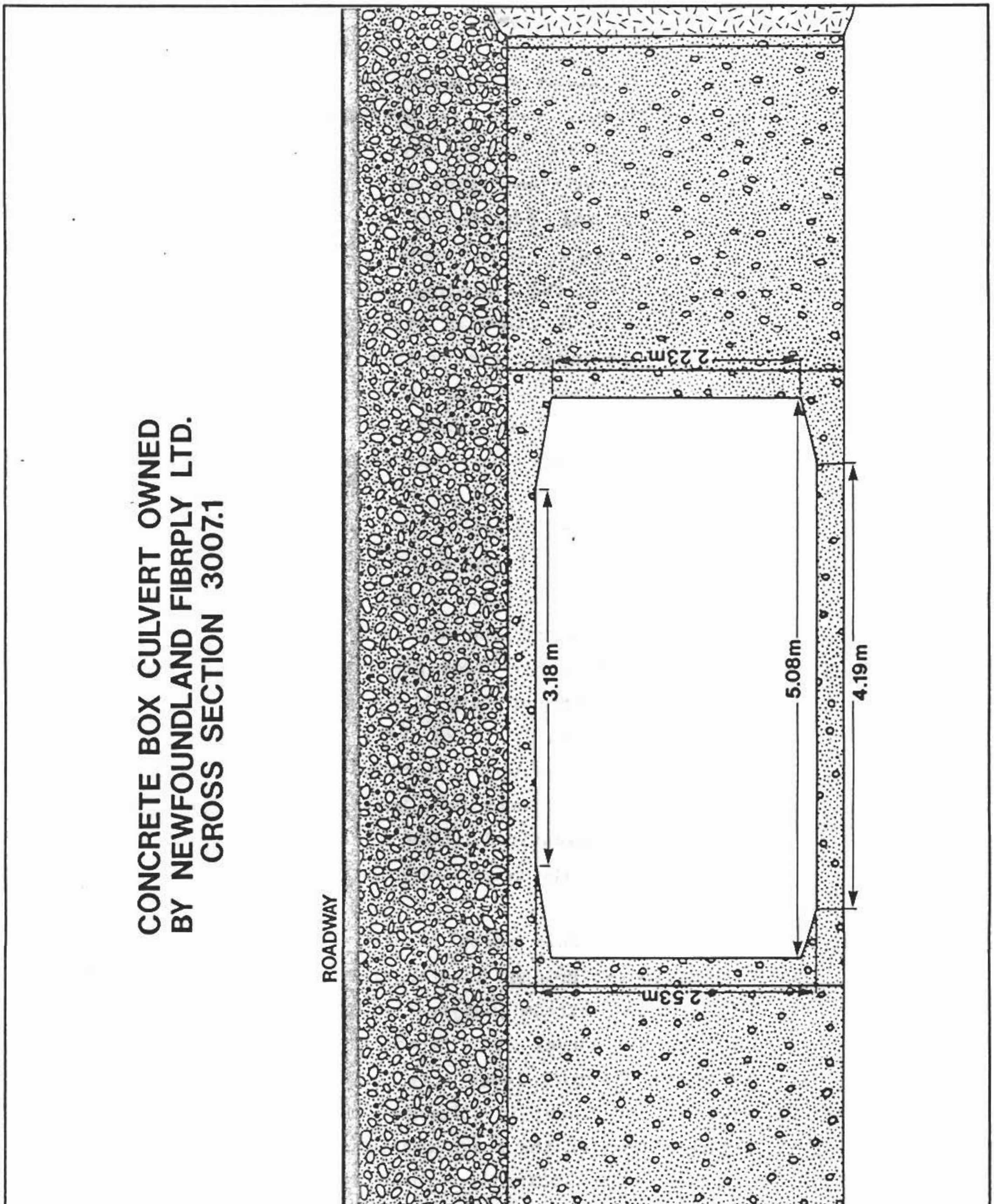
CONCRETE BOX CULVERT OWNED
BY NEWFOUNDLAND FIBRPLY LTD.
CROSS SECTION 3007.1
ROADWAY
2.23m
2.53m
3.18 m
5.08m
4.19m

Figure 3

invert in a causeway which spans the flood plain. The upstream bridge, owned by Newfoundland Hardwoods Ltd., contains an arched culvert with a stop-log control on the inlet (see details on Figure 4). This stop-log control was created for the purpose of providing water for fire fighting purposes. Unfortunately, it also acts as a flow constriction as the stop-logs are kept in place all year except during maintenance of the plant water inlet pipes.

When the water level rises above the arched culvert bridge, water goes over the right bank and flows around the bridge through an area that is a few hundred metres wide between the bridge and the Newfoundland Hardwoods Ltd. boiler room.

In the lower portion of this reach, the left bank is a steep infilled area. The right bank was swampy, containing dead trees and scruff brush until the trunk sewer construction cleared and infilled the area. Upstream of this is a wide boggy flood plain that has been encroached to a small extent on both banks during the course of the study. During the fall of 1980, tree stumps and debris were dumped on the upper portion of the left bank of the reach, beside the Sweet Feed building. The right bank was filled with excavated organic soil found during the installation of the trunk sewer in 1982.

(b) Mount Pearl: This reach starts at a wooden foot bridge (Steady Waters Bridge) located at the end of Forest Avenue, and extends upstream just past Winston Avenue, not far from the Water Survey of Canada hydrometric gauge site (027M010). A rock ledge acts as a natural control with a dramatic increase in channel slope downstream of the bridge. Most of the reach consists of a meandering channel cut into low wetlands. These wetlands are covered in coarse grasses and brush. The flood plain is generally boggy and covered in thick bush capable of supporting a person. Debris often jams at the right angle river bends, causing some local backwater and overflowing of channel

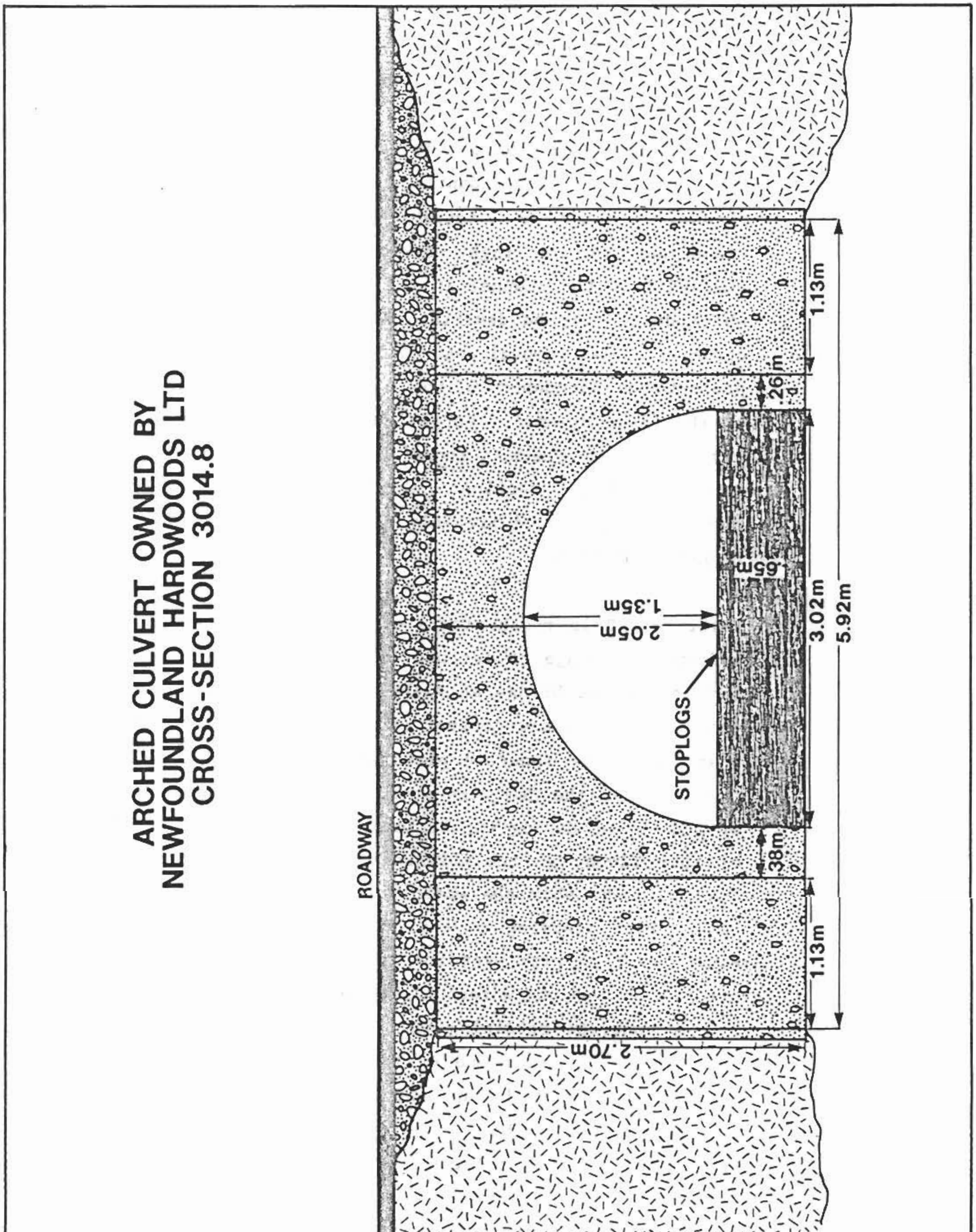
ARCHED CULVERT OWNED BY
NEWFOUNDLAND HARDWOODS LTD
CROSS-SECTION 3014.8
ROADWAY
1.13m
.26m
.65m
1.35m
2.05m
3.02m
5.92m
STOPLOGS
.38m
1.13m
2.70m

Figure 4

and/or cutoffs at bends. The large flood plain on the left bank is slowly being infilled by owners bordering the Topsail Road.

The upper portion of the reach displays several gravel bars due to the significantly increased channel slope. The channel becomes a straight ditch with the railway embankment on the right bank. Some material had been removed from the channel in the past years, to form a dike which could only be described as random and inefficient in design. This removal was undertaken in an attempt to protect the landowner on the right bank from the movement of the river channel. Large trees are on the banks of the river in this reach. There is evidence of old channels through this section which, although grown in substantially, could probably still carry some flow during high flow events.

(c) Kilbride: This reach starts at the Water Survey of Canada hydrometric gauge (Waterford River at Kilbride, 02ZM008), just below the bridge at Kilbride, and extends upstream to the ruins of an old control structure for the Bowring Park boat lake. The concrete piers which were used to hold the stop-logs of the dam were moved (dropped about 1.2 metres) by bed scour during the September 1, 1948 flood. Pictures of this flood, in the possession of the Superintendant of Bowring Park, show the water flowing over Waterford Bridge Road. Past flooding at the downstream end has caused the water level to rise over the left bank and flow around the bridge and down the Waterford Bridge road.

A concrete retaining wall has been built on the left bank at the upstream end of the reach. On the downstream section of the wall, an addition has been added. While this appears to have contained many floods, it was overtopped at the upstream interface of the low and high retaining walls during the November 26, 1981, event.

The channel is well defined and gravel lined, a few clumps of bushes have grown along the banks at various spots along the channel. The constriction of the channel under the Crosstown Arterial Overpass is rip-rapped along the right bank. A concrete ledge exists in the stream below this rip-rap and an hydraulic jump had been reported to occur below this point during high flows.

4.3.2 Model Structure

The HEC-2 program can compute water surface profiles for river channels of any cross-section for either subcritical or supercritical flow conditions. The effects of various hydraulic structures such as bridges, culverts, weirs, embankments and dams may be considered in the computation. The principal use of the program is for determining profiles for floods of various frequencies for both natural and modified conditions. The model requires a series of cross-sections and hydraulic parameters depicting the main reaches of the river, usually established by changes in slope, conveyance area, roughness or man-made structures. Equations 1 and 2 are used to calculate the profile commencing at a known initial downstream water level and progressing reach by reach upstream following the Standard Step procedure.

The model was set up to represent all relevant physical and hydraulic features for each of the three reaches.

All cross-sections were assigned a numeric code, adopting the convention of looking downstream from left to right bank. Their associated code corresponds to those for the field measured cross-sections except that, for each of the three reachs, a constant was added to the cross-section number in order to differentiate between each reach (a constant of 1000, 2000 and 3000 was applied to the reaches at Kilbride, Mount Pearl and Donovans respectively).The cross-section numbers do not represent a distance from a given point.

The computation begins at a control section (location of known water surface elevation) in the river channel and proceeds upstream for subcritical flow or downstream for supercritical flow. In cases where flow passes from subcritical to supercritical, or vice versa, during computations, it is necessary to compute the entire profile twice, assuming alternately subcritical and supercritical flow. From the above results, the water surface profile can be determined.

Manning's 'n' can be specified in the program in a variety of ways. This permits consideration of the various factors on which the parameter depends, such as type and amount of vegetation, channel configuration and stage. In order to solve for Manning's 'n' during calibration from known high water marks along the river reach, the discharge, relative ratios of the 'n' values for the channel and overbanks, and the water surface elevation at each cross-section must be known. Another method is to specify the discharge and an assumed set of 'n' values and have the program compute a set of water surface profiles which can be compared with the high water profile. The latter technique was employed in this study

Energy losses, due to expansion and contraction of flow and caused by changes in channel geometry, are estimated by employing expansion and contraction coefficients. These coefficients, which are given in the HEC-2 users manual (8). The coefficients are multiplied by the absolute difference in velocity heads between the cross-sections to estimate the energy loss caused by the transition.

Energy losses caused by structures such as bridges and culverts are computed in two parts. First, the losses due to expansion and contraction of the cross-section on the upstream and downstream sides of the structure are computed. Second, the loss through the structure itself is computed by either the normal bridge routine or the special bridge routine.

The normal bridge routine is particularly applicable for bridges without piers, bridges under high submergence and for low flow through circular and arch culverts. The normal bridge routine is automatically used by the computer for bridges without piers and under low flow control even if data are prepared for the special bridge routine.

The special bridge routine computes losses through the structure for low flow, weir flow and pressure flow or for any combination of these. The special bridge routine can be used for any bridge but should be used for trapezoidal bridges with piers where low flow occurs, for pressure flow through circular or arch culverts and whenever flow passes through critical when going through the structure.

The set-up of the model for each reach is shown in Appendix F.

4.3.3 Data Requirement

The basic data required for the calibration and verification of the mathematical hydraulic model and for the simulation of the 1:20 and 1:100 year flood profiles are river cross-sections at appropriate locations for each of the three delineated reaches, and measured water surface profiles under a range of suitably high and fairly stable flows. In order to determine the areal extent of land which would be inundated by the 1:20 and 1:100 year flood flows, accurate topographic maps are required.

4.3.3.1 Cross-Section Data

Field surveys were undertaken as part of the present study to measure typical channel and floodplain cross-sections. In order to do so, a bench mark network was established in the vicinity of each of the three reaches. Those bench marks were established by differential levelling with a precision of ± 0.005 metres. The cross sections were then surveyed from these bench marks.

For the reach at Kilbride, a total of 15 cross-sections were surveyed. In addition to the surveyed cross-sections, five cross-sections were interpolated from data for adjacent sections. The interpolated sections were felt to be representative and a field survey was considered unnecessary. These extra cross-sections were provided in order to satisfy the HEC-2 program's requirements with regard to the adequate definition of channel geometry for sudden transitions - for example at bridges and culverts - and between pairs of adjacent sections for which the velocity differences exceeded desirable limits.

Later on during the study, it was found that the simulated water profile for the upper 95% confidence limit, 1:100 year flood was greater than the maximum elevation surveyed for a few cross-sections. Those sections were extended using 1:1250 scale topographical maps. Since the model was not found to be sensitive to those changes, the most reasonable and accurate cross-sections were adopted. The location of each cross-section is shown on Figure 5 and the cross-sections plots are shown in Appendix A. The cross-sections are presented in HEC-2 input format in Appendix F.

Along the reach at Mount Pearl, a total of 14 cross-sections were surveyed. A few of these had to be extended in the same manner as was done for the Kilbride reach, using 1:1250 scale topographic mapping. The locations of the cross-sections are shown on Figure 6 and the cross-section plots are shown in Appendix A. The cross-sections are presented in HEC-2 input format in Appendix F.

For the reach near Donovans, a total of 20 cross-sections were surveyed and two were added by interpolation of adjacent cross-sections to provide adequate definition of channel geometry for sudden transitions, and where the difference in velocity head was significant between cross-sections. Additional cross-sections were surveyed near the two culvert crossings. Three of these cross-sections were extended using 1:2500 scale topographic mapping in order to accomodate high flows. During the study period, a trunk sewer was constructed in the Donovans reach. Cross-sectional surveys taken before and after construction indicated there was an insignificant change in the topography of the flood plain. The locations of the cross-sections are shown on Figure 7. Plots of each cross-section are given in Appendix A and the HEC-2 input data is presented in Appendix F. All cross-section elevations are shown to geodetic datum.

4.3.3.2 Hydrometric Data

Water surface profiles, under a range of suitably high and fairly stable streamflows, were required for the calibration and verification of the mathematical hydraulic model used to simulate the 1:20 and 1:100 year flood profiles.

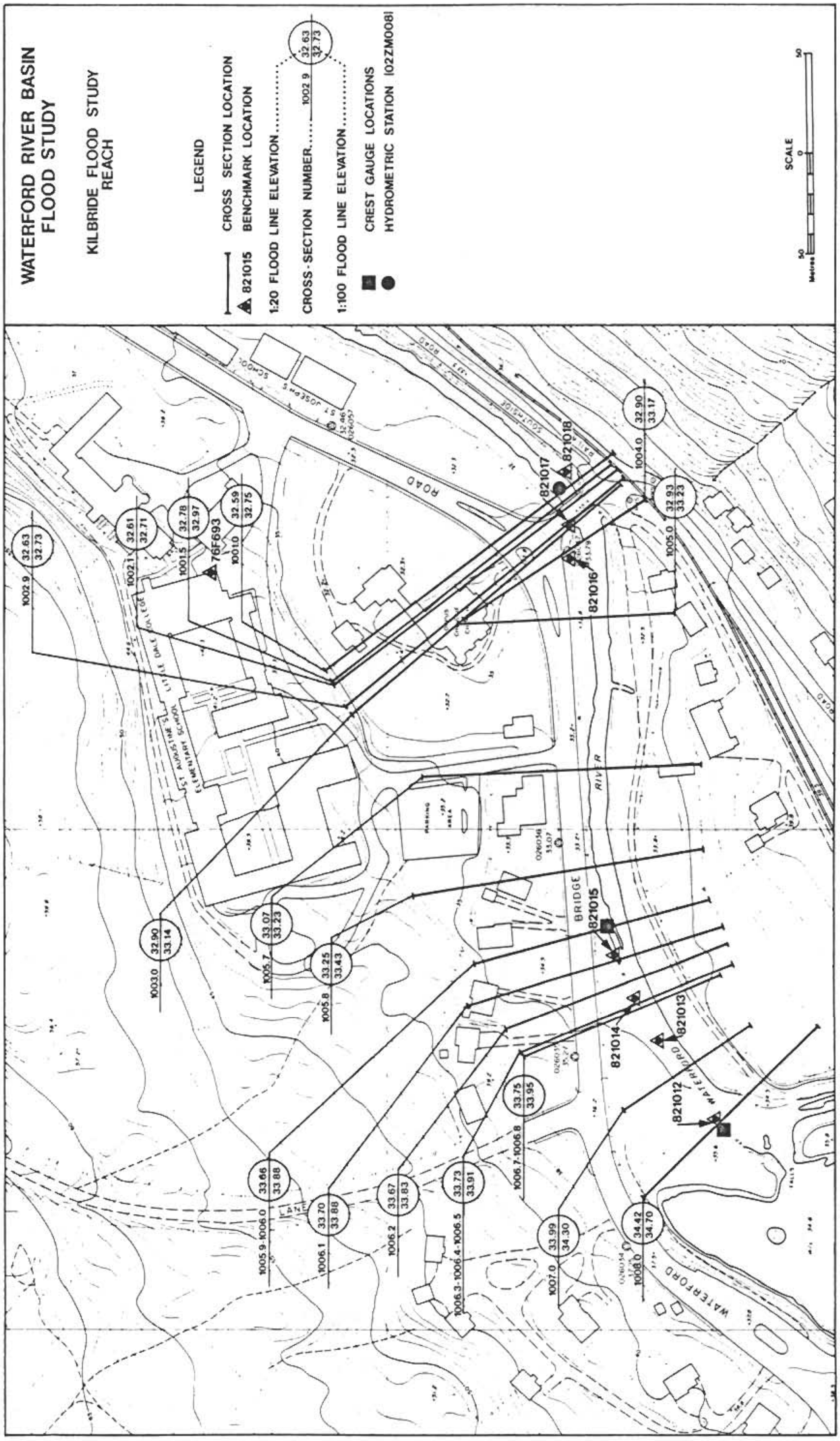
WATERFORD RIVER BASIN
FLOOD STUDY
KILBRIDE FLOOD STUDY
REACH
LEGEND
CROSS SECTION LOCATION
821015 BENCHMARK LOCATION
1:20 FLOOD LINE ELEVATION
CROSS-SECTION NUMBER
1:100 FLOOD LINE ELEVATION
CREST GAUGE LOCATIONS
HYDROMETRIC STATION (02ZM008)
SCALE
Metres
ROAD
BRIDGE
RIVER
WATERFORD
ST AUGUSTINE'S ELEMENTARY SCHOOL
LITTLE DALE COLLEGE
ST JOSEPH'S SCHOOL ROAD
SOUTHSIDE ROAD

Figure 5

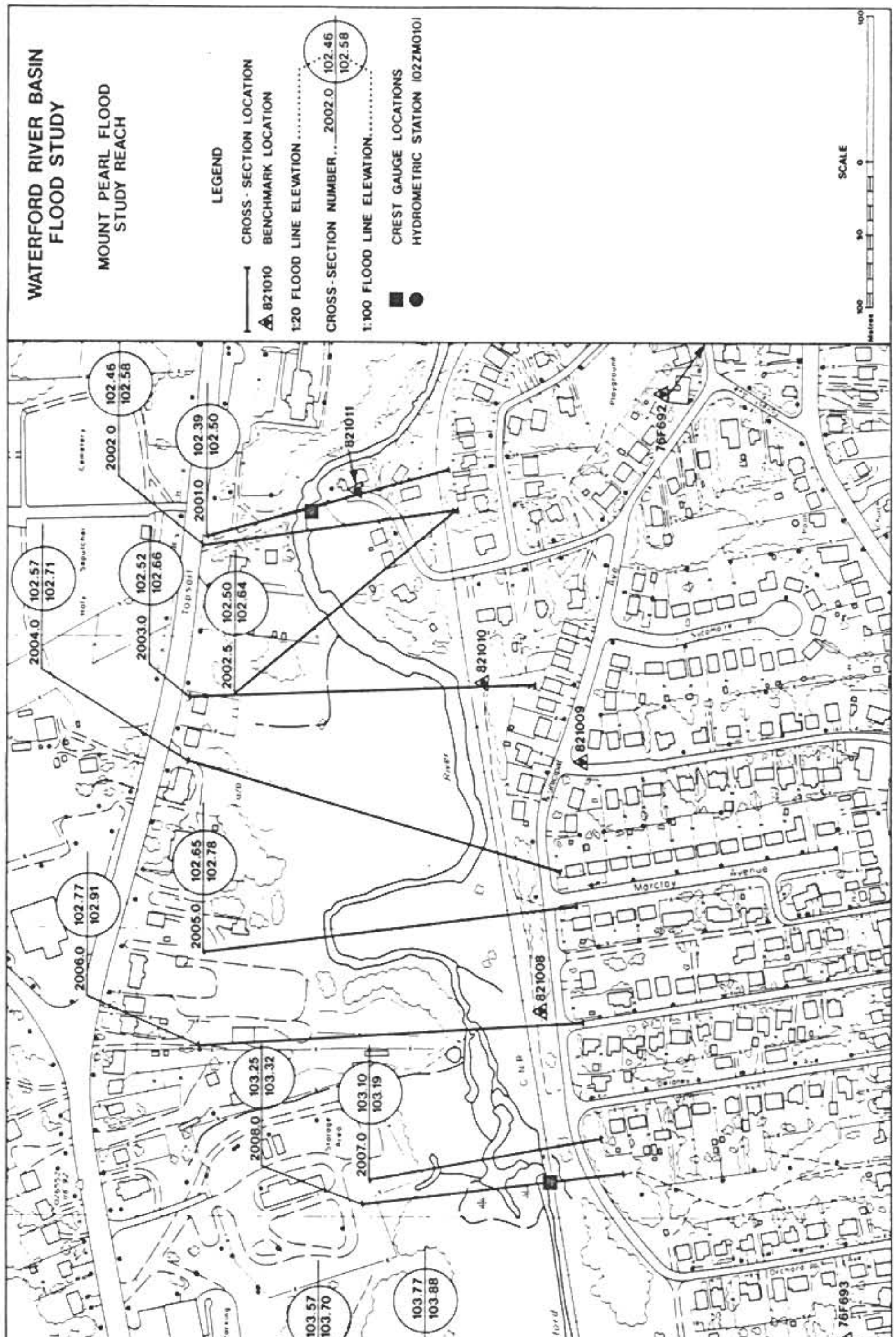
WATERFORD RIVER BASIN
FLOOD STUDY
MOUNT PEARL FLOOD
STUDY REACH
LEGEND
CROSS - SECTION LOCATION
821010 BENCHMARK LOCATION
1:20 FLOOD LINE ELEVATION
CROSS - SECTION NUMBER 2002.0
102.46
102.58
1:100 FLOOD LINE ELEVATION
CREST GAUGE LOCATIONS
HYDROMETRIC STATION 02ZM010
SCALE
Metres
100
50
0
100
2001.0
102.39
102.50
2002.0
2002.5
102.50
102.64
2003.0
102.52
102.66
2004.0
102.57
102.71
2005.0
102.65
102.78
2006.0
102.77
102.91
2007.0
103.10
103.19
2008.0
103.25
103.32
2009.0
103.57
103.70
2010.0
103.77
103.88
2011.0
103.80
103.91
2012.0
104.31
104.37
2013.0
104.50
104.55
821011
821010
821009
821008
821007
821006
76F692
76F693
Topsail
Waterford
River
Marclay
Avenue
Sycamore
Birch
Orchard
Cemetery
Holy Sepulchre
Parking
Storage Area
Playground
School

Figure 6

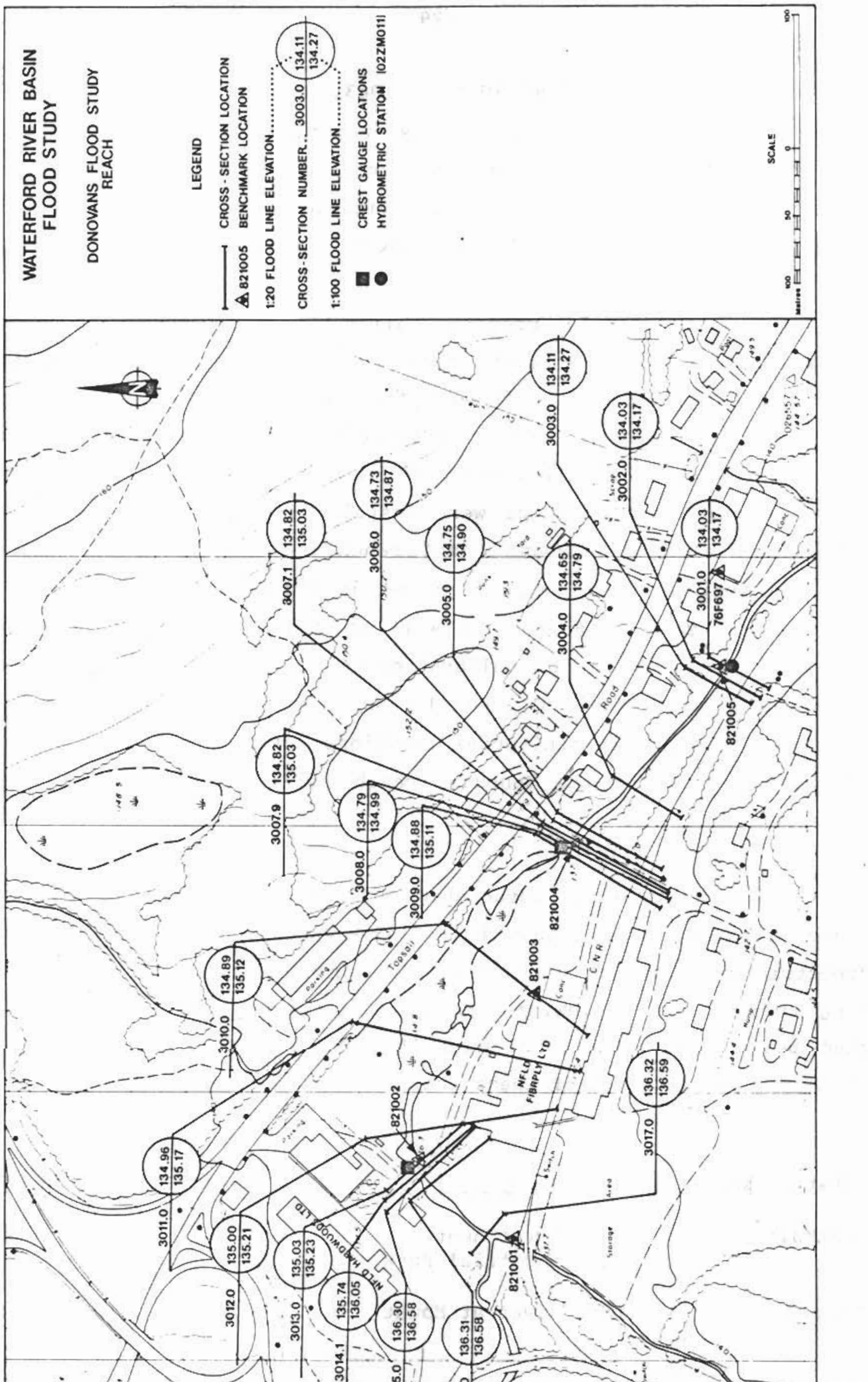
WATERFORD RIVER BASIN
FLOOD STUDY
DONOVANS FLOOD STUDY
REACH
LEGEND
CROSS-SECTION LOCATION
821005 BENCHMARK LOCATION
1:20 FLOOD LINE ELEVATION
CROSS-SECTION NUMBER 3003.0
134.11
134.27
1:100 FLOOD LINE ELEVATION
CREST GAUGE LOCATIONS
HYDROMETRIC STATION (02ZM011)
SCALE
Metres
100
50
0
100
3003.0
134.11
134.27
3002.0
134.03
134.17
3001.0
134.03
134.17
76F697
821005
3004.0
134.65
134.79
3005.0
134.75
134.90
3006.0
134.73
134.87
3007.1
134.82
135.03
3007.9
134.82
135.03
3008.0
134.79
134.99
3009.0
134.88
135.11
821004
821003
3010.0
134.89
135.12
NFLD FIBRPLY LTD
821002
3011.0
134.96
135.17
3012.0
135.00
135.21
3013.0
135.03
135.23
NFLD HARDWOOD LTD
3014.1
135.74
136.05
3015.0
136.30
136.58
3016.0
136.31
136.58
821001
3017.0
136.32
136.59
Road
CNR
Topsail
Storage

Figure 7

During the period of 1981 to 1983, a total of 12 water surface profiles were measured. Of these, only four high flow events were considered to be useful, due to the number of data points measured for a particular event, in the calibration and verification of HEC-2. These profiles are presented in tabular form in Appendix G.

The water profiles used for calibration were staked and surveyed by the staff of the Newfoundland Department of the Environment. It should be noted that staking of the flood profile was undertaken after the peak and not during the peak. The verification simulations were generally undertaken with water profiles compiled from crest gauges. For the study, a total of eight crest gauges were installed (three along the Donovans reach, three along the Mount Pearl reach and two along the Kilbride reach). A typical crest gauge is shown in Appendix B. Their locations are shown on Figures 5, 6 and 7 as well as Figure 2. Measured water surface elevations used in the calibration/verification of the HEC-2 model are presented in Appendix G. Due to vandalism of gauges, data was not always available at all monitoring locations. Also, there was a problem in obtaining data for the Donovans reach due to the construction of a trunk sewer.

The streamflows, associated with each of the measured water surface profiles, were obtained from the water levels measured at the hydrometric station for each reach in conjunction with the applicable rating curve. The following continuous recording hydrometric stations listed below were established under the Canada-Newfoundland Hydrometric Agreement. The location of these are shown in Figures 2, 5, 6 and 7.

Station No.	Station Name	Drainage Area	Period of Record
02ZM011	Waterford River near Donovans Industrial Park	11.2 km^2	1981 - present
02ZM010	Waterford River at Mount Pearl	17.8 km^2	1981 - present
02ZM008	Waterford River at Kilbride	52.7	1974 - present

The rating curves used for each hydrometric station are shown in Appendix D.

4.3.3.3 Hydraulic Structures

There are two bridges in the Donovans reach. These are described in section 4.3.1. The stop-logs in the Newfoundland Hardwoods Ltd. bridge are kept in place all year with removal being only for the purpose of maintenance of the water inlet pipes going to the plant.

4.4 MODEL CALIBRATION AND VERIFICATION

4.4.1 General

The calibration and verification of the hydraulic model is one of the most important steps to ensure a reliable simulation of water surface profiles. This was accomplished by using a split sample technique in which the model was first calibrated for each reach to two or three recorded events, and then verified by simulating one or two additional events using the same parameters (i.e. Manning's 'n' and expansion and contraction coefficients). When the recorded events are lower than the design events, the largest event is generally reserved for verification. If suitable results are obtained, then confidence is increased that the simulations are accurate.

The discharge values used for each reach correspond with those at the hydrometric station. Given the accuracy of the flow values, the sensitivity of the model to discharge values (discussed in Section 4.5.3) and the fact that there was less than a 10% difference in drainage area between the upstream and downstream end of each reach, flows were not distributed along the reaches.

4.4.2 Model Calibration

The surveyed profiles, tabulated in Appendix G and shown graphically in Appendix H, indicate that water surface elevations were not surveyed for each of the cross-sections used in the hydraulic model. For this reason, it was not possible to use the HEC-2 program to solve for

Manning's 'n' for each reach. Instead, a trial and error procedure was employed whereby Manning's 'n' values, assigned individually to a number of subreaches, were gradually varied in a series of runs of the HEC-2 program, until the computed water surface profile matched the observed data as closely as possible.

The Manning's 'n' values were first estimated by using the Cowan's equation (1) and adjusted accordingly to obtain a satisfactory calibration. The Cowan's equation is:

$$n = (n_0 + n_1 + n_2 + n_3 + n_4)m_5 \qquad 3$$

where n_0 is a basic 'n' value for a straight, uniform, smooth channel in the natural materials involved, n_1 is a value added to n_0 to correct for the effect of surface irregularities, n_2 is a value to account for variations in shape and size of the channel cross-section, n_3 accounts for obstructions, n_4 accounts for vegetation and flow conditions, and m_5 is a correction factor to account for meandering of the channel. Values of n_0 to n_4 and m_5 were selected from a table, which is reproduced in Appendix E, according to the given conditions.

The coefficients of contraction were taken as 0.10 for the gradual transition sections and 0.30 for culvert sections. The coefficients of expansion were taken as 0.30 for gradual transition sections and 0.50 for culvert sections. These values are in line with values suggested by the Hydrologic Engineering Centre (Ref. 8)

For the reach at Kilbride, two events were used for calibration. These are listed below by date and respective discharge at hydrometric station 02ZM008 (Waterford River at Kilbride):

1. September 2, 1983 - 18.5 m^3/s
2. October 26, 1983 - 39.3 m^3/s

Since supercritical flow conditions were not expected, a downstream control was required. Section #1001, which corresponds to the hydrometric station location, was selected as the starting control since the water level was available there for each event.

The Manning's 'n' for the channel for the calibrated model ranged from 0.015 to 0.04 along the reach, while, for the overbanks, it varied from 0.03 to 0.060. The Manning's 'n' and the coefficients of contraction and expansion for the calibrated model are listed by cross section in Table 2. The final calibration run profiles are displayed in graphical form in Appendix H as well as in tabular form in Appendix G.

For the reach at Mount Pearl, two events were also selected for calibration. These are listed here by date and associated discharges:

1. June 20, 1982 - 8.67 m^3/s
2. October 26, 1983 - 12.0 m^3/s

The discharge corresponds to the discharge measured at the hydrometric station 02ZM010 (Waterford River at Mount Pearl). It is important to note that the hydrometric station does not correspond to Cross-section #2001 (downstream cross-section used as control). For the design events where the water elevation was not available at Cross-section #2001, the water level at the hydrometric station (not to geodetic datum) was used, adjusting it by adding 0.5 m (which is the average difference in water elevation between those two sections for the measured events, the range being 0.48 to 0.51 for the three events). This adjustment factor had to be used because of the lack of information at Station #2001 to develop an appropriate rating curve.

The Manning's 'n' values for this reach ranged from 0.025 to 0.250 for the channel, and from .030 to .275 for the overbanks. Those extremely high Manning's 'n' values are for Sections #2006 to #2010 where the degree of meandering is severe and the effect of debris is extremely significant. The values are comparable to those presented in Chow (1). The Manning's 'n' and the coefficients of contraction and expansion are listed by cross-section in Table 3. The final calibration run profiles are displayed in graphical form in Appendix H and in tabular form in Appendix G.

TABLE 2

PARAMETERS FOR CALIBRATED MODEL

REACH AT KILBRIDE

Cross-Section Number	Distance from Downstream End of Reach (m)	Manning's 'n' Left Bank	Manning's 'n' Channel	Manning's 'n' Right Bank	Coefficients Contraction	Coefficients Expansion
1001	0.0	.030	.015	.030	.30	.50
1001.5	6.0	.030	.015	.030	.30	.50
1003	16.6	.030	.015	.030	.10	.30
1004	27.6	.030	.020	.030	.10	.30
1005	63.8	.030	.020	.030	.10	.30
1005.7	138.8	.030	.020	.030	.10	.30
1005.8	187.8	.035	.030	.035	.10	.30
1005.9	219.7	.035	.030	.035	.10	.30
1006.1	236.8	.035	.040	.050	.10	.30
1006.2	248.8	.030	.040	.050	.10	.30
1006.3	261.3	.030	.040	.050	.10	.30
1006.5	262.0	.035	.040	.050	.10	.30
1006.7	263.5	.060	.040	.050	.10	.30
1007	302.0	.045	.040	.045	.10	.30
1008	327.5	.045	.040	.045	.10	.30

TABLE 3

PARAMETERS FOR CALIBRATED MODEL
REACH AT MOUNT PEARL

Cross-Section Number	Distance from Downstream End of Reach (m)	Manning's 'n' Left Bank	Manning's 'n' Channel	Manning's 'n' Right Bank	Coefficients Contraction	Coefficients Expansion
2001	0	.090	.080	.090	.30	.50
2002	15	.090	.080	.090	.10	.30
2002.5	76	.100	.090	.100	.10	.30
2003	177	.100	.090	.100	.10	.30
2004	277	.120	.110	.120	.10	.30
2005	409	.120	.110	.120	.10	.30
2006	553	.175	.150	.175	.10	.30
2007	658	.275	.250	.275	.10	.30
2008	726	.275	.250	.275	.60	.80
2009	859	.275	.250	.275	.10	.30
2010	925	.150	.130	.150	.10	.30
2011	978	.030	.025	.030	.10	.30
2012	1069	.045	.035	.045	.10	.30
2013	1128	.030	.025	.030	.10	.30

For the reach at Donovans, the three calibration events used were:

1. June 20, 1982 - 4.47 m^3/s
2. September 2, 1983 - 4.45 m^3/s
3. October 26, 1983 - 6.72 m^3/s

The discharge corresponds to the flow at the hydrometric station 02ZM011 (Waterford River at Donovans) located at Section #3001. Two culverts, one at Cross-section #3007.1 (see Figure 3) and the other at Cross-section #3014.8 (see Figure 4) were simulated using the special bridge method. The stop-logs located at Cross-section #3014.8 were considered by adding a head loss estimated with the following equation:

$$H_L = kv^2$$

where H_L = head loss (m)
k = constant
v = velocity of the water at section considered (m/s)

An initial head loss was assumed and then 'k' was adjusted using the computed velocity from the model. The 'k' obtained for the calibration events is equal to 0.14.

The Manning's 'n' ranged from 0.03 (in the culverts) to 0.085 in the channel and was set at 0.090 for the overbanks. The Manning's 'n' and *the coefficients of contraction and expansion are listed by cross-section* in Table 4. The final calibration simulation run profiles are displayed in graphical form in Appendix H and in tabular form in Appendix G.

Critical flow conditions were encountered in the HEC-2 runs for all of the reaches. However, when flood profiles were calculated by starting the computation at a known water level at the upstream end of the study reach, the difference between these profiles and the profiles obtained by assuming sub-critical flow conditions were insignificant. The jumps occurred over a short distance and the profile corrected itself quickly. Therefore, subcritical flow conditions were assumed throughout the study.

TABLE 4

PARAMETERS FOR CALIBRATED MODEL

REACH AT DONOVANS

Cross-Section Number	Distance from Downstream End of Reach (m)	Manning's 'n' Left Bank	Manning's 'n' Channel	Manning's 'n' Right Bank	Coefficients Contraction	Coefficients Expansion
3001	0.0	.090	.085	.090	.30	.50
3002	9.0	.090	.085	.090	.10	.30
3003	16.0	.090	.085	.090	.10	.30
3004	115.0	.090	.085	.090	.10	.30
3005	161.0	.090	.085	.090	.30	.50
3006	168.0	.090	.085	.090	.10	.30
3007.1	168.5	.090	.030	.090	.10	.30
3007.9	171.85	.090	.085	.090	.30	.50
3008	172.35	.090	.085	.090	.10	.30
3009	178.35	.090	.085	.090	.30	.50
3010	274.35	.090	.085	.090	.10	.30
3011	357.35	.090	.085	.090	.10	.30
3012	430.35	.090	.085	.090	.10	.30
3013	450.35	.090	.085	.090	.30	.50

TABLE 4 (Cont'd)

PARAMETERS FOR CALIBRATED MODEL

REACH AT DONOVANS

Cross-Section Number	Distance from Downstream End of Reach (m)	Manning's 'n'			Coefficients	
		Left Bank	Channel	Right Bank	Contraction	Expansion
3014.1	450.45	.090	.030	.090	.30	.50
3014.7	456.68	.090	.030	.090	.30	.50
3014.8	456.78	.090	.085	.090	.10	.30
3014.9	456.88	.090	.085	.090	.10	.30
3015	457.28	.090	.085	.090	.10	.30
3016	464.28	.090	.085	.090	.10	.30
3017	519.28	.090	.085	.090	.10	.30

4.4.3 Model Verification

For each verification event, the downstream control elevation and the discharge were determined. The Manning's 'n' and the coefficients of contraction and expansion, determined during calibration, were used. These data were input to the model and the water surface profiles were computed. The events used for Kilbride are:

1. November 26, 1981 - 47.26 m^3/s
2. September 15, 1983 - 10.2 m^3/s

The verification run profiles are displayed in graphical form in Appendix H and in tabular form in Appendix G.

For the reach at Mount Pearl, two events were selected:

1. June 21, 1982 - 8.54 m^3/s
2. October 4, 1982 - 11.1 m^3/s

Unfortunately, at the time that those water profiles were measured, only two crest gauges were installed, one at cross-section #2001 and one at #2008, of which only one station could be used for verification purposes since one was the control station (#2001). The results are displayed in graphical form in Appendix H and in tabular form in Appendix G.

At Donovans, only one event was available for verification, which was October 4, 1982 with a peak discharge of 6.89 m^3/s. Unfortunately, none of the crest gauge locations correspond to those for stations for which water elevation measurements were taken for the calibration events. It is, therefore, recommended that additional hydraulic information be collected for the purpose of future model verification. The results are shown in graphical form in Appendix H and in tabular form in Appendix G.

As can be seen from the verification runs, the model for each reach is acceptable, but it is recommended that further calibration and verification be undertaken for Donovans when more data are available. It is important to remember that the model was calibrated using staked events that occurred after the peak, when flows and water levels were relatively stable. Errors in profile measurement will be more significant if the time difference between the upstream and downstream staking is great. This was the case for the September 2, 1983 event at Donovans where it took 53 minutes to stake the whole reach. Also, the shape of the hydrograph depends on the type of runoff event.

4.5 SENSITIVITY TESTING

4.5.1 General

In order to determine the sensitivity of the water surface profile to selected variations in channel elevation, discharge, downstream control elevation, boundary roughness, and expansion and contraction coefficients, various sensitivity simulations were undertaken for each reach. The testing was done by varying one parameter at a time and holding the other parameters at their previously determined values. The resulting water surface profile was then compared to the calibrated profile evaluating the actual change in water surface elevation at ten representative cross-sections. Testing of each reach was undertaken using the measured event with the highest flow.

The parameters changed in the computer modelling were: the bottom of the entire channel by $\pm$ 0.15 metre; the discharge by $\pm$ 30%; the starting water level by $\pm$ 0.15 metre; all Manning's 'n' values by $\pm$ 10%; and, finally, all expansion and contraction coefficients by $\pm$ 10%. The results are shown in Tables 5, 6 and 7, and are discussed in the following sections.

4.5.2 Sensitivity to Channel Elevation

To assess the influence of channel changes, the calibrated hydraulic model was run with a plus or minus 0.15 metre change in the channel bottom. This was thought to be a high value for channel change. As well, it is unlikely a channel would change by that amount throughout

TABLE 5

HEC-2 SENSITIVITY ANALYSIS FOR KILBRIDE REACH

Change in Water Surface Elevation (metres) at Cross-Section

PARAMETER	PARAMETER VARIATION	1001	1004	1005	1005.7	1006	1006.2	1006.3	1006.7	1007	1008
Bottom of Channel	+0.15 m	0.0	0.0	0.0	+0.03	+0.03	+0.0	+0.01	-0.02	+0.07	+0.06
	-0.15 m	0.0	-0.11	-0.08	-0.30	-0.34	-0.35	-0.47	-0.47	-0.33	-0.40
Discharge	+30%	0.0	+0.11	+0.11	+0.16	+0.19	+0.16	+0.18	+0.18	+0.23	+0.29
	-30%	0.0	-0.06	-0.07	-0.24	-0.20	-0.21	-0.22	-0.22	-0.25	-0.34
Starting Water Level	+0.15 m	+0.15	+0.13	+0.13	0.0	0.0	+0.01	0.0	0.0	0.0	0.0
	-0.15 m	-0.15	-0.13	-0.13	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Mannings 'n'	+10%	0.0	0.0	+0.01	0.0	+0.05	+0.05	+0.06	+0.06	+0.09	+0.06
	-10%	0.0	0.0	-0.01	0.0	-0.03	-0.05	-0.06	-0.06	-0.09	-0.07
Expansion and Contraction Coefficients	+10%	0.0	+0.01	+0.01	0.0	0.0	0.0	0.0	0.0	0.01	0.0
	-10%	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	-0.02	-0.01

TABLE 6

HEC-2 SENSITIVITY ANALYSIS FOR REACH AT MOUNT PEARL

Change in Water Surface Elevation (Metres) at Cross-Section

PARAMETER	PARAMETER VARIATION	2001	2002.5	2004	2005	2006	2008	2009	2010	2011	2013
Bottom of Channel	+0.15 m	0.0	+0.00	+0.00	+0.01	+0.01	+0.09	+0.01	+0.00	+0.01	+0.01
	-0.15 m	0.0	-0.07	-0.08	-0.07	-0.06	-0.25	-0.04	-0.04	-0.04	-0.24
Discharge	+30%	0.0	+0.04	+0.06	+0.08	+0.10	+0.13	+0.12	+0.09	+0.10	+0.06
	-30%	0.0	-0.04	-0.07	-0.09	-0.11	-0.15	-0.15	-0.12	-0.10	-0.06
Starting Water Level	+0.15 m	+0.15	+0.12	+0.10	+0.08	+0.05	+0.01	-0.01	-0.01	0.0	0.0
	-0.15 m	-0.15	-0.11	-0.08	-0.05	-0.02	0.0	0.0	0.0	0.0	0.0
Mannings 'n'	+10%	0.0	+0.01	+0.01	+0.02	+0.04	+0.05	+0.04	+0.03	+0.04	+0.02
	-10%	0.0	-0.01	-0.02	-0.03	-0.03	-0.05	-0.05	-0.04	-0.04	-0.01
Expansion and Contraction Coefficients	+10%	0.0	0.0	+0.01	0.0	-0.02	0.0	0.0	+0.01	0.0	0.0
	-10%	0.0	0.0	0.0	0.0	-0.01	0.0	-0.01	-0.01	0.0	0.0

TABLE 7

HEC-2 SENSITIVITY ANALYSIS FOR DONOVANS REACH

Change in Water Surface Elevation (Metres) at Cross-Section

PARAMETER	PARAMETER VARIATION	3001	3002	3004	3007.1	3010	3011	3012	3014.1	3014.8	3017
Bottom of Channel	+0.15 m	0.0	0.0	+0.05	+0.03	+0.04	+0.03	+0.02	+0.01	+0.34	+0.31
	-0.15 m	0.0	-0.01	-0.20	-0.13	-0.15	-0.22	-0.28	-0.20	-0.59	-0.56
Discharge	+30%	0.0	0.0	+0.14	+0.13	+0.15	+0.14	+0.13	+0.23	+0.20	+0.19
	-30%	0.0	-0.01	-0.20	-0.17	-0.19	-0.17	-0.16	-0.09	-0.04	-0.06
Starting Water Level	+0.15 m	+0.15	+0.14	-0.02	-0.01	-0.01	0.0	-0.01	-0.01	0.0	0.0
	-0.15 m	-0.15	-0.15	+0.04	+0.02	+0.03	+0.02	+0.01	0.0	0.0	0.0
Mannings 'n'	+10%	0.0	0.0	+0.06	+0.04	+0.05	+0.05	+0.04	+0.05	+0.0	+0.01
	-10%	0.0	-0.01	-0.06	-0.04	-0.04	-0.04	-0.05	-0.07	0.0	-0.01
Expansion and Contraction Coefficients	+10%	0.0	0.0	+0.01	0.0	0.0	0.0	0.0	0.0	+0.01	0.0
	-10%	0.0	-0.01	-0.01	0.0	0.0	0.0	-0.01	0.0	0.0	0.0

an entire study reach. Changes in the water profile of up to a maximum of 0.47 metres for Kilbride, 0.24 metres for Mount Pearl and 0.59 metres for Donovans were calculated. These values were all as a result of a decrease in the channel bottom. Much lower values were encountered for an increase in channel bottom elevations. It can be concluded that the flood profile is somewhat sensitive to decreases in channel bed elevations while it is relatively insensitive to increases in channel bed elevation. An exception to the latter is the upper portion of the Donovan's reach for which further data collection and calibration was recommended in Section 4.4.3.

4.5.3 Sensitivity to Peak Discharge

When simulating the water level profiles for each event, the discharge associated with each event had to be estimated using different rating curves. The rating curves have been revised a few times by the Water Survey of Canada when additional measurements of streamflow became available (see curves, Appendix D). The difference between the rating curves for a single station for these stations could easily be as much as 30% for high flows. Sensitivity simulations were then undertaken to determine the effect of variations of ± 30% in discharge on the water level profile. The variations in water surface elevation are generally less than 0.20 metre with a maximum variation of 0.34 metre for the upstream end of the Kilbride reach. Therefore, the water profile is not very sensitive to large variations in discharge.

4.5.4 Sensitivity to Starting Water Level

Since the starting water level in many ways controls the resulting profile, it was decided to examine the influence of a change of plus or minus 0.15 metre in that value. As anticipated, the variation in water surface elevation was significant near that starting cross-section, but quickly decreased to zero. Therefore, any error in the starting water level would influence the first few cross-sections but would have relatively little, if any, influence upstream.

4.5.5 Sensitivity to Roughness Coefficient

It was found that by varying the Manning's 'n' roughness coefficient by ± 10%, the resulting variation in water surface elevation was insignificant, always less than 0.05 metres. Therefore, a small error in Manning's 'n' coefficients would insignificantly affect the results.

It should be noted for the Mount Pearl reach, some unusually large values of Manning's 'n' had to be chosen for proper model calibration. However, these values correspond to the potential values described by Chow (1).

4.5.6 Sensitivity to Expansion and Contraction Coefficients

Since the coefficients of expansion and contraction were selected by using the HEC-2 user's manual (8) as a guide, it was decided to vary them by ± 10%. The resulting variation in water surface elevation was always less than 0.02 metres, indicating that the model is essentially insensitive to errors in that parameter.

4.6 1.20 and 1:100 YEAR FLOOD PROFILES

4.6.1 Flood Profiles

The main objective of this investigation was to determine flood profiles along the study reach for floods with recurrence intervals of 20 and 100 years.

With the use of the 1:20 and 1:100 year rainfalls and their upper 95% confidence limits, the 1:20 and 1:100 year design flood flows and their upper 95% confidence limits were computed using the single event hydrologic simulation model HYMO (5).

Since the available rating curves were undefined for high flows, these had to be extended to have the starting water levels corresponding to the streamflows. Those curves were extended using the following equation (2):

$$y = a + bx^n$$

where y = water level (m)

x = discharge (m^3/s)

a, b, n = constants

a = 27.15, 132.77 and 99.80 for the Kilbride, Donovan's and Mount Pearl reaches respectively.

b = 3.30, 0.40 and 1.56 for the Kilbride, Donovan's and Mount Pearl reaches respectively.

c = 0.12, 0.43 and 0.17 for the Kilbride, Donovan's and Mount Pearl reaches respectively.

The constants were determined using points of known water level and discharge from the existing curve.

Then, using the calibrated HEC-2 hydraulic model with the cross-sectional information as shown in Appendix A, Manning's 'n' and coefficients of expansion and contraction as previously determined, the water profiles for flows of design return period were determined.

Table 8 shows the flows and the starting water levels for the three reaches for the desired profiles. Those profiles are shown in tabular form in Tables 9, 10 and 11, and in graphical form in Appendix J for the reaches at Kilbride, Mount Pearl and Donovans respectively.

4.6.2 <u>Flood Hazard Areas</u>

The extent of the flooded areas associated with the 1:20 and 1:100 year flood profiles was plotted on existing topographic maps (the Mount Pearl and Donovans reaches on 1:2500 scale maps and the Kilbride reach on 1:1250 scale maps). An interpretation of the backwater profiles and associated computer output, together with an assessment of the extent of flooded areas was undertaken in order to identify flood hazard locations. The areal extent of flooding is shown on the maps inserted at the back of this report.

TABLE 8

FLOWS AND STARTING WATER ELEVATIONS

1:20 and 1:100 Year Profiles

	Flow (m^3/s)	Starting WL (m)
Kilbride		
20 year	64.6	32.59
20 year + 95% CL	78.0	32.72
100 year	81.5	32.75
100 year + 95%	102.0	32.90
Mount Pearl		
20 year	19.9	102.39
20 year + 95% CL	24.1	102.48
100 year	25.3	102.50
100 year + 95% CL	31.9	102.61
Donovans		
20 year	14.4	134.03
20 year + 95% CL	17.6	134.14
100 year	18.4	134.17
100 year + 95% CL	23.5	134.33

TABLE 9

1:20 AND 1:100 YEAR FLOOD PROFILES (Metres)

KILBRIDE

Cross-Section Number	20 Year	20 Year + 95% C.L.	100 Year	100 Year + 95% C.L.
1001	32.59	32.72	32.75	32.90
1001.5	32.78	32.94	32.97	32.15
1002.1	32.61	32.70	32.71	32.72
1002.9	32.63	32.71	32.73	33.04
1003	32.90	33.09	33.14	33.53
1004	32.90	33.12	33.17	33.60
1005	32.93	33.17	33.23	33.68
1005.7	33.07	33.20	33.23	33.59
1005.8	33.25	33.39	33.43	33.63
1005.9	33.66	33.83	33.88	34.17
1006	33.67	33.84	33.89	34.18
1006.1	33.70	33.85	33.88	34.09
1006.2	33.67	33.80	33.83	33.99
1006.3	33.74	33.87	33.90	34.08
1006.4	33.73	33.87	33.91	34.08
1006.5	33.74	33.88	33.92	34.09
1006.7	33.75	33.91	33.95	34.15
1006.8	33.76	33.91	33.95	34.15
1007	33.99	34.22	34.30	34.71
1008	34.42	34.64	34.70	34.97

TABLE 10

1:20 AND 1:100 YEAR FLOOD PROFILES (Metres)

MOUNT PEARL

Cross-Section Number	20 Year	20 Year + 95% C.L.	100 Year	100 Year + 95% C.L.
2001	102.39	102.48	102.50	102.61
2002	102.46	102.56	102.58	102.71
2002.5	102.50	102.61	102.64	102.78
2003	102.52	102.63	102.66	102.80
2004	102.57	102.68	102.71	102.85
2005	102.65	102.76	102.78	102.93
2006	102.77	102.88	102.91	103.06
2007	103.10	103.19	103.19	103.30
2008	103.25	103.35	103.32	103.45
2009	103.57	103.69	103.70	103.85
2010	103.77	103.86	103.88	104.01
2011	103.80	103.89	103.91	104.03
2012	104.31	104.37	104.37	104.41
2013	104.50	104.54	104.55	104.62

TABLE 11

1:20 AND 1:100 YEAR FLOOD PROFILES (Metres)

DONOVANS

Cross-Section Number	20 Year	20 Year + 95% C.L.	100 Year	100 Year + 95% C.L.
3001	134.03	134.14	134.17	134.33
3001.9	134.06	134.18	134.20	134.37
3002	134.03	134.14	134.17	134.32
3003	134.11	134.24	134.27	134.44
3004	134.65	134.77	134.79	134.95
3005	134.75	134.87	134.90	135.06
3006	134.73	134.85	134.87	135.04
3007.1	134.82	134.99	135.03	135.30
3007.9	134.82	134.99	135.03	135.30
3008	134.79	134.95	134.99	135.25
3009	134.88	135.06	135.11	135.39
3010	134.89	135.07	135.12	135.40
3011	134.96	135.13	135.17	135.44
3012	135.00	135.17	135.21	135.47
3013	135.03	135.19	135.23	135.49
3014.1	135.74	136.01	136.05	136.26
3014.7	136.30	136.54	136.58	136.81
3014.8	136.30	136.54	136.58	136.81
3014.9	136.30	136.55	136.58	136.81
3015	136.30	136.55	136.58	136.81
3016	136.31	136.55	136.58	136.81
3017	136.32	136.55	136.59	136.82

The floodlines approach the Newfoundland Fibrply Ltd. building, but for the most part, existing development is not inside the flood risk area in the Donovans reach. Flooding due to the 1:20 and 1:100 year design events is largely below existing development in the Mount Pearl reach. The design floods surround the Corpus Christie Church in the Kilbride reach. A small number of other buildings are affected as well. In general, there are only a few buildings affected by the design floods. However, since the watershed is being developed, it is recommended that future development be restricted in the flood prone areas.

5.0 FLOODING AND URBANIZATION

5.1 General

Usually, urbanization decreases the amount of pervious area and depressional storage and improves the capability of an area to convey stormwater runoff through and out of the area. As a result, there is a significant decrease in the amount of water which infiltrates into the soil and there is a corresponding increase in total volume of surface runoff, via the storm drainage system, for a given climatological input - other initial conditions such as soil moisture or antecedent precipitation being equal. Peak runoff rates are usually higher and occur sooner than under natural conditions. Thus there may be increased erosion and sedimentation as well as increased flooding.

When this study was initiated, it was expected that there would be considerable urbanization of the Waterford River basin within the 5-year study period. This would possibly have provided the opportunity to examine the effect of flooding due to urbanization and to develop a flood management strategy. The anticipated development did not occur. However, some general observations can still be made with respect to the flood profiles as a result of both the HYMO (5) and HEC-2 modelling.

5.2 Sensitivity of Floodlines

As mentioned in Section 3.0, the Urban Hydrology Study (5) concluded that no significant increase in streamflows have resulted due to future urbanization over the period 1974 to 1981. It was also concluded that no significant increases were likely in the future. This is largely due to the fact that the amount of pervious versus impervious areas would not change significantly. This is an unusual situation, but it is unusual for an area to contain such a high portion of impervious areas as does the Waterford River Basin.

If any increases in discharge peaks occur along the Waterford River in the near future (10 to 20 years or so), then they are likely to be small and within the accuracy associated with the HYMO modelling work (maximum ± 30%). When running the HEC-2 model, it was speculated that the discharge curves could be in error by as much as ± 30%. Therefore, as mentioned in Section 4.5.3, a sensitivity analysis was undertaken using a ±30% variation in discharge. This analysis showed the water profile is not very sensitive to large variations in discharge.

Tables 12, 13 and 14 present the 1:20 and 1:100 year development scenario profiles for the three reaches. The design flows used in the profile development were taken from the HYMO modelling report (5). Generally, the flood elevations are increased by 0.1 to 0.2 metres. Both this and the sensitivity analyses showed the flood profiles are not very sensitive to discharge.

In summary, the modelling work indicated that there would be little effect on flooding due to urbanization in the forseeable future.

If appropriate flood plain restrictions are enforced, it is unlikely the extent of the flood plain would change significantly by urbanization. If development, including alteration of the flood plain, is allowed to occur, the extent of the flood plain will, of course, be altered. The HEC-2 model developed for this study can be used to determine the extent of flooding under such changes in development within the area.

TABLE 12

FLOOD PROFILES (Metres)

KILBRIDE

Cross-Section Number	1:20 Year	1:20 Year Development Scenario $Q = 74.8 m^3/s$ SWL = 32.69 m	1:100 Year	1:100 Year Development Scenario $Q = 93.3 m^3/s$ SWL = 32.84 m
1001	32.59	32.69	32.75	32.84
1001.5	32.78	32.91	32.97	33.08
1002.1	32.61	32.68	32.71	32.73
1002.9	32.63	32.70	32.73	32.88
1003	32.90	33.05	33.14	33.35
1004	32.90	33.07	33.17	33.40
1005	32.93	33.12	33.23	33.48
1005.7	33.07	33.17	33.23	33.33
1005.8	33.25	33.35	33.43	33.55
1005.9	33.66	33.79	33.88	34.05
1006	33.67	33.80	33.89	34.06
1006.1	33.70	33.81	33.88	34.00
1006.2	33.67	33.77	33.83	34.92
1006.3	33.74	33.84	33.90	34.01
1006.4	33.73	33.84	33.91	34.01
1006.5	33.74	33.85	33.92	34.01
1006.7	33.75	33.88	33.95	34.06
1006.8	33.76	33.88	33.95	34.06
1007	33.99	34.16	34.30	34.54
1008	34.43	34.59	34.70	34.86

TABLE 13

FLOOD PROFILES (Metres)

MOUNT PEARL

Cross-Section Number	1:20 Year	1:20 Year Development Scenario $Q = 74.8m^3/s$ SWL = 32.69 m	1:100 Year	1:100 Year Development Scenario $Q = 93.3m^3/s$ SWL = 32.84 m
2001	102.39	102.47	102.50	102.57
2002	102.46	102.55	102.58	102.67
2002.5	102.50	102.60	102.64	102.73
2003	102.52	102.62	102.66	102.75
2004	102.57	102.67	102.71	102.80
2005	102.65	102.75	102.78	102.88
2006	102.77	102.87	102.91	103.00
2007	103.10	103.19	103.19	103.25
2008	103.25	103.35	103.32	103.40
2009	103.57	103.68	103.70	103.79
2010	103.77	103.85	103.88	103.96
2011	103.80	103.88	103.91	103.99
2012	104.31	104.36	104.37	104.38
2013	104.50	104.53	104.55	104.60

TABLE 14

FLOOD PROFILES (Metres)

DONOVANS

Cross-Section Number	1:20 Year	1:20 Year Development Scenario $Q = 74.8m^3/s$ SWL = 32.69 m	1:100 Year	1:100 Year Development Scenario $Q = 93.3m^3/s$ SWL = 32.84 m
3001	134.03	134.14	134.17	134.29
3001.9	134.06	134.18	134.20	134.33
3002	134.03	134.14	134.17	134.28
3003	134.11	134.24	134.27	134.40
3004	134.65	134.77	134.79	134.91
3005	134.75	134.87	134.90	135.02
3006	134.73	134.85	134.87	135.00
3007.1	134.82	134.99	135.03	135.23
3007.9	134.82	134.99	135.03	135.23
3008	134.79	134.95	134.99	135.18
3009	134.88	135.06	135.11	135.32
3010	134.89	135.07	135.12	135.33
3011	134.96	135.13	135.17	135.37
3012	135.00	135.17	135.21	135.40
3013	135.03	135.19	135.23	135.42
3014.1	135.74	136.01	136.05	136.21
3014.7	136.30	136.54	136.58	136.76
3014.8	136.30	136.54	136.58	136.76
3014.9	136.30	136.55	136.58	136.76
3015	136.30	136.55	136.58	136.76
3016	136.31	136.55	136.58	136.76
3017	136.32	136.55	136.59	136.77

6.0 REMEDIAL MEASURES

6.1 General

On the basis of the identification of historical flooding and utilizing the results of the HEC-2 backwater model for the 1:20 and 1:100 year floods, it was possible to identify alternative remedial measures for alleviating the flood hazard along the Waterford River.

Flood damages can be reduced by:

1. Structural measures which directly affect the flood characteristics; and

2. Non-Structural measures which are intended to modify the loss burden, either by reducing the potential for continued development in flood prone lands or by providing some form of economic relief from flood losses.

A detailed analysis of possible remedial measures was beyond the scope of the present investigations. However, based on the results of the study, it has been possible to identify some alternative remedial measures for further future consideration. Each of the measures, or combination thereof, would serve to reduce the flood risk.

6.2 Structural Measures

The following structural flood control measures should be considered:

1. Floodproofing of structures located in the flood plain.
   The possibility of raising structures to a level above that associated with the 1:100 year storm should be considered. In assessing the feasibility of this scheme, it is evident that not all of the flood prone structures could be flood-proofed in a cost effective manner (i.e. commercial buildings). It is also possible to construct a dyke or flood berm such that a small group of buildings are protected.

2. Improve hydraulic capacity of the bridges in Donovans Reach. In particular, if the flow capacity of the Newfoundland Fibrply Ltd. bridge were increased, the flood plain upstream would be reduced. In addition, by increasing the flow capacity, the chances of ice or debris jamming would be reduced. It is recommended that this be given consideration if and when the bridge(s) is being replaced.

6.3 Non-Structural Measures

In developing areas such as along the Waterford River, flood plain regulations should be implemented to restrict future development and thus reduce the potential for continued increases in flood damages. In this case, a two-zone floodway/flood-fringe concept is envisaged where zoning regulations would prohibit future development in the high hazard areas. Additional development might be permitted in the flood-fringe areas, depending on the degree hazard and the implemlentation of flood proofing measures to protect these developments. Other non-structural measures which might be considered include:

1. Maintain a program of debris clearing to reduce the potential of a flood as a result of debris blockage.
2. Relocation of flood prone structures. This could include the expropriation of properties most susceptable to damage.
3. Develop a low cost flood warning system to reduce the potential risk to loss of life and property damage during peak runoff events.

7.0 CONCLUSIONS

The governments of Canada and Newfoundland undertook a five-year urban hydrology study of the Waterford River on a work shared basis starting April 1, 1980. This study examines the flooding component of that overall study.

The primary purpose of this study was to determine 1:20 and 1:100 year return period open water flood profiles and to identify flood prone areas along the Waterford River.

The following summarizes the results of the flood study:

1. After a successful calibration/verification of the HEC-2 backwater model using the results of the field program, the model was utilized to calculate flood profiles along the study reaches for existing and potential future conditions. The sensitivity tests show that the model is not sensitive to variations in expansion and contraction coefficients. The model is most sensitive to a decrease in channel bottom elevation, which emphasizes the importance of the cross-sectional information. Small changes in the starting water level affects the water profile for only a very short distance at the downstream end of each reach. From the sensitivity tests for discharge and Manning's 'n', it can be seen that for the events used in calibration and verification, the model is not sensitive to reasonable variations in Manning's 'n'.

2. The areal extent of the 1:20 and 1:100 year floods are shown on topographic maps which are enclosed next to the back cover of this report.

3. There would be little increase in flooding due to urbanization unless the channel or flood plain were altered or urbanization of an extremely dense nature occurred.

4. The most attractive structural alternatives for reducing existing flood losses presently appear to be:
   (i) floodproofing, either individual or small groups of buildings; or
   (ii) improvement of hydraulic capacity of the bridge in the upstream end of the Donovan reach.

5. The most attractive non-structural measures for reducing flood losses presently appear to be:
   (i) implementation of floodplain regulations to prevent development in flood susceptible areas (within the 1:20 year flood risk area);
   (ii) relocation of floodprone structures as identified by the map; or
   (iii) implementation of a low cost flood warning system.

8.0 RECOMMENDATIONS

The following summarizes the recommendations of the flood study.

1. Additional hydraulic information should be collected for the purpose of further model verification for the Donovan's reach.

2, The areal extent of the 1:20 and 1:100 year design floods should be delineated on new topographic mapping and the flood risk areas should be designated under the Canada-Newfoundland Flood Damage Reduction Program. These maps should be used for future regulation of development along the Waterford River. The aerial extent of flood hazard lands as shown on the inserted maps should be adopted as identifying the potential extent of flooding within the study area until such time as updated mapping is available.

3. If it is desired to monitor the effect of development on flooding, then channel changes, streamflows, and water levels should continue to be monitored. The stream gauges established for the study should be maintained. Cross-sections should be taken periodically at the same sites as for the study, and water levels should continue to be monitored for high flow events.

4. The HEC-2 backwater model should be used to evaluate the effect of proposed floodplain changes.

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