Canada – Newfoundland
**Flood Damage Reduction**
Program

**Water Resources Division**
**Hydrological Modelling Section**

# Hydrotechnical Study of the Stephenville Area

## MAIN REPORT

Volume 1 of 2


**Nolan Davis & Associates Limited**

in association with

**Cumming – Cockburn & Associates Limited**

Department of Environment

Environment Canada

CUMMING-COCKBURN & ASSOCIATES LIMITED

*Consulting Engineers*

CCL CONSULTANTS

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7075
February 8, 1984

Government of Newfoundland and Labrador,
Department of Environment,
Water Resources Division,
Elizabeth Towers,
St. John's, Newfoundland,
A1C 5T7.

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

Gentlemen:

Re: Hydrotechnical Study of the Stephenville Area

We take pleasure in submitting our final report on the above mentioned study. The comments and suggestions from the Technical Committee on the previous interim and draft reports have been incorporated in this version.

The methodology and findings of our investigations are discussed herein with additional information on the field program provided as a supplementary report. We respectfully suggest that serious consideration be given to immediately implementing the recommendtions found in our report.

We would like to express our sincere thanks to you and other members of the Committee for your cooperation and assistance throughout this study.

All of which is respectfully submitted.

Yours very truly,

NOLAN DAVIS & ASSOCIATES LIMITED,

B. Davis

B. Davis, P. Eng.

CUMMING-COCKBURN & ASSOCIATES LIMITED,

H. S. Belore

H. S. Belore, P. Eng.

ASSOCIATION OF PROFESSIONAL ENGINEERS PROVINCE OF NEWFOUNDLAND
H. S. BELORE
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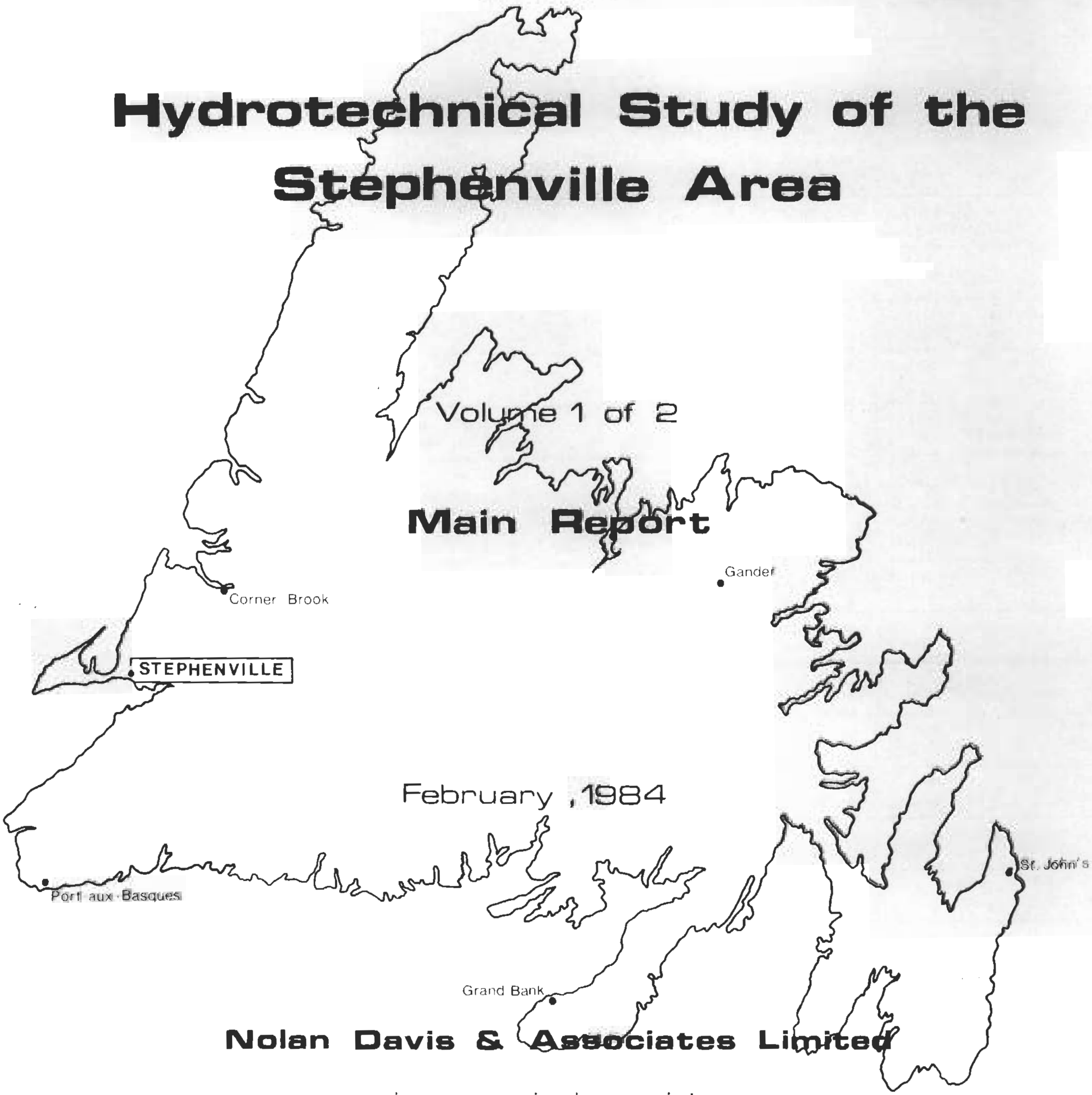
Flood Damage Reduction Program

# Hydrotechnical Study of the Stephenville Area

Volume 1 of 2

## Main Report

February, 1984




**Nolan Davis & Associates Limited**

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| | |
|---|---|
| ACLS | Area controlled by lakes and swamps (%) |
| AMC II | Average Antecedent Moisture Condition |
| AMC III | Saturated Antecedent Moisture Condition |
| b | Watershed unit hydrograph parameter |
| B | Surveyed bridge section |
| BBK | Field Surveyed cross-section (Blanche Brook) |
| C | Surveyed culvert section |
| C.K. | Coefficient of kurtosis |
| CL | Confidence limit |
| CMP | Corrugated metal pipe |
| cms | Cubic metres per second ($m^3/s$) |
| CN | Average Soil Cover Complex Number |
| CNFDRP | Canada-Newfoundland Flood Damage Reduction Program |
| C.S. | Coefficient of skew |
| d/s | Downstream |
| DA | Drainage area ($km^2$ or $mi^2$) |
| f(t/water) | flow capacity is a function of tailwater elevation |
| g | Acceleration due to gravity ($m/s^2$) |
| h | Depth of flow (m) |
| i | Year |
| k | Frequency factor |
| K | Hydrograph recession parameter (hrs) |
| L | Watershed length (km or mi) |
| LN | Log Normal probability distribution |

| | |
|---|---|
| LP3 | Log Pearson Type III probability distribution |
| M | Snowmelt (mm) |
| $M_1$ | Logarithmic transformed value of mean - short term station |
| $M_2$ | Logarithmic transformed value of mean - long term station |
| MAF | Mean Annual Flood ($m^3/s$) |
| MAR | Mean Annual Runoff (mm) |
| $M_{cc}$ | Melt due to condensation and convection (mm) |
| $M_g$ | Melt due to heat transfer from the ground (mm) |
| $M_p$ | Melt due to rainfall (mm) |
| $M_{rl}$ | Melt due to long wave radiation (mm) |
| $M_{rs}$ | Melt due to short wave radiation (mm) |
| n | Manning's roughness coefficient |
| N | Number of years |
| P | Precipitation amount (mm or in) |
| Q | Amount of runoff (mm or in) |
| $q_p$ | Unit hydrograph peak flow rate ($m^3/s$) |
| $Q_p$ | Annual maximum instantaneous peak flow ($m^3/s$) |
| $\overline{Qp}$ | Mean annual maximum instantaneous peak flow ($m^3/s$) |
| QP | Annual flood peak ($m^3/s$) |
| $QP_{100}$ | Maximum Instantaneous 1:100 year recurrence interval flow rate ($m^3/s$) |
| $QP_{20}$ | Maximum Instantaneous 1:20 year recurrence interval flow rate ($m^3/s$) |
| $QP_N$ | Cumulative moving mean at year n |
| $QP_T$ | Derived design flood peak at selected locations in the study area for return period T-years ($m^3/s$) |
| r | Coefficient of Correlation |
| R | Hydraulic radius |
| s | $\frac{1000}{CN} - 10$ |

| | |
|---|---|
| SD | Standard deviation |
| $SD_1$ | Logarithmic transformed value of standard deviation - short term |
| $SD_2$ | Logarithmic transformed value of standard deviation - long term |
| $S_f$ | Boundary frictional effect |
| SHAPE | Watershed shape parameter (1/km) |
| SLOPE | Slope of watershed (%) |
| SLP | Slope of watershed (m/km or ft/mi) |
| $S_o$ | Bottom channel slope (m/m) |
| $SQ_p$ | Standard deviation of maximum instantaneous peak flow series |
| $T_p$ | Hydrograph time to peak parameter (hrs) |
| 3PLN | 3 Parameter Log Normal probability distribution |
| u/s | Upstream |
| v | Velocity in direction of flow (m/s) |
| W | Surveyed dam or weir section |
| WC | Field surveyed cross-section (Warm Creek) |
| x | Distance in direction of flow (m) |
| $\mu y$ | Logarithmic transformed value of mean |
| $\sigma y$ | Logarithmic transformed value of standard deviation |
| $Y_T$ | Logarithmic transformed flood estimate |

# Acknowledgements

The information and conclusions presented in this report were derived with assistance from several individuals and organizations.

The following members of the Stephenville Project Committee provided significant input and direction throughout the study:

| | |
|---|---|
| - Dr. W. Ullah | Government of Newfoundland and Labrador Department of Environment |
| - Ms. E. Langley | Inland Waters Directorate, Environment Canada |
| - Mr. R. Picco | Government of Newfoundland and Labrador Department of Environment |

The background information presented in this report was obtained from several sources including personnel from the Atmospheric Environment Services; the Water Survey of Canada; the Water Planning and Management Branch and the Tides and Water Levels Branch - all of Environment Canada. Background data collected by the Water Resources Branch of Environment Newfoundland also proved most useful during these investigations.

The field work was supervised and undertaken by Mr. B. Davis, Mr. W. Pye and Mr. I. Wiseman respectively. The office studies were undertaken by Mr. C. Jarratt and Mr. S. Smith and supervised by Mr. H. Belore.

We would also like to express our appreciation for the time and effort of all others who contributed to this project by way of information, discussions and otherwise.

# Executive Summary

## Introduction

Blanche Brook is a relatively small stream with a watershed area of 118.6 $km^2$, including Warm Creek, its major tributary (sub-watershed area of 53.2 $km^2$). Throughout its history the Town of Stephenville has experienced periodic flooding and associated economic losses in developed low-lying areas adjacent to these streams. Additional development in flood prone areas has occurred in recent years, and there is continuing pressure for further development of floodplain lands in the Stephenville area.

On May 22, 1981, the Province of Newfoundland and the Government of Canada entered into a General Agreement Respecting Flood Damage Reduction; recognizing that the potential for future flood damages can be reduced by controlling the use of areas prone to flooding. The primary purpose of this Hydrotechnical Investigation of the Stephenville area was to determine flood discharge and associated water levels and flood prone areas for the 1:20 and 1:100 recurrence interval flood events. A secondary objective was to identify possible flood remedial measures for future investigation. The extent of the study area for the hydraulic and floodplain mapping investigations extended along the floodplain and channel of Blanche Brook and Warm Creek from their outlets to points just upstream of the Hansen Highway.

## Main Findings

Computer simulation techniques were utilized in order to estimate the peak flow rates and associated flood levels which could be expected to occur on the average of once in every twenty and one hundred years. The influence of various hydrologic and hydraulic factors on flood levels was also examined by means of sensitivity testing. The following points briefly summarize the main findings of the hydrotechnical investigations:

1) Peak flows at the outlet of Blanche Brook were estimated to be 112.0 $m^3/s$ and 166.5 $m^3/s$ respectively for the 1:20 and 1:100 year flood events. The accuracy of these estimates was verified by use of field observations and by comparison to several secondary techniques.

2) The backwater model was found to give acceptable estimates of flood profiles compared to conditions observed during the course of the investigations. The 1:100 and 1:20 flood levels were then determined and were plotted on new topographic maps produced at a scale of 1:2500 for the study area. The main flood hazard areas were then identified as follows:

   i) Blanche Brook
      - outlet of Blanche Brook up to Main Street
      - Main Street and Massachusetts Drive area
      - area south of Whites Avenue

   ii) Warm Creek
      - Carolina Avenue to Noels Pond
      - large flooded areas upstream of Noels Pond
      - overtopping of highways upstream of Noels Pond
      - Community of Noels Pond

3) Flood levels in the study area were found to be most sensitive to the presence of ice or debris jamming in the watercourse during the occurrence of peak flows. Historically, some blockage has been observed at the following locations:
      - Minnesota Drive Bridge
      - Main Street Bridge
      - Mississippi Drive Bridge
      - Culverts under Route 490
      - Route 460 Bridge

It was found that blockage of culverts and bridges could locally increase "open water" flood levels by 0.90 to 1.2 m depending on location.

Main Recommendations

1) The 1:20 and 1:100 year flood profiles and associated flood plains as delineated on topographic maps at a scale of 1:2500 should be adopted and utilized for future regulation of development along Blanche Brook and Warm Creek.

2) The potential for significant flooding (and associated ice jams) exists mainly at man-made flow restrictions in the study area. The structures located at Main Street, Carolina Avenue, Mississippi Drive and Route 490 should be enlarged to reduce the potential for future ice and debris jam formation. In the interim, it is strongly recommended that a maintenance program should include removal of any accumulated ice or debris from the channel and floodplain.

3) A system of flood control berms or dykes could be constructed immediately downstream of Main Street on Blanche Brook and to the west of Warm Creek downstream of Mississippi Drive. This could include assessment of the possibility of raising part of Minnesota Drive to act as a dyke to protect adjacent flood prone areas.

4) Additional feasibility investigations should be undertaken in order to determine the cost-benefit ratio for possible structural flood measures along Blanche Brook and Warm Creek.

The main report and associated appendices and supplementary report describe in detail the methodology and findings of the hydrotechnical investigations.

# Introduction 1.0

# Introduction 1.0

## 1.1 General

Historically, the development of urban centres in many areas of Canada including Newfoundland has taken place on flood prone lands. These lands were developed by the first settlers because of their agricultural productiveness and, in some cases, so the the river could be utilized as the main transportation route. These early uses of the floodplain have evolved into present day highly urbanized communities which still attempt to utilize floodplain lands. An increasing trend towards urban developments in Canada has resulted in an increased potential for higher flood losses. A nation-wide survey of potential flood hazards (10)* has indicated that more than 200 communities in Canada have some developments located in flood hazard areas. In particular, the serious consequences of flooding in Newfoundland and more specifically, in the Stephenville area have been well documented in a number of reports (22, 27)

There is continuing pressure to develop additional lands in the Stephenville floodplain areas, as evidenced by recent attempts to provide some form of flood and erosion control along some reaches of the channel. The development pressures have led to an increased potential for future flood losses in the Blanche Brook and Warm Creek floodplain. However, structural measures to provide protection are very costly to construct and do not provide absolute protection from flood damages. In addition, structural measures to provide flood protection tend to invite additional development in the floodplain. Controls to prevent development in the flood prone areas may be a more desirable means for reducing the potential for increased flood losses in the future.

* Note: (10) Number(s) in brackets denote sources given in the list of references.

On May 22, 1981, the Province of Newfoundland and the Government of Canada entered into a General Agreement Respecting Flood Damage Reduction. The main objective of this Agreement is to reduce the potential for flood damages in floodplains and along the shores of lakes, rivers and the sea. This Agreement also recognizes that the potential for future flood damages can be reduced by controlling the areas prone to flooding.

The General Agreement Respecting Flood Damage Reduction allows the two levels of government to enter into a number of other agreements on specific aspects of flood damage reduction, including but not limited to, land use planning, flood proofing, flood risk mapping, flood forecasting, flood control works and flood studies.

To provide for the identification and delineation of flood prone areas in Newfoundland, the "Agreement Respecting Flood Risk Mapping" was also signed on May 22, 1981. Under the terms of this agreement, a number of flood prone areas in the Province are to be mapped and flood risk zones delineated and ultimately designated as areas where the Federal and Provincial governments will agree to restrict their funding of new development. These agreements were amended in May, 1983 and a related "Studies Agreement" was signed in June, 1983. (In this report, projects completed under these agreements are referred to as work done under the Canada Newfoundland Flood Damage Reduction Program; CNFDRP for short.)

The Town of Stephenville was considered to be of high priority in regard to the potential for reducing future flood losses and is the first area in the province to be studied in detail under this program. Subsequent to the findings of this investigation, future work may be required to provide structural measures for flood damage reduction and/or to provide regulations for preventing future development in flood prone areas.

The primary purpose of the present study was to determine flood discharge and water levels and to identify flood prone areas and possible remedial measures along Blanche Brook and Warm Creek.

## 1.2 Authorization and Scope of Study

The agreements previously mentioned provide for the establishment of two committees; the Steering Committee which is responsible for general administration of the agreements and the Technical Committee which provides technical support to the Steering Committee. On September 15, 1982, Nolan-Davis & Associates Limited, in association with Cumming-Cockburn & Associates Limited were commissioned by the Newfoundland Department of Environment on behalf of the Steering Committee to undertake a "Hydrotechnical Study of the Stephenville Area. As described in the Terms of Reference, the main objective of this investigation was to develop the 1:20 and 1:100 year return period flood hydrographs and associated backwater profiles for the study area.

The following points summarize the overall scope of the investigations:

1. Review of background information to characterize the flooding problem
2. Evaluate the significance of various factors affecting flooding in the Stephenville area
3. Design, coordinate and manage a field program for the purpose of collecting hydrologic and hydraulic data for model calibration and validation
4. Determine 1:20 and 1:100 year recurrence interval open water flood events and backwater profiles from:
    - the confluence of Blanche Brook at St. Georges Bay of the Gulf of St. Lawrence, to the Hansen Hwy (Route 460);
    - the confluence of Warm Creek with Blanche Brook upstream to the Community of Noels Pond
5. Produce the 1:20 and 1:100 year flood profiles and plot the 1:20 and 1:100 year return period flood lines on topographic maps to determine the areal extent of flood prone areas
6. Undertake sensitivity analyses of peak flow estimates and backwater profiles

7. Assess the significance of ice jamming, debris jamming and other hydraulic factors affecting flood lines
8. Identify possible remedial measures for flood management and flood damage reduction which may be analysed as required in possible future phases of the flood hazard investigations.

The scope of the study is described in detail in the Terms of Reference (12).

## 1.3 Study Area Description

The general location of the study area is depicted on Figure 1.1. The Stephenville area watershed is drained primarily by two watercourses flowing in a southerly direction; Blanche Brook and Warm Creek. Blanche Brook is a relatively small stream (drainage area, including Warm Creek, of 118.6 $km^2$ to its outlet) flowing through the Town of Stephenville. The stream originates in the highlands north of the Town and flows southerly along a moderately steep channel to discharge into St. George's Bay in the Gulf of St. Lawrence at Stephenville. Warm Creek is a major subwatershed of Blanche Brook. This stream is characterized as a relatively small brook (drainage area of 53.2 $km^2$ to its outlet). The stream originates in the highlands north of Stephenville and flows southerly to discharge into Noels Pond, where it changes course to drain primarily westerly until reaching the Town of Stephenville, where it resumes a more southerly course.

The analysis of hydrologic and climatologic characteristics encompasses the entire Blanche Brook watershed in order to determine the hydrologic response of the watershed to specific storm events. Hydraulic investigations were undertaken along the floodplain and channel of Blanche Brook and Warm Creek from their outlets to points just north of the Hansen Hwy (Route 460). The watershed and limit of flood profiles considered in this investigation are shown on Figure 1.2.

Within the study reaches, several areas have been identified as having a high flood potential (22, 27). One such major floodplain is located between the Noels Pond reservoir and the community of Noels Pond on Warm Creek. The other main area consists of the stream reaches south of Main and Carolina Streets, especially in the areas of the old Labatt Brewery and the Canadian Tire store in the Town of Stephenville.

While there has been limited development of the floodplain in recent years, some single family homes, a large shopping mall, government offices and several business establishments have been located and/or upgraded near the streams.

The trend toward greater development in the future is also apparent, and several large tracts of land have been under consideration for development. Reclamation of interval land has also been accomplished by land filling in flood prone areas. With development over the years, a number of access structures have been installed over the study watercourses to facilitate travel in the Town, and some of these create impediment to flow resulting in increased backwater flooding. Some rip-rap and gabion work for erosion protection has also been installed along some channel reaches (see Section 4.2.2).

There has also been appreciable development in the community of Noels Pond. This is largely a residential community with homes being built in relatively close proximity to the stream. In many cases, these locations appear to be prone to flood damage under high water conditions.

Other areas of the watershed are largely undeveloped and are comprised of steeply sloping topography with extensive forest cover. (Some forested areas have been cut over - see discussion of land use in Section 3.3.4.) Historical flood problems have been confined mainly to developed areas along the main channels.

## 1.4 Overview of Study Methodology

As indicated previously, the basic purpose of this investigation was to provide the 1:20 and 1:100 year open water flood profiles and floodplain extent for Warm Creek and Blanche Brook. The accurate determination of flood profiles along the study reach depends on several hydrological and hydraulic factors, including the following:

- historical flood conditions in the study area
- climatological characteristics of the Blanche Brook watershed; including rainfall and snowmelt characteristics
- land use in the watershed including degree of urbanization, etc.
- peak discharge rates associated with the 1:20 and 1:100 year return period floods
- effects of tides, ice jams, debris jams and other hydraulic factors along the study reach
- existing stream channel and floodplain hydraulic characteristics and man-made changes such as bridge and channel constrictions, berms and dyking, etc.
- natural and artificial flood storage in the study area

The complex interrelationships between the above mentioned factors have been considered in the course of undertaking our climatologic, hydrologic and hydraulic investigations.

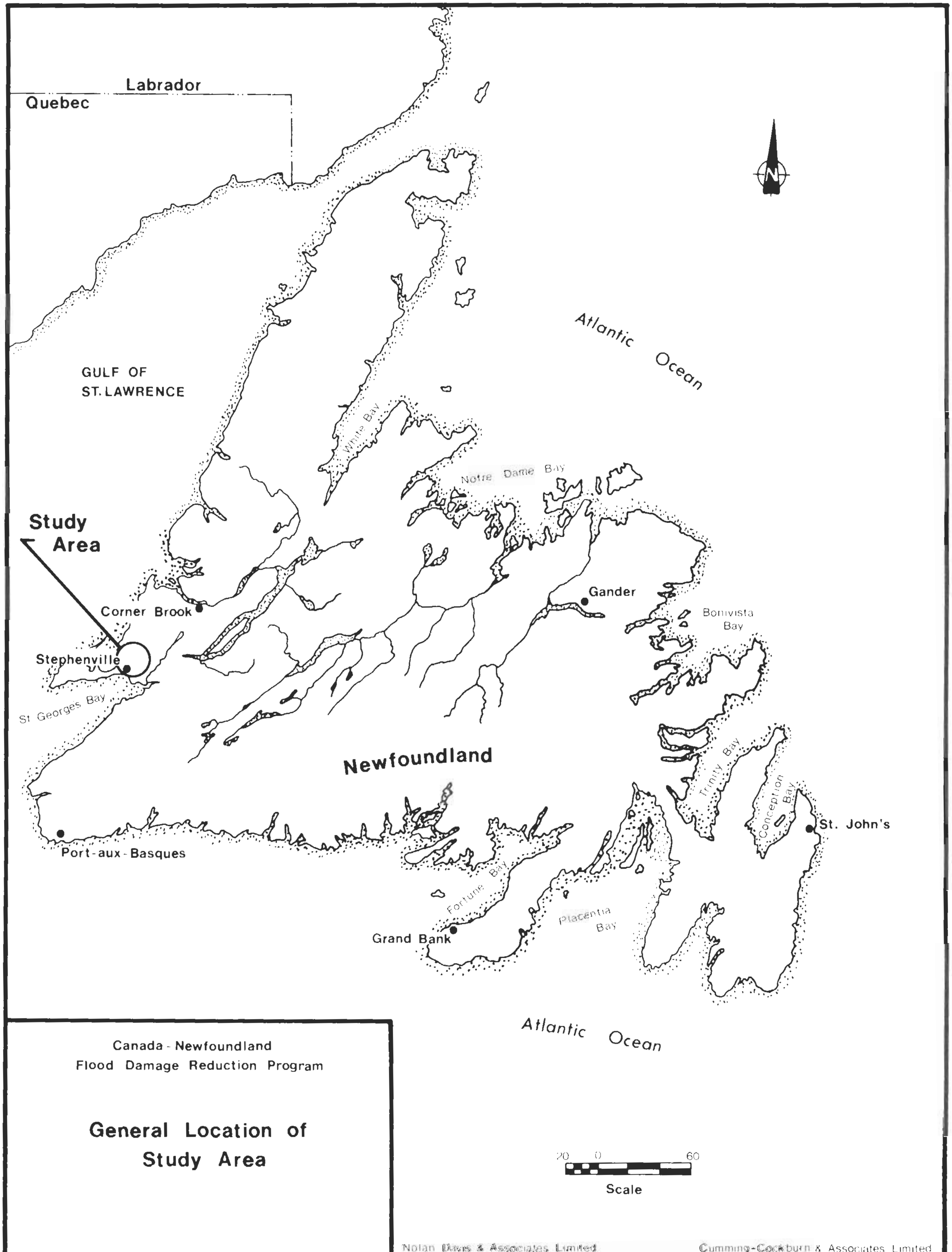
The first step in the investigation was the collection and review of available background information and existing data on climatologic and flood characteristics. These are discussed in Chapter 2.0, entitled "Background Information".

The next step was the determination of appropriate 1:20 and 1:100 year peak discharge rates. Alternative estimates by statistical and deterministic analyses were derived and compared as discussed in Chapter 3.0, "Hydrologic Analyses". Where appropriate, this included model calibration and sensitivity and error analyses in order to achieve an appropriate level of accuracy for the discharge estimates.

Thirdly, the peak flow estimates were converted to flood water levels (profiles) along the study reaches by means of a computer model of hydraulic characteristics. This is discussed in Chapter 4.0, including sensitivity testing of the most important hydraulic parameters and relevant model calibration and verification using documented events.

Finally, the approximate extent of the flood hazard area was plotted on copies of new topographic mapping for the lower part of the study area. It was then possible to identify potential remedial measures which might be considered in more detail in the future for alleviating the potential for future flood losses. These measures are identified in Chapter 5.0.

As required by the Terms of Reference (12), the "Hydrologic and Hydraulic Procedures for Floodplain Delineation" (19) and "Survey and Mapping Procedures for Flood Plain Delineation" (26), developed by Environment Canada were used as basic guidelines throughout the course of these investigations.




Canada - Newfoundland
Flood Damage Reduction Program

**General Location of Study Area**

**Figure 1.1**

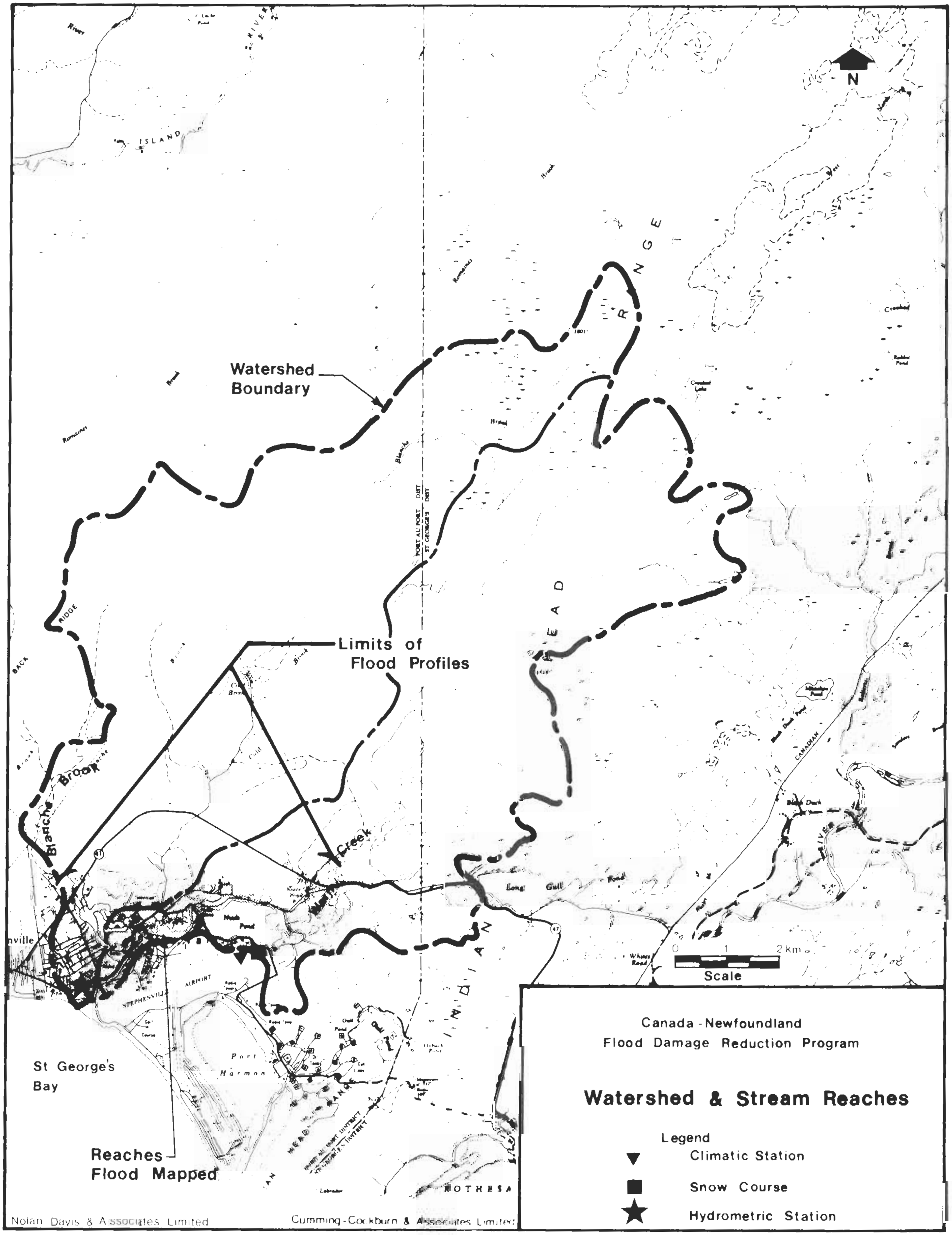

N
Watershed Boundary
Limits of Flood Profiles
Blanche Brook
Creek
Long Gull Pond
St George's Bay
Port Harmon
Reaches Flood Mapped
0 1 2 km
Scale
Canada - Newfoundland
Flood Damage Reduction Program
Watershed & Stream Reaches
Legend
Climatic Station
Snow Course
Hydrometric Station
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Figure 1.2

# Background Information 2.0

# Background Information 2.0

## 2.1 General

The Town of Stephenville has experienced periodic flooding throughout its history. A number of investigations documenting flood conditions in the Stephenville watershed have been completed in recent years, and have identified that the predominant cause of flooding has been excessive rainfall during the spring months, leading to rapid snowmelt and ice/debris jams at various man-made constrictions.

Existing general studies on peak flows for the island portion of the Province (13, 38) can be used to give an overall assessment of flood conditions in the area. The most recent of these studies was one of the first projects to be carried out under the Canada-Newfoundland Flood Damage Reduction Program (13) and supercedes the previous investigation (38). These peak flow estimates are discussed in more detail in Section 3.0.

Subsequent to severe flooding which occurred as the result of an autumn rainstorm in 1973, a report entitled, "An Evaluation of Flooding in Stephenville, Newfoundland" was completed by Environment Canada (22). This included peak flow estimates and determination of flood profiles culminating in the preliminary development of the aerial extent of the mean annual and 1:100 year flood plain. Various structural adjustments, ranging from regional alternatives such as upstream storage, floodways, and diversions, to more localized alternatives such as dykes and flood-proofing were discussed. Those measures which appeared to be promising were subjected to conceptual design. An analysis of the cost of these measures and their potential to reduce the physical extent of the hazard was also discussed. Non-structural measures were assessed on a similar basis, although the discussion was substantially more qualitative in nature, compared to the assessment of possible structural modifications.

A report on historic flooding in Newfoundland was recently completed by Environment Canada (25), which includes a perspective on flooding in the Stephenville area. The general findings in regard to historical flooding in the study area are summarized briefly in Table 2.1. The existing studies on the hydrologic, hydraulic and climatic characteristics of the Blanche Brook and Warm Creek basins have confirmed that a high potential exists for future flood losses in the study area.

## 2.2 Summary of Historical Flood Conditions

Documentation on historical floods on Blanche Brook and Warm Creek covers the period 1948 to 1982. Table 2.1 presents a summary of flood susceptible areas along Blanche Brook and Warm Creek, together with an indication of the extent of damage and the cause of flooding. (Additional discussions on historical flooding in the Stephenville area are provided in Appendix A.)

The earliest records of flooding along the study watercourses suggest that all flood damages usually occurred in the vicinity of Main Street and Carolina Avenue and in the vicinity of Noels Pond. It is these areas which continue to experience problems during flood events. Historically, most damage producing floods have taken place during the occurrence of the spring runoff. However, perhaps the most severe flood conditions occurred during the autumn of 1973 when mild temperatures and heavy rain (69.9 mm) over a 3-day period resulted in extensive flooding at several locations in the watershed.

The review of background information has also revealed that some development of the floodplain has occurred during the past twenty years. Recent records of historical flooding suggest that major floods or significant flood damage have occurred over the last 10-20 years, with an increased frequency of damage claims over the last ten years (25).

Furthermore, long-time residents of the area believe that flooding has become more significant in recent years, corresponding to the increased logging operations in the upstream study reaches allowing more debris to accumulate in the channels.

The approximate locations of flood sensitive areas are depicted graphically on Figure 2.1.

## 2.3 Existing Data

Field surveys of channel and floodplain characteristics were carried out in the study area by Environment Canada (27). This included the collection of channel and floodplain cross-sections, determination of the stream channel profiles, and measurements of bridges, culverts and hydraulic structures. However, due to possible sedimentation and erosion processes and the resulting changes in cross-sectional properties since the time of the survey, it was necessary to update this information as part of the present investigations, as described in Section 4.2.1.

For example, a comparison of cross-section measurements from these surveys (undertaken in 1974 and 1982 respectively) indicates that some erosion and sedimentation has occurred in the reaches downstream of the confluence of Warm Creek with Blanche Brook. However, it is man-made alterations to the floodplain in terms of dyking and berming of the channel banks that have most significantly altered the hydraulic configuration of the watercourses in recent years.

Some data describing local hydrologic and climatologic characteristics were also available from several agencies. Background data on streamflow, water levels, tidal levels, rainfall, snowfall, snow accumulation and ice cover characteristics were also collected and reviewed as discussed briefly in the following paragraphs.

Streamflow records within the study area are collected by the Water Survey of Canada for the hydrometric station located on Blanche Brook downstream of the confluence with Warm Creek (Gauge No. 02YJ002)* (23, 24). This data is available for the period July, 1978 up to date, and is published as mean daily discharge. A summary characterizing available discharge measurements is presented in Table 2.2. Additional discharge measurements are also available for nearby streams. For example, discharge measurements have been taken by the Water Survey of Canada on the Harrys River below Highway Bridge since April, 1968 (Gauge 02YJ001). Water level records and rating tables are also available at these gauge locations.

Some information on the influence of tides on water levels is also available near the confluence of Blanche Brook with St. Georges Bay. Tidal information is published under the authority of the Canadian Hydrographic Services, Department of Fisheries and Oceans (30). St. Georges Bay (Port Harmon) is referred to as a secondary tidal port where tidal information is given in relation to the primary tidal port at Harrington Harbour on the eastern coast of the Gulf of St. Lawrence. Relevant tide characteristics are summarized in Table 2.3.

Background information describing climatic characteristics of the Blanche Brook watershed in the Stephenville area is available from the Atmospheric Environment Service and other agencies such as Newfoundland Department of Environment (2, 3, 4, 5, 6, 9, 28). Meteorologic data is measured at the Stephenville Airport location as shown on Figure 1.2. The data collected is comprised mainly of daily precipitation and temperature records (for 38 years of record); however, continuous measurements, for which a total of 13 years of data is available to date, are also collected at the station. A summary of monthly average temperature and precipitation characteristics for Stephenville is given in Table 2.4. Additional information on rainfall intensities and frequency analysis are discussed in Appendix C. While no climatic data are published for the upper areas of the watershed, the available data base was considered to be suitable for the purposes of this investigation.

* Hydrometric stations are operated under the Canada-Newfoundland Hydrometric Survey Agreement

In addition, snow course measurements are collected by Environment Canada (29) only at the Stephenville Airport, at the location indicated on Figure 1.2. To date, snow course data have been collected for the winters of 1967 to 1982. These data are collected approximately every two weeks during the months of December through May. A summary of available snow cover data for the study watershed is presented in Table 2.5. Snow cover amounts were noted as being highly variable, yet frequently in excess of 90 cm depth.

While little information on ice characteristics is available for the study watercourses, some data on extent of ice cover is available through the Water Survey of Canada. Similarly, historical documentation of ice related flood events on the study channels is available through local agencies and Newfoundland Department of Environment . However, this data is sparse and tends to be available only in general terms such as photographs of some ice related flood events and newspaper articles commenting on ice jams. (Additional data on ice characteristics and ice thickness should be collected in the future.)

TABLE 2.1

Summary of Recent Flood Problems

On Blanche Brook and Warm Creek

| Watercourse | Year | Documented Flooding and/or Flood Damages |
|---|---|---|
| Warm Creek | Sept. 1948 | - Harmon Air Force base blanketed by lakes of water |
| | May 1969 | - Mississippi Drive bridge overtopped & footings eroded<br>- overbank flooding about the Carolina Ave. bridge |
| | May 1972 | - debris jamming at Mississippi Drive caused the structure to be washed out<br>- Carolina Bridge threatened and temporary dykes constructed<br>- section of earth fill (10'-15') washed out when Missouri Drive overtopped |
| | Aug. 1973 | - nightclub washed out above Noels Pond<br>- four families forced to evacuate homes near Noels Pond (±4-6 in. water on floor)<br>- $21,000 spent to repair washed out watermain<br>- $48,000 damages to CNR lines and flood related problems near Noels Pond |
| | Dec. 1977 | - Noels Pond area flooded as a result of debris blocking bridges and culverts |
| | Apr. 1982 | - extensive basement flooding in community of Noels Pond industrial access route (Rte 490) blamed for backing up<br>- water |
| | June 1983 | - flooding in community of Noels Pond<br>- minor basement blooding |
| Blanche Brook | May 1969 | - residential area u/s of Main Street flooded (several basements flooded)<br>- airport area, curling rink & brewery field flooded<br>- localized areas of erosion |
| | Feb. 1973 | - ice piled up across Main St. bridge<br>- sidewalk had to be replaced<br>- Manpower office and Harmon complex slightly damaged by ice<br>- buildings on Main St. had parking lots blocked and water rose to almost floor level at the Co-Op and flooded Minnesota Dr. opposite brewery |
| | Aug. 1973 | - water line broken due to high water & debris<br>- 1.5' water on brewery floor when temporary dyke broke<br>- $70,000 damage to brewery |
| | Nov. 1974 | - debris blocking at Carolina Ave. and Hansen Hwy. |
| | March 1979 | - 80 ft. section of retaining wall felled by high flows and ice<br>- $100,000 to $120,000 damages to replace cribbing |
| | July, 1979 | - Hansen Hwy. bridge jammed by debris<br>- centre pier of bridge moved by debris<br>- basement flooding and Harmon Convenience Store on on Caroline Ave. foundered<br>- Hansen Hwy. bridge subsequently replaced in 1982 at cost estimated in the order of hundreds of thousands of dollars |
| | Apr. 1982 | - access flooding in Park Ave.-White Trailer Court area no damages claimed |

Source: Reference (25)

TABLE 2.2

Summary of Available Discharge Measurements

Blanche Brook 02YJ002

Period of Record - 1978 to 1981

| | Jan | Feb | Mar | Apr | May | Jun | July | Aug | Sept | Oct | Nov | Dec | Year |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Mean Discharge ($m^3/s$) | 4.23 | 2.25 | 5.69 | 8.23 | 6.58 | 2.99 | 8.84 | 2.41 | 4.16 | 18.7 | 5.98 | 3.50 | 5.58 |
| *Max. Daily Discharge ($m^3/s$) | 31.1 | 11.7 | 62.9 | 50.9 | 25.8 | 19.3 | 51.8 | 13.6 | 22.7 | 33.1 | 36.8 | 32.2 | 62.9 |

NOTES: * Max. Daily Discharge Recorded for Period of Record
(See Appendix B for instantaneous flows)

Source: Reference (23)

TABLE 2.3

Tidal Characteristics

| | Primary Tide Gauge Harrington Harbour (m)* | Secondary Tide Gauge Port Harmon (m)* |
|---|---|---|
| Mean High Tide ** | 1.8 | 1.35 |
| Maximum High Tide *** | 2.2 | 1.6 |
| Maximum Recorded Tide **** | 2.9 | 2.3 |
| Reference Station/ Index Number | 2550 | 2710 |

NOTES: * Elevations expressed are related to local chart datum. For approximate conversion to geodetic at Port Harmon subtract the mean water level of 0.82 m.

** Mean High Tide is the mean tide for Higher High Water

*** Maximum High Tide is the mean large tide for higher high water

**** Maximum Recorded Tide is the recorded extreme for highest high water

Source : Reference (30)

TABLE 2.4

Summary of Temperature and Precipitation

for Stephenville Airport*

| | Jan | Feb | Mar | Apr | May | Jun | Jul | Aug | Sept | Oct | Nov | Dec | Year |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Mean Temperature (°C) | -5.0 | -6.2 | -2.8 | 1.8 | 6.9 | 11.9 | 16.0 | 16.1 | 11.9 | 7.0 | 2.9 | -2.6 | 4.8 |
| Mean Max. Temperature (°C) | -1.6 | -2.3 | 1.0 | 5.2 | 11.0 | 16.1 | 19.8 | 19.8 | 15.7 | 10.3 | 5.8 | 0.4 | 8.4 |
| Mean Min. Temperature (°C) | -8.3 | -10.0 | -6.6 | -1.7 | 2.8 | 7.8 | 12.2 | 12.3 | 8.2 | 3.7 | 0.0 | -5.5 | 1.2 |
| Years of Record | 38 | 38 | 39 | 36 | 39 | 38 | 39 | 39 | 38 | 38 | 39 | 38 | -- |
| Mean Total Precipitation (mm) | 115.4 | 89.9 | 81.4 | 59.5 | 80.6 | 86.3 | 96.1 | 104.1 | 104.5 | 111.6 | 123.0 | 114.1 | 1166.5 |
| Mean Snowfall (cm) | 95.2 | 76.0 | 58.5 | 22.0 | 4.2 | 0.0 | 0.0 | 0.0 | T** | 3.6 | 24.4 | 80.7 | 364.6 |
| Years of Record | 38 | 38 | 39 | 36 | 37 | 38 | 39 | 39 | 37 | 37 | 39 | 38 | -- |

NOTES: * Period of Record based on date of station installation to 1980
** T : Trace

SOURCE: Reference (18, 28)

TABLE 2.5

Summary of Available Snow Cover Data for Stephenville Airport for Period of Record*

| WINTER | DECEMBER | | | JANUARY | | | FEBRUARY | | | MARCH | | | APRIL | | | MAY | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Day | Snow Depth (cm) | Water Equiv. (mm) | Day | Snow Depth (cm) | Water Equiv. (mm) | Day | Snow Depth (cm) | Water Equiv. (mm) | Day | Snow Depth (cm) | Water Equiv. (mm) | Day | Snow Depth (cm) | Water Equiv. (mm) | Day | Snow Depth (cm) | Water Equiv. (mm) |
| 1967-68 | | | | 1 | T | 0.0 | | | | | | | | | | | | |
| | | | | 15 | 65 | 17.3 | | | | | | | | | | | | |
| 1968-69 | 1 | 20 | 28 | 1 | 26.9 | 68.6 | 1 | 37.1 | 122 | 1 | 32.3 | 117 | 1 | 41.7 | 150 | 1 | T | 0.0 |
| | 15 | 27 | 96 | 15 | 49.5 | 109 | 15 | 31.2 | 109 | 15 | 41.6 | 157 | 15 | 46.7 | 175 | | | |
| 1969-70 | | | | 1 | 15.0 | 17.8 | 1 | 50.5 | 96.5 | 1 | 55.1 | 140 | 1 | 62.2 | 221 | 1 | T | 0.0 |
| | | | | 15 | 42.9 | 48.3 | 15 | 26.4 | 78.7 | 15 | 67.3 | 203 | 15 | 54.1 | 221 | | | |
| 1970-71 | 1 | 0.0 | 0.0 | 1 | 32.5 | 76.2 | 1 | 71.6 | 175 | 1 | 55.6 | 190 | 1 | 66.5 | 201 | 1 | 0.0 | 0.0 |
| | 15 | 32.0 | 45.7 | 15 | 56.1 | 114 | 15 | 47.2 | 157 | 15 | 62.0 | 198 | 15 | 29.5 | 102 | | | |
| 1971-72 | 1 | 17.5 | 30.5 | 1 | 65.5 | 140 | 1 | 74.7 | 203 | 1 | 83.3 | 239 | 1 | 69.3 | 229 | 1 | 51.3 | 122 |
| | 15 | 49.3 | 55.9 | 15 | 62.0 | 178 | 15 | 67.1 | 208 | 15 | 94.2 | 256 | 15 | 72.4 | 246 | 15 | 28.4 | 99.1 |
| 1972-73 | 15 | 51.3 | 96.5 | 1 | 44.2 | 112 | 1 | 99.1 | 221 | 1 | 122 | 315 | 1 | 99.1 | 315 | 1 | 38.6 | 150 |
| | | | | 15 | 73.4 | 175 | 15 | 99.8 | 318 | 15 | 124 | 330 | 15 | 91.7 | 320 | 15 | 4.3 | 15.2 |
| 1973-74 | | | | 1 | 19.0 | 27.9 | 1 | 58.4 | 127 | 1 | 73.9 | 178 | 1 | 99.6 | 295 | 1 | 63.7 | 188 |
| | | | | 15 | 50.0 | 104 | 15 | 71.1 | 170 | 15 | 98.6 | 264 | 15 | 84.1 | 249 | 15 | 20.1 | 66.0 |
| 1974-75 | | | | 1 | 36.1 | 55.9 | 1 | 70.4 | 140 | 1 | 75.2 | 239 | 1 | 89.2 | 307 | 1 | 17.0 | 68.6 |
| | | | | 15 | 44.2 | 68.6 | 15 | 76.4 | 211 | 15 | 88.9 | 279 | 15 | 62.2 | 231 | | | |
| 1975-76 | 15 | 24.9 | 68.6 | 1 | 37.6 | 63.5 | 1 | 47.2 | 127 | 1 | 80.8 | 185 | 1 | 47.0 | 124 | | | |
| | | | | 15 | 53.1 | 102 | 15 | 56.9 | 152 | 15 | 86.1 | 208 | 15 | 24.1 | 43.2 | | | |
| 1976-77 | Nov 15 | 20.1 | 27.9 | 1 | 61.7 | 140 | 1 | 99.8 | 351 | 1 | 93.0 | 279 | 1 | 61.2 | 180 | 1 | 24.1 | 76.2 |
| | Dec 1 | 20.6 | 25.4 | 15 | 74.9 | 168 | 15 | 110 | 305 | 15 | 89.7 | 287 | 15 | 62.0 | 208 | 15 | P | --- |
| | 15 | 57.6 | 104 | | | | | | | | | | | | | | | |
| 1977-78 | 1 | 17 | - | 1 | 51 | 91 | 1 | 69 | 185 | 1 | 77 | 216 | 1 | 89 | 262 | 1 | 61 | 196 |
| | 15 | 49 | 81 | 15 | 49 | 112 | 15 | 65 | 163 | 15 | 94 | 267 | 15 | 88 | 272 | 15 | 0 | 0 |
| 1978-79 | 1 | 36.3 | 58 | 1 | 52.1 | 160 | 1 | 27.2 | 119 | 1 | 40.6 | 130 | 1 | 0.0 | 0 | | | |
| | 15 | 62.7 | 119 | 15 | 64.5 | 178 | 15 | 48.8 | 132 | 15 | 18.3 | 84 | | | | | | |
| 1979-80 | 15 | 14.7 | 22.9 | 1 | 14.5 | 30.5 | 1 | 84.3 | 160 | 1 | 102 | 231 | 1 | 53.6 | 183 | | | |
| | | | | 15 | 32.8 | 71.1 | 15 | 78.7 | 173 | 15 | 84.1 | 234 | 15 | 30.0 | 112 | | | |
| 1980-81 | 15 | 27.4 | 27.9 | 1 | 35.6 | 76.2 | 1 | 52.6 | 137 | 1 | 26.4 | 78.7 | | | | | | |
| | | | | 15 | 35.8 | 112 | 15 | 34.8 | 117 | 15 | 15.2 | 55.9 | | | | | | |
| 1981-82 | 1 | 12.7 | 20.3 | 1 | 39.1 | 66.0 | 1 | 110 | 264 | 1 | 144 | 442 | 1 | 108 | 396 | 1 | 47.0 | 206 |
| | 15 | T | 0.0 | 15 | 102 | 190 | 15 | 124 | 318 | 15 | 124 | 401 | 15 | 108 | 363 | | | |
| 1982-83 | 1 | 12.9 | 20.3 | 1 | 20.8 | 43.2 | 1 | 35.1 | 68.6 | 1 | 37.6 | 83.8 | 1 | 10.7 | 25.4 | | | |
| | 15 | 34.5 | 40.6 | 15 | 7.1 | 20.3 | 15 | 29.2 | 53.3 | 15 | - | - | | | | | | |

T = Trace
P = Precipitation

NOTES: * Source: Reference (29)

# Hydrologic Analyses 3.0

# Hydrologic Analyses 3.0

## 3.1 General

The 1:20 and 1:100 year recurrence interval peak flows were determined for the Blanche Brook watershed using alternative estimating techniques:

1) Statistical Analyses - Regional Flood Frequency Analysis
   - Single station transfer from nearby watershed
2) Deterministic Analysis - Instantaneous Unit Hydrograph technique

These alternative estimating techniques were used for comparison purposes and in order to assess the reliability and accuracy of the available peak flow estimates.

To provide peak flow estimates, a regional flood frequency analysis was initially applied using procedures developed previously by Poulin (38) and more recently by the Canada-Newfoundland Flood Damage Reduction Program (13).

A reliable flood frequency analysis could not be conducted on the existing data base at the hydrometric station on Blanche Brook (No. 02YJ002) since the period of record is too short (5 years). The transfer of statistical estimates from a nearby watershed (Harrys River, 02YJ001) with a longer hydrometric record was undertaken in an attempt to overcome the short period of record on Blanche Brook.

The deterministic computer modelling technique (more specifically, the HYMO model), was deemed necessary due to possible limitations imposed by the Statistical Analysis (e.g. prediction of flood volume is not possible).

The following sections outline the procedures used in the development and application of these techniques for estimating peak discharges associated with the 1:20 and 1:100 year recurrence interval flood events.

## 3.2 Statistical Analyses

### 3.2.1 Introduction

The hydrometric station (No. 02YJ002) located on Blanche Brook is a continuous type recorder which began operation on July 13, 1978. The available period of record of this station is, therefore, too short for undertaking a reliable single station statistical analysis of peak discharge.

The results of the recently completed Regional Flood Frequency Analysis (13) were initially applied to determine peak flow conditions at various locations along the watercourses. These flows were compared to estimates by Poulin's technique, which is a similar method but is based on a shorter period of record.

Secondary statistical analyses were then undertaken for comparison purposes. Harrys River is located about 15 km east of Blanche Brook. This watershed is gauged at Station No. 02YJ001 using a continuous type recorder which has been in operation since April, 1968. The sample statistics of the Harrys River annual maximum flow series were transferred to Blanche Brook in order to derive an adjusted set of sample statistics for Blanche Brook. These derived sample statistics were then used to obtain peak flow estimates for comparison purposes.

### 3.2.2 Regional Flood Flow Estimates

The results of the recent regional analysis have been utilized in this study to derive peak flow estimates for the instantaneous flood flows on Blanche Brook at the gauge location.

The regression equations developed are based on a single station instantaneous flood frequency analysis at 11 hydrometric stations with at least 10 years of record located in Southern and Eastern Newfoundland. The regression equations are developed in the following form:

$$\log_{10} QP_T = C1 + C2 \log_{10} DA + C3 \log_{10} MAR + C4 \log_{10} ACLS + C5 \log_{10} SHAPE \quad (3.1)$$

where $QP_T$ = T year maximum instantaneous peak flow
C1,C2,C3,C4,C5 = constants (refer to appendix B for specific values)
ACLS = area controlled by lake and swamp (% of drainage area)
SHAPE = (0.28 x basin perimeter) DA (1/km)
MAR = Mean Annual Runoff (mm) over the area
DA = Drainage Area in $km^2$

Various parameter values and additional equation details can be found in Appendix B. It has been found that flood flow estimates calculated by the regional equations are fairly sensitive to parameter errors (especially to the MAR parameter) (13). A detailed discussion of the sensitivity of these equations to the input parameters is presented in Appendix B.

The 1:20 and 1:100 year recurrence interval peak flow estimates calculated using the regression equations are presented in Table 3.1 for the gauge location.

A secondary check of the regression equations was undertaken by comparing estimates by the regional method to single station flood frequency estimates for the nearby Harrys River Watershed. The results of this analysis, presented in detail in Appendix B, tend to indicate that the regression equations are applicable to this region and give reliable approximations of instantaneous flood flows.

Additional verification of the peak flow estimates was also undertaken using the results of a previous flood frequency analysis undertaken by

Poulin (38). This technique for estimating peak flows on ungauged watersheds is similar to the recent regional analysis but is somewhat out of date due to the shorter period of record (data base available was up to 1969) used in developing the prediction equations. In addition, this method requires adjustment of mean daily predictions to instantaneous values. The details of this technique are discussed in Appendix B and the results of our application are compared to the new regional predictions in Table 3.1. This comparison indicates that the peak flows are similar to the new regression estimates which tend to further confirm the estimates by the latter technique.

### 3.2.3. Single Station Statistical Analysis of Harrys River Flood Peaks and Statistical Transfer to Blanche Brook

When measured streamflow data necessary to undertake a reliable flood frequency analysis is inadequate, a statistical transfer technique can sometimes be used to transfer sample statistics from a nearby watershed with sufficient data. The Harrys River gauge was selected for use in this analysis as it is in close proximity to the Blanche Brook watershed and because it is the only watercourse with sufficient hydrometric data available. Statistical tests for trend and persistence and a single station statistical analysis were undertaken on the Harrys River annual data series (see Appendix B). Although the data appeared to be free of trend and persistence, it was found that the 1:20 and 1:100 year peak flow estimates are significantly sensitive to the maximum recorded peak flow (1973 event) as to warrant some caution in attempting a statistical transfer to Blanche Brook (see Appendix B).

In summary, the statistical transfer technique yielded flood estimates comparable to the regional techniques. However, due to the sensitivity of peak flow estimates in relation to the available data series, the results of the statistical transfer analysis were only used for secondary comparison purposes.

### 3.2.4 Summary of Statistical Analyses

The following points briefly summarize the results of the statistical analyses:

1) The available period of record of the discharge data for the Blanche Brook hydrometric station is too short for undertaking a reliable single station flood frequency analysis.

2) Peak flow estimates by the CNFDRP Regional Flood Frequency Analysis technique appear to provide reasonable estimates of instantaneous peak flows for Blanche Brook (see Appendix B and Table 3.1). Estimates by this method should be used for comparison to the deterministic model.

3) Peak Flow estimates by Poulin's Regional Flood Frequency method (38), require conversion of mean daily estimates to instantaneous values. This method is outdated and has recently been replaced by a Regional Flood Frequency Analysis conducted by the Canada-Newfoundland Flood Damage Reduction Program. However, peak flow estimates by Poulin's method tend to confirm the magnitude of the 1:20 and 1:100 year peak flows by alternative estimating techniques.

4) The available peak flow record for Harrys River is free of trend and persistence, and it is, therefore, possible to undertake a standard frequency analysis at this location using the available data base. However, it is evident that the single station analysis on the Harrys River is very sensitive to extreme flows which have occurred recently (e.g. 1973). Therefore, the use of flow transfer techniques from the Harrys River is not recommended.

## 3.3 Deterministic Analyses

### 3.3.1 Introduction

The 1:20 and 1:100 year peak flows were also estimated by using a synthetic unit hydrograph procedure known as HYMO (44). The HYMO model was selected to provide peak flow estimates based on rainfall and snowmelt inputs since the model is capable of taking into account the following factors:

1) Capability to model time characteristics of unit hydrographs
2) Can account for variations across the watershed in soil types and land use characteristics
3) Includes routing components which can account for peak flow attenuation in the channel and floodplain
4) Includes reservoir routing capabilities (e.g. Noels Pond)
5) Can provide distributed flow simulations from several sub-watersheds and tributaries
6) Can account for variation in unit hydrographs from various sub-watersheds
7) Accounts for antecedent soil moisture conditions.

In addition, this model has previously proven capabilities for simulating peak flows in a number of other practical applications for various watersheds with hydrologic characteristics similar to Blanche Brook, including simulation of snowmelt hydrographs (e.g. see Ref. 15).

The input requirements of this simulation technique include both meteorological (rainfall/snowmelt) data and physiographic characteristics (land use, time to peak values, constituent soil characteristics, etc.) of the study area.

The following sections describe the hydrologic procedures used in the development and application of the HYMO model in the determination of design flow estimates for the Blanche Brook watershed. A comparison of the deterministic and statistical estimates is then given in Section 3.4.

### 3.3.2 HYMO Model Structure

In order to transform the meteorological input into runoff hydrographs, the HYMO program uses a synthetic unit hydrograph technique and the Soil Conservation Service rainfall-runoff relationships (41).

The program generates a hydrograph for each selected sub-drainage area of the watershed. The hydrographs, beginning at the upstream end of the basin, are added together and routed downstream until the storm event hydrograph at the outlet of the entire watershed is obtained.

The routing of the simulated hydrograph requires as input, characteristics of the channel and floodplain cross-sections, hydraulic roughness coefficients, and reach slopes and lengths. In order to refine the flood routing computations, rating curves from the preliminary HEC-2 backwater model were input to the model to obtain an accurate representation of the channel routing characteristics.

The reservoir routing effects were also modelled using the HYMO program. The input requirements are the discharge/storage relationship for the reservoir, details of which can be found in Appendix C. Field measurements and available topographic maps were used to obtain these relationships for winter and summer conditions of the reservoir. The only major impoundment in the watershed is Noels Pond which was modelled in this manner.

In order to simulate the hydrologic response of the Blanche Brook watershed, and to provide distributed flow estimates, the area was divided into 8 sub-catchments which included 4 channel routings and one reservoir routing. In doing so, inflows and outflows from Noels Pond could be modelled directly and distributed peak flow estimates could also be determined along the study reach.

The watershed discretization (see Figure 3.1) was established through examination and interpretation of existing 1:12,500 scale mapping of the study area. This mapping was used to measure sub-drainage areas, slopes and land-use characteristics, etc. Figure 3-2 illustrates, in schematic form, the modelling breakdown applied in this analysis.

### 3.3.3 Meteorologic Data

The climate of the Blanche Brook watershed can be characterized as being cool with an average annual temperature of about 4.8°C and an average annual precipitation of about 1166 mm (18, 28) (point measurement of precipitation for Stephenville Airport). Flood conditions in the watershed are generated as a result of rainfall, snowmelt and/or rain on snow conditions. However, severe rainstorm events can also occur resulting in extreme flow conditions.

i) Design Storm Conditions

The design storm pattern used in the deterministic computer model was based on an analysis of recorded events at two meteorologic stations in the Province (St. John's and Gander, Newfoundland) (4). The temporal distribution derived by the Atmospheric Environment Service interpretation of 6-12 hr. rainstorms was also compared with an analysis of severe recorded events which have occurred in the vicinity of the study area (11). It was found that the AES temporal distribution derived for Gander and St. John's gave close agreement to that derived from recorded historical events (see Appendix C). The former analysis would result in slightly more conservative peak flows and, therefore, it is recommended that the AES temporal distribution should be used in this study (see Table 3.2).

For the purposes of this analysis, no areal reduction factors were applied to the rainfall amounts due to the relatively small size of the Blanche Brook watershed. This was based on a review of historical storms which occurred in the Stephenville area and substantiated by previously computed relationships between drainage area and rainfall reduction factors (31).

The 12-hour temporal distribution was used based on an analysis of the time of concentration of the Blanche Brook watershed. From recorded events, it was determined that the time of concentration for the Blanche Brook watershed at the outlet is approximately 10 hours. This was substantiated by applying various empirical equations which yielded comparable values for the time of concentration. Flood simulations using 6-hour and 24-hour storm distributions resulted in lower peak discharge values which also confirms the use of a 12-hour storm distribution.

The total point rainfall was obtained from Rainfall-Intensity-Duration-Frequency (RIDF) curves derived for the Stephenville meteorologic station. The resulting total rainfall for the 1:100 and 1:20 year 12-hour storms was found to be 92.6 mm and 74.6 mm respectively (based on 13 years of data) (see Appendix C).

ii) Snowmelt

Based on an analysis of discharge records (Harrys River and Blanche Brook) and a review of historical flooding events, the predominant cause of flooding on Blanche Brook and Warm Creek has been excessive rainfall during spring months leading to rapid snowmelt runoff events. Floods have also frequently occurred during late summer and autumn months as well, such as in late August, 1973, November, 1974, November, 1977, October, 1978 and July, 1979. The 1973 rainfall event occurred over a 3 day period of time and resulted in the highest flood on record on the Harrys River. Analysis of the flood generating mechanism implies completely saturated conditions for the watershed. On the other hand, the 1973 rainfall was apparently the second highest recorded at the Stephenville Airport meteorologic station. This suggests that shorter more intense rainstorms are more critical for the Blanche Brook.

An analysis of typical snowmelt conditions was undertaken in order to provide some insight into the relative importance of spring and summer flood conditions. The rainfall plus snowmelt for various recurrence intervals was established by analysis of available data for the Stephenville Airport meteorological station. This analysis uses historical recorded meteorologic data to obtain the 1 to 10 day melt plus rainfall totals available for runoff. The algorithm used was derived from an energy budget method developed by the U.S. Army Corps of Engineers for forested areas. Using this algorithm and recorded meteorologic data from Stephenville, the 1:100 year and 1:20 year 24-hour melt plus rain amounts were found to be about 107 mm and 81 mm respectively. During snowmelt sequences, the 24-hour duration was found to give more critical snowmelt conditions in comparison to the 12-hour snowmelt event. (Additional details on the snowmelt algorithm are given in Appendix C).

A secondary analysis using an alternative energy budget equation was also undertaken by calculating the theoretical energy inputs for the spring melt period (April/May) for the Stephenville Airport meteorological station. This secondary check confirmed the snowmelt estimates by the procedure previously discussed. (A discussion of this secondary analysis is presented in Appendix C.)

### 3.3.4 Hydrologic Input Parameters

i) Time to Peak and Recession Constant

In order to simulate the response of a specific watershed to a rainfall event using the HYMO procedures, two unit hydrograph parameters must first be established. They are the time to peak and the recession constant for the unit hydrograph. To facilitate calibration of these parameters to the available streamflow records on the watercourse, both parameters were first derived using various empirical equations developed for the HYMO model (44). (See Appendix C for details.)

However, since the available empirical equations were not originally derived for Canadian watersheds and hydrologic conditions, it is necessary to modify the unit hydrograph parameters to reflect local runoff characteristics. These initial estimates of unit hydrograph characteristics were modified by calibration of the HYMO model in order to more accurately reflect the specific characteristics of the watershed. Available streamflow hydrographs were deconvoluted to derive the unit hydrographs for a number of events at the gauge location. Hydrograph deconvolution involves deriving the unit hydrograph characteristics from observed flow events. To deconvolute the recorded events, the Collins method (31) was used since it is a convenient technique which has been computer programmed. The Collins method is an iterative technique which utilizes a recorded hydrograph and incremental rainfall amounts to directly compute the shape of the watershed unit hydrograph. A summary of the relevant physiographic and unit hydrograph parameters which were determined for each subwatershed are given in Table 3.3.

ii) Soils

The soil types were interpreted from unpublished superficial geology maps (scale 1:2400) available from the Newfoundland Department of Mines and Energy (17). The soils are mainly comprised of sandy, gravely glacial deposits which constitute a hydrologic soils group of between B and CB (see Table 3.4). The distribution of the various hydrologic soils groups in the watershed is presented in Figure 3.3. (Additional information on soil cover complex numbers is given in Appendix C.)

iii) Land Use

The Blanche Brook watershed is primarily forest covered with a significant portion of the land use being open areas, bogs, residential and industrial. Utilizing forestry maps at a scale of 1:2400, it was determined that an area of about 14% of the watershed has been cut over.

The land use distribution was calculated based on interpretation of existing 1:12,500 and 1:50,000 topographic mapping. The watershed land use distribution is presented in Figure 3.4, including the distribution of cut over forested area.

iv) Soil Cover Complex Number

A runoff index factor combining the hydrologic soil group and land use characteristics is referred to as a hydrologic soil cover complex number, CN. Simulated runoff volumes are proportional to the complex number according to the following rainfall-runoff relationship:

$$Q = \frac{(P - 0.2s)^2}{P + 0.8s} \tag{3.2}$$

where $P$ = precipitation amount (inches)

$s = \frac{1000}{CN} - 10$

$Q$ = amount of runoff (inches)

(Equation 3.2 was originally developed in English units. To convert the amount of runoff to millimetres, multiply by 25.4.)

Using the available soils and land use information, average soil cover complex numbers were determined for each of the 8 sub-basins (see Appendix C for details of procedures for estimating soil cover complex numbers). The average soil cover complex numbers are summarized in Table 3.3.

The antecedent soil moisture condition was calculated for a number of historical peak flow events which have occurred on Blanche Brook. By comparison of the calculated value to average conditions estimated for the watershed, it was determined that AMC III (Antecedent Moisture Condition III) is representative of the runoff characteristics of the watershed prior to peak flow events. AMC III is representative of saturated soil conditions.

### 3.3.5 Model Calibration and Validation

Utilizing the available meteorological data from recording stations in the area and the streamflow records from the Blanche Brook station, the

HYMO model was calibrated to summer storm flood events (as described later).

The general procedures for calibration of the HYMO model are summarized as follows:

1) isolate suitable calibration and validation events from available meteorologic and hydrologic data
2) from meteorologic data, select a storm which is representative of the associated flood conditions under consideration
3) from hydrologic data determine the average watershed base flow adjusted runoff volume for the storm under consideration
4) using the total rainfall amount and the average runoff volume, compute the average curve number for the entire watershed
5) using the average curve number for the entire watershed, compute the temporal distribution of the recorded runoff for the selected event for input to the Collins method (31)
6) utilizing the Collins method of deconvoluting runoff hydrographs, calculate the unit hydrograph parameters K and $T_p$ for the watershed at the gauge
7) using available empirical equations, estimate unit hydrograph parameters K and $T_p$ for the watershed at the gauge
8) calculate factors which convert the estimated K and $T_p$ unit hydrograph parameters to the actual parameters calculated by the Collins method
9) apply these same factors to the subwatershed estimated unit hydrograph parameters
10) compare computed outflow hydrograph with recorded outflow hydrograph
11) repeat above procedure for several recorded events until average subwatershed unit hydrograph parameters are obtained
12) validate model by using other recorded events.

From a review of the meteorologic and hydrometric data available for Blanche Brook, available flood events were reviewed and specific events selected for calibration and validation purposes. The events selected are summarized as follows:

1) A summer rainfall event which occurred during the period of July 17-19, 1979 and deposited 88 mm of rain at the Stephenville meteorologic station. The runoff from this event was calculated to be approximately 41.9 mm and resulted in a maximum instantaneous flow rate of 113 $m^3/s$. Based on baseflow separation calculations, essentially all of the maximum instantaneous flow rate was found to be runoff.

2) A summer rainfall event which occurred during the period of July 28-30, 1979 and deposited 44.4 mm of rain at the Stephenville meteorologic station. The runoff from this event was calculated to be approximately 27.8 mm and contributed to a maximum instantaneous flow rate of 62.7 $m^3/s$. Based on baseflow separation calculations, the maximum direct runoff rate was estimated to be about 57 $m^3/s$.

3) A summer rainfall event which occurred during the period of October 11-13, 1980 and deposited 60 mm of rainfall at the Stephenville meteorological station. The runoff from this event was calculated to be approximately 30.3 mm and resulted in a maximum instantaneous flow rate of 63.6 $m^3/s$. The maximum direct runoff rate was estimated to be 61.1 $m^3/s$.

4) A spring rainfall plus snowmelt event which occurred April 5-8, 1981. The runoff from this event was calculated to be about 80 mm resulting in a peak flow of about 53 $m^3/s$ at the gauge.

The HYMO computer model is only intended for simulating direct (overland) runoff resulting from input of storm events. The observed hydrographs were, therefore, separated into direct and groundwater runoff components. The most widely used separation procedure consists of extending the recession existing before the storm to a point under the peak of the hydrograph. From this point, a linear base flow is assumed to intersect the recession limb of the hydrograph at a point located a fixed point in time after the peak of the hydrograph. According to Linsley (35), the time base of direct runoff (from the recession point under the peak) should remain relatively constant from storm to storm. The time base of direct runoff was estimated to be approximately 40 hours, using procedures recommended by Linsley (35). In general, the base flow adjustment was not significant compared to the magnitude of peak flows (less than 1% to 2% of the peak).

The hydrograph deconvolution for selected events resulted in unit hydrograph shape factors that were quite similar to those calculated by the empirical equations (see Tables C.1 and C.2 of Appendix C). However, a slight modification of the model parameters was justified to overcome a small peak discharge underestimation which was found to occur if HYMO simulations were undertaken using the theoretical unit hydrograph parameters. This is consistent with similar calibration of the HYMO model on other watersheds.

The unit hydrograph parameters which resulted in the "best" comparison of simulated and recorded hydrographs are summarized in Table 3.3.

A graphical comparison of simulated and recorded hydrographs is presented in Figures 3.5, 3.6 and 3.7. By means of the comparisons to observed events, it is evident that the calibrated HYMO model accurately simulates peak runoff rates to within an average of 10% of the recorded peak flows for the summer rainstorm events, while being on the conservative side for an event occurring in the fall. Runoff volumes were simulated to within an average of 1% of observed amounts.

The model was further validated using a spring rainfall plus snowmelt event which occurred April 5-8, 1981. A comparison of observed and simulated flow for this event indicated good simulation results (e.g. runoff volume was simulated to within about 3%). On the basis of the calibration and validation simulations; it was concluded that the model gave good simulation results for both rainfall and snowmelt events.

### 3.3.6 Sensitivity Analysis

The peak flow rate for the dimensionless unit hydrograph is estimated by the following equation (41):

$$q_p = \frac{b\ DA\ Q}{T_p} \tag{3.3}$$

where $b$ = watershed unit hydrograph parameter estimated as a function of K and $T_p$

$DA$ = drainage area ($mi^2$)

$Q$ = volume of runoff (inches)

$T_p$ = time to peak (hrs)

$q_p$ = unit hydrograph peak flow rate ($ft^3/s$)

(To convert $q_p$ to $m^3/s$ divide by 35.31)

The model sensitivity to variations in input parameters was tested by varying estimated parameters within prescribed ranges. The calibration results indicate that the model appears to be accurately simulating the time of the peak flow rate. This parameter was, therefore, kept constant throughout the sensitivity analyses.

One parameter which has a direct influence on the peak magnitude of the unit hydrograph is the "b" value. (The "b" value is a function of K and $T_p$.) This parameter is reflective of the "peakiness" of a unit hydrograph. To check the parameter sensitivity the "b value" for each subwatershed was adjusted ±20%, which is a realistic variation based on experience in other applications. It is evident from the results presented in Table 3.5 that the peak discharges for the 1:100 year event at the gauge location vary approximately plus 6% and minus 8% corresponding to plus and minus 20% changes to the b values. This percentage change in peak discharge was also similar for other hydrologic reference points analysed.

Another factor to which the estimated flows are sensitive is possible changes in the storage available in Noels Pond due to operation of the outlet structure. Simulations of the rainstorm design events have been carried out assuming the summer condition of Noels Pond (i.e. stop logs in place). It was found (see Table 3.5) that for a 1:100 year rainstorm event with the stop logs removed from Noels Pond (i.e. "winter operation" condition), the peak flow would be reduced downstream on Blanche Brook (at gauge location) by approximately 6% when compared to simulations undertaken with all logs in place. The small difference is due to the fact that the generation of significant peak flows at the gauge location are influenced relatively more by Blanche Brook, which is not affected by artificial storage.

It should be noted that by utilizing the "winter operation" of Noels Pond during a 1:100 year rainstorm event, flow reductions within the reach between the pond outlet and the confluence are not as significant (3%) as that estimated for the gauge location. The reduction in peak flow at the gauge location during a 1:100 year rainstorm event is primarily due to the additional hydrograph lag induced by the "winter operation" of Noels Pond. However, the additional lag is less than one half of an hour when compared to the summer operation of Noels Pond.

A further analysis was undertaken to determine the significance of Noels Pond on peak discharges in the downstream reaches. It was found that for a 1:100 year rainstorm event Noels Pond (summer operation) reduces the peak flow about 25% immediately downstream of the reservoir and about 15% at the gauge location. Another major influence which Noels Pond was found to have on the peak flow regime is that it tends to lag the hydrograph produced by Warm Creek by 3-4 hours. This result is consistent with local general observations on peak flow conditions.

The effect of changing the antecedent soil moisture condition on estimated peak flows was also analysed. Saturated conditions (AMC III) were originally selected based on historical flood events. It was found that the peak flows for a 1:100 year rainstorm event are reduced by an average of 43% when an AMC II condition (more representative of drier soil conditions) is assumed.

### 3.3.7 Peak Flow Estimates

Subsequent to model calibration and sensitivity testing, peak design flows were estimated for several locations in the watershed. The simulation locations (hydrologic reference points) are located on Figure 3.1. Meteorologic and hydrologic input parameters were determined as discussed in Sections 3.3.3 and 3.3.4 respectively. It was found that the average base flow adjustment would be less than 1% of the peak flow value simulated by the HYMO model (see Section 3.3.5 for base flow analysis procedures).

Peak flow estimates for snowmelt conditions were found to be smaller than for summer rainstorm conditions. This is attributed to the relatively small drainage area for the Blanche Brook watershed, the higher incremental precipitation intensities associated with summer rainstorm events and the relatively fast response time of the watershed. This is consistent with findings in previous studies for relatively small watersheds.

It should be noted that during the 1:100 year rainstorm event, it is likely that due to high water levels, some spill from Noels Pond may occur at a low spot on Connecticut Drive (see Figure 5.1). However, because the magnitude of this spill is less than 3% of the total discharge downstream, and because the time of occurrence would be after the peak discharge in the downstream reaches, the effects were assumed to be negligible. The potential for spill could be eliminated in the future by raising Connecticut Drive to an elevation above 24.5 m.

The resulting peak flow estimates produced by the HYMO model simulations are summarized in Table 3.6.

### 3.3.8 Summary of Deterministic Analyses

1) Upon review of the meteorologic and hydrometric data available for Blanche Brook, several events were selected for calibration. Good

comparison of simulated and observed peak flows were obtained. Also good comparisons of simulated and observed runoff volumes resulted from the model calibration and verification.

2) Using results of the HYMO model, the peak flows were simulated for summer rainstorm events. Table 3.6 summarizes the results of the simulations for the design rainstorm events.

3) Based on the present simulation model, it was found that about 35% of the flow to the gauge at the time of the peak is generated from the Warm Creek tributary. The peak discharge of Blanche Brook occurs 3-4 hours before the peak discharge from Warm Creek. This finding is consistent with observations.

4) It was found that AMC III better represents the soil moisture condition found in the watershed prior to the occurrence of peak flow events.

5) The downstream effects of having the stop logs at the outlet of Noels Pond removed before the event are negligible during severe flood events such as the 1:100 year rainstorm event (see Table 3.5).

6) The flood routing model indicated that a potential for some spill from Noels Pond exists at a low point along Connecticut Drive for severe flood events. The potential for spill could be eliminated in the future by raising Connecticut Drive to an elevation above 24.5 m.

## 3.4 Main Conclusions and Recommendations of the Hydrologic Analyses

Table 3.7 presents a brief summary comparison of peak flows estimated (to the gauge location) by the various techniques on Blanche Brook. In general terms, the alternative estimating techniques provided comparable estimates of peak flows for the 1:20 and 1:100 year flood events.

It was found that the design flows estimated by the alternative techniques are generally within the ±95% confidence limits estimated for each technique. The main findings and conclusions of the hydrologic investigations are summarized as follows:

1) The available period of record of the data base at the Blanche Brook hydrometric station is too short for undertaking a reliable flood frequency analysis.

2) Statistical tests indicated that it was possible to undertake a single station flood frequency analysis using the nearby peak flow data base available for the Harrys River. However, peak flow transfer to Blanche Brook (by transfer of sample statistics) has indicated that the single station analysis on the Harrys River is very sensitive to extreme flows which have occurred in 1973. The statistical transfer of flows from the Harrys River to Blanche Brook is, therefore, not recommended for further use in this investigation. However, application of this procedure has tended to confirm the order of magnitude of design flows estimated by other techniques.

3) Regional statistical analyses were used for undertaking peak flow estimates by two alternative estimating equations. The first method (38) was originally derived (using a data base available to 1969) for estimating mean daily flood peaks and requires definition of a peaking factor for estimating instantaneous peaks. On the other hand, the CNFDRP Regression Equations (13) provide estimates of the instantaneous peak flows for the 1:20 and 1:100 year recurrence interval floods using an up-to-date data base which resulted in an estimating equation with a smaller standard error. The latter method is, therefore, preferred for purposes of comparison to other estimating techniques.

4) A successful calibration of the HYMO model was undertaken on Blanche Brook and Warm Creek. Good comparison of simulated and observed peak flows and runoff volumes were obtained.

5) The HYMO model was used to simulate peak flows for the 1:20 and 1:100 year design events. It was found that summer rainstorms resulted in higher flows than spring runoff events. The comparison of peak flows to alternative estimating techniques (see Table 3.7) has confirmed that the unit hydrograph model produces reasonable estimates of the 1:20 and 1:100 year peak flows.

6) The available storage in Noels Pond was found to have only a small effect on peak flows at the outlet of Blanche Brook. However, in order to estimate flood levels in Noels Pond, hydrograph routing using the HYMO reservoir routing technique was necessary. This required determination and use of inflow hydrographs simulated by the HYMO procedures.

7) The use of peak flows and associated hydrographs generated by the HYMO unit hydrograph procedures is recommended for determining flood profiles for the purposes of this investigation. The model was found to accurately simulate peak flows and accounts for the effects of reservoir and channel storage in lagging and attenuating the flow peaks at downstream locations. The distributed flow estimates (taking into account areal variations in watershed characteristics) are summarized in Table 3.6. The resulting 1:20 and 1:100 year flood profiles are discussed in Section 4.0.

TABLE 3.1

Blanche Brook Regional Flood Estimates*

for Specified Recurrence Intervals

| Recurrence Interval (years) | Poulin** ($m^3/s$) | FDRP*** ($m^3/s$) |
|---|---|---|
| 20 | 134.3 | 126.1 |
| 100 | 172.2 | 162.8 |

NOTES:

* At gauge location

** Ref (38). Includes instantaneous peaking factor of 1.60

*** Canada-Newfoundland Flood Damage Reduction Program; Ref (13)

TABLE 3.2

Rainfall Distributions for the 12-hour Design Storms

| Time (hours) | 12 hr. Temporal Distributions* (%) |
|---|---|
| 0 - 1 | 5 |
| 1 - 2 | 8 |
| 2 - 3 | 8 |
| 3 - 4 | 10 |
| 4 - 5 | 10 |
| 5 - 6 | 14 |
| 6 - 7 | 13 |
| 7 - 8 | 8 |
| 8 - 9 | 10 |
| 9 - 10 | 8 |
| 10 - 11 | 4 |
| 11 - 12 | 2 |
| | 100 |

* Source - Analysis by Environment Canada (4)

TABLE 3.3

Physiographic and Unit Hydrograph Parameters for Sub-Watersheds

| Basin Nos. | Area ($km^2$) | Length (km) | Equivalent Slope** (m/m) | Recession Parameter (hrs) (K) | Time to Peak Parameter (hrs) $T_p$ | Average* Soil Cover Complex Number (CN) |
|---|---|---|---|---|---|---|
| 101 | 47.21 | 17.82 | 0.0131 | 2.94 | 3.27 | 87 |
| 102 | 15.48 | 9.75 | 0.0207 | 1.58 | 1.65 | 87 |
| 103 | 1.16 | 1.08 | 0.0333 | 0.48 | 0.35 | 89 |
| 201 | 35.40 | 12.55 | 0.0208 | 1.84 | 2.24 | 88 |
| 202 | 9.08 | 3.72 | 0.0188 | 1.91 | 1.15 | 92 |
| 203 | 6.89 | 4.29 | 0.0334 | 0.82 | 0.84 | 88 |
| 204 | 1.85 | 1.43 | 0.0236 | 0.71 | 0.51 | 90 |
| 300 | 1.52 | 1.59 | 0.0158 | 0.97 | 0.59 | 91 |

NOTES

* AMC III (represents saturated soil moisture conditions)

** Equivalent slope; determined by channel segments (see ref (43))

TABLE 3.4

Summary of Hydrologic Soil Groups

| Hydrologic Soil Group | Percent of Watershed |
|---|---|
| CB | 7.7 |
| C | 36.2 |
| CD | 40.4 |
| D | 14.5 |

TABLE 3.5

Sensitivity Analysis of the HYMO Deterministic Model

| Changes to Model | Average Change in Peak Discharges at Gauge % |
|---|---|
| Vary b Value | |
| Increase b value 20% | 6% |
| Decrease b value 20% | -8% |
| ii) Modify storage available in Noels Pond | |
| Winter Operation of Noels Pond Reservoir Assumed | -6% |
| iii) Modify Antecedent Soil Moisture Condition | |
| AMC II Condition assumed | -43% |

NOTES:

* Percent changes given are for the 1:100 year, 12-hour rainstorm

TABLE 3.6

Peak Flow Estimates on Blanche Brook and Warm Creek by

HYMO Deterministic Computer Model

| Hydrologic Reference Pt.* | Location | Return Period** 1:20 ($m^3/s$) | 1:100 ($m^3/s$) |
|---|---|---|---|
| Blanche Brook: | | | |
| 1 | Outlet | 112.0 | 166.5 |
| 2 | Gauge Location | 110.8 | 165.6 |
| 3 | Minnesota Drive | 79.4 | 106.8 |
| 4 | Hansen Hwy. (Rte 460) | 58.1 | 78.4 |
| Warm Creek: | | | |
| 5 | At confluence with Blanche Brook | 48.6 | 68.1 |
| 6 | Noels Pond Outlet weir | 50.6 | 68.6 |
| 7 | Noels Pond Inlet (Rte 490) | 59.5 | 83.5 |
| 8 | At tributary d/s of Rte 460 | 62.0 | 82.7 |
| 9 | Route 460 | 51.2 | 68.2 |

NOTES:

* Refer to Figure 3.1

** Refer to Tables 3.2 and C.8 for rainfall distributions and amounts assumed for summer condition

TABLE 3.7

Comparison of Peak Flow Estimates on Blanche Brook*

| Hydrologic Flow Estimation Method | Maximum Instantaneous Flow Rate ($m^3/s$) Recurrence Interval - years) 1:20 | 1:100 |
|---|---|---|
| Regional - CNFDRP | 126.1 | 162.8 |
| Regional - Poulin** | 134.3 | 172.2 |
| Deterministic Model | 110.8 | 165.6 |

NOTES:

* At gauge location

** Peaking factor of 1.6 applied to mean daily Q to yield maximum instantaneous flow rate

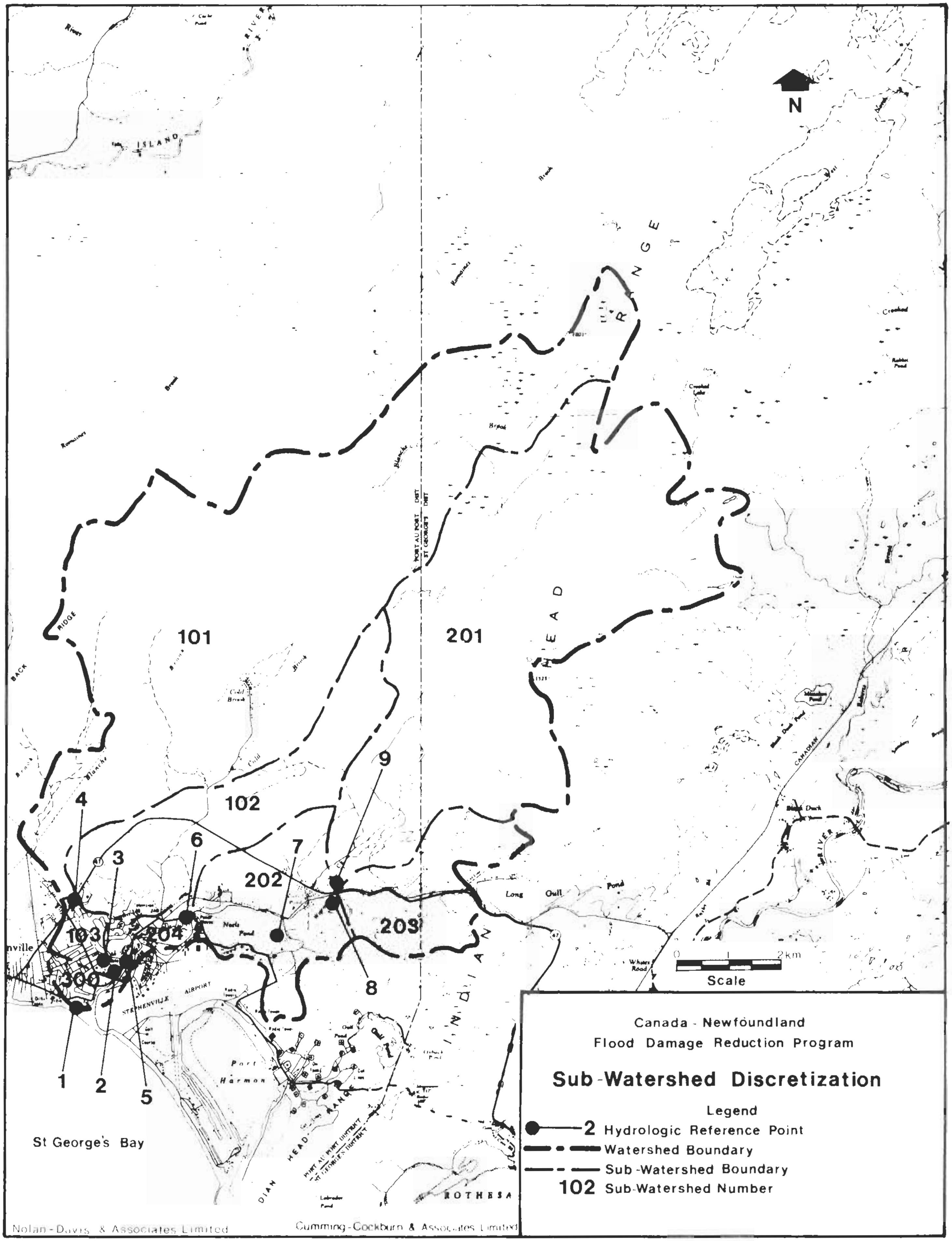
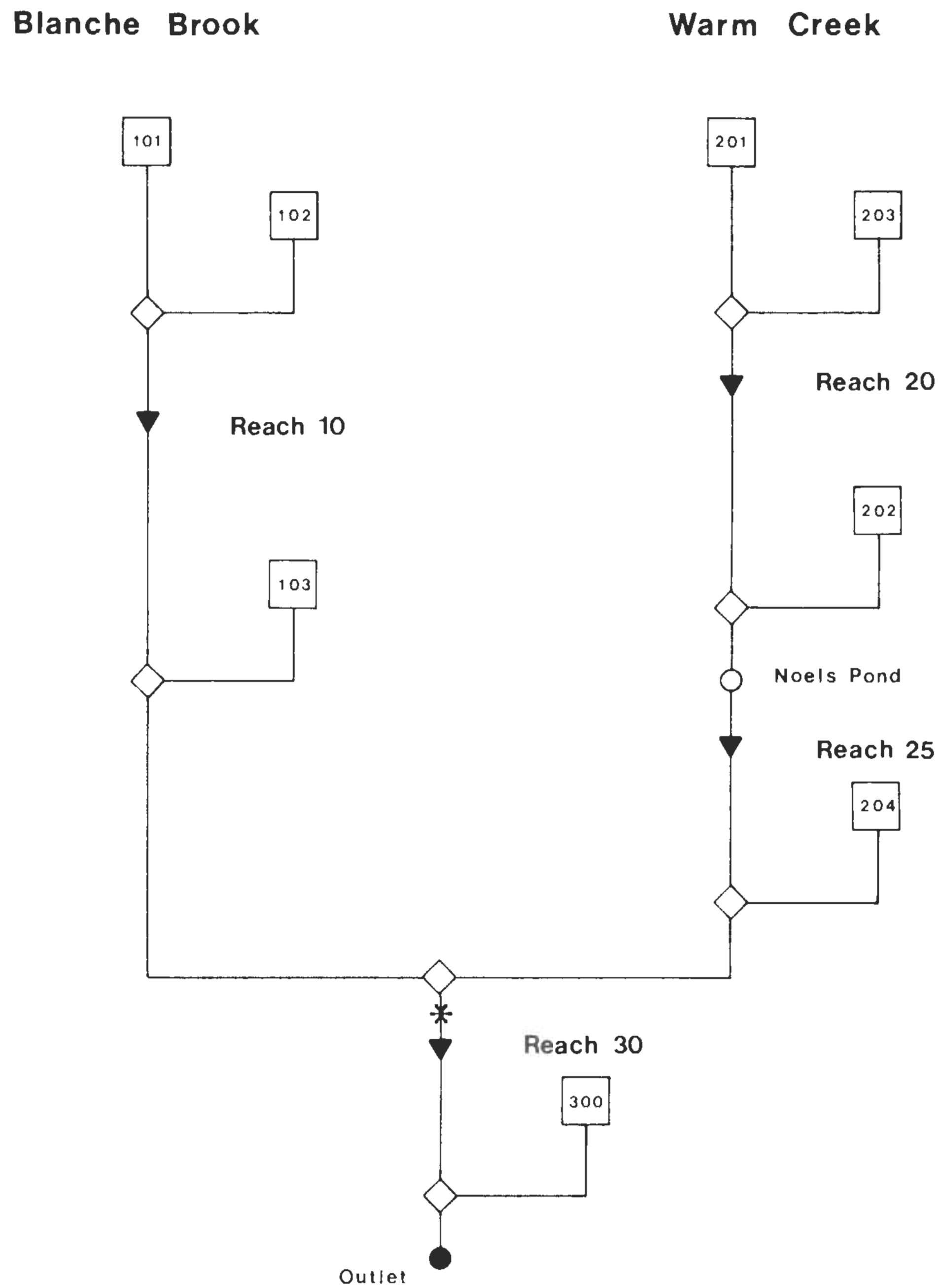

N
101
102
103
201
202
203
204
300
1
2
3
4
5
6
7
8
9
St George's Bay
Scale
0 1 2km
Canada - Newfoundland
Flood Damage Reduction Program
Sub-Watershed Discretization
Legend
2 Hydrologic Reference Point
Watershed Boundary
Sub-Watershed Boundary
102 Sub-Watershed Number
Nolan-Davis & Associates Limited
Gumming-Cockburn & Associates Limited

Figure 3.1




Canada - Newfoundland
Flood Damage Reduction Program

## HYMO Model Schematic

Legend

| | | | |
|---|---|---|---|
| 103 | Subcatchment No. | ◇ | Hydrograph Addition |
| ○ | Reservoir Routing | * | W.S.C. Gauge Location |
| ▼ | Reach Routing | | |



Figure 3.2

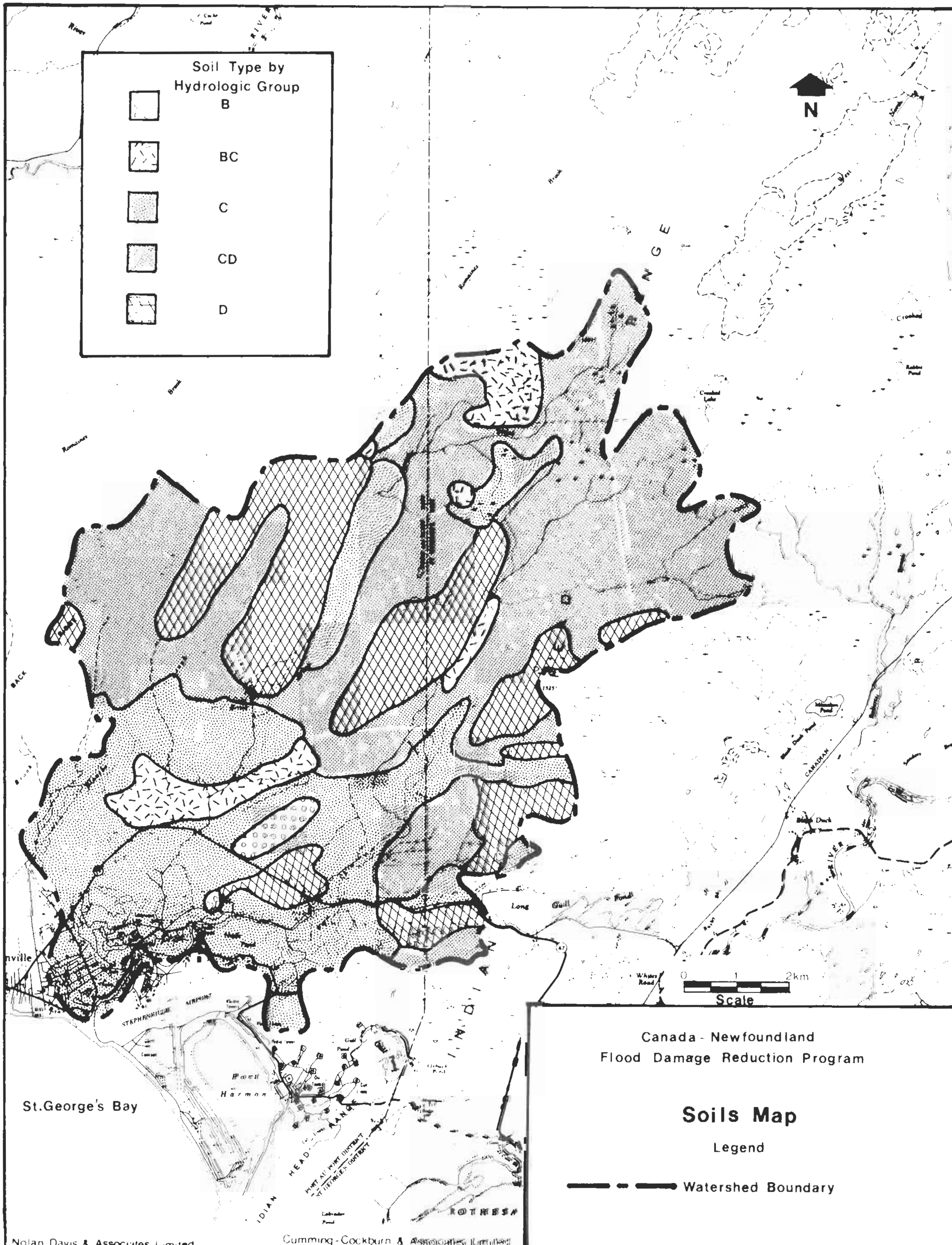

Soil Type by Hydrologic Group
B
BC
C
CD
D
N
St.George's Bay
Long Gull Pond
Harmon
0 1 2km
Scale
Canada - Newfoundland
Flood Damage Reduction Program
Soils Map
Legend
Watershed Boundary
Nolan Davis & Associates Limited
Cumming-Cockburn & Associates Limited

Figure 3.3

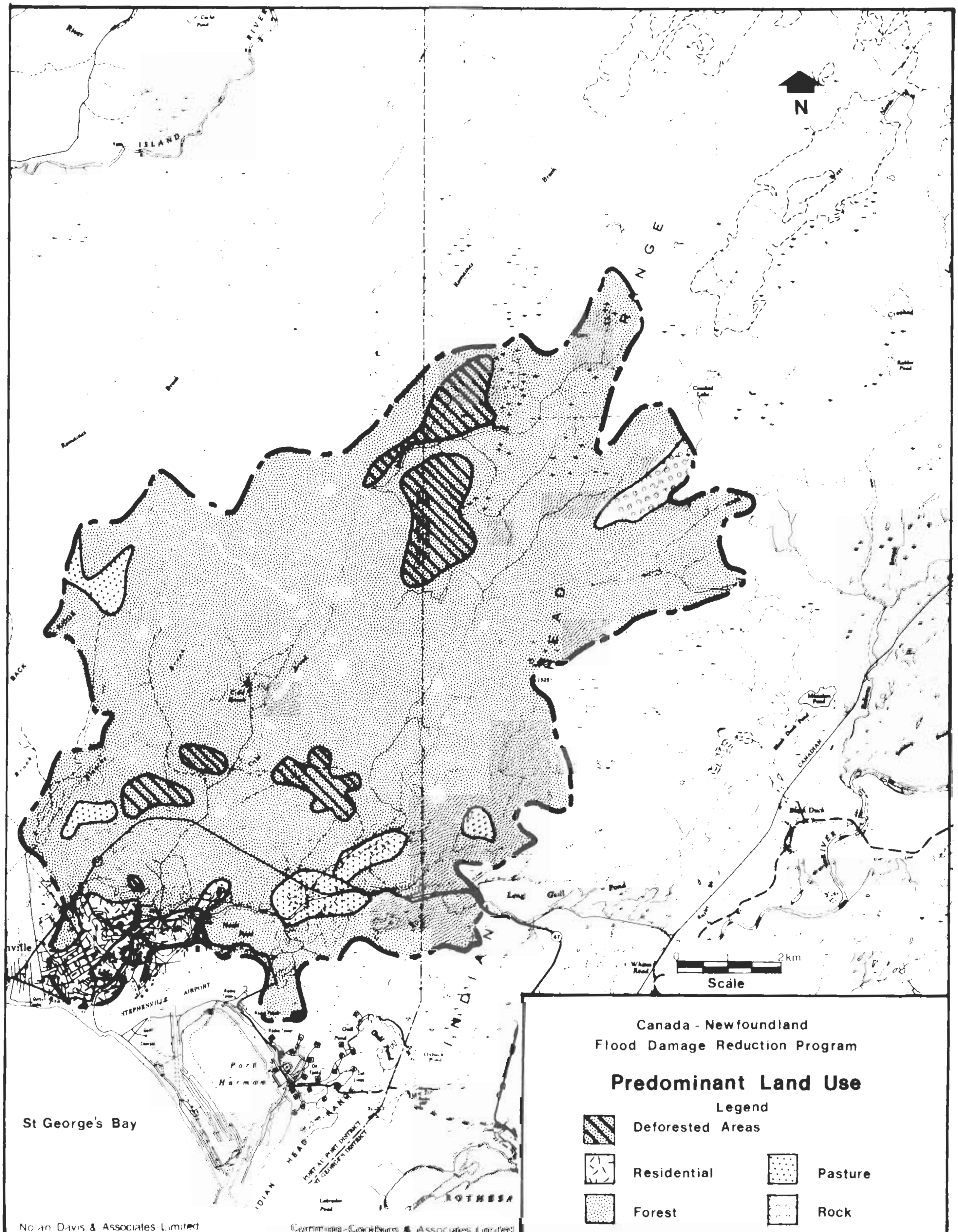
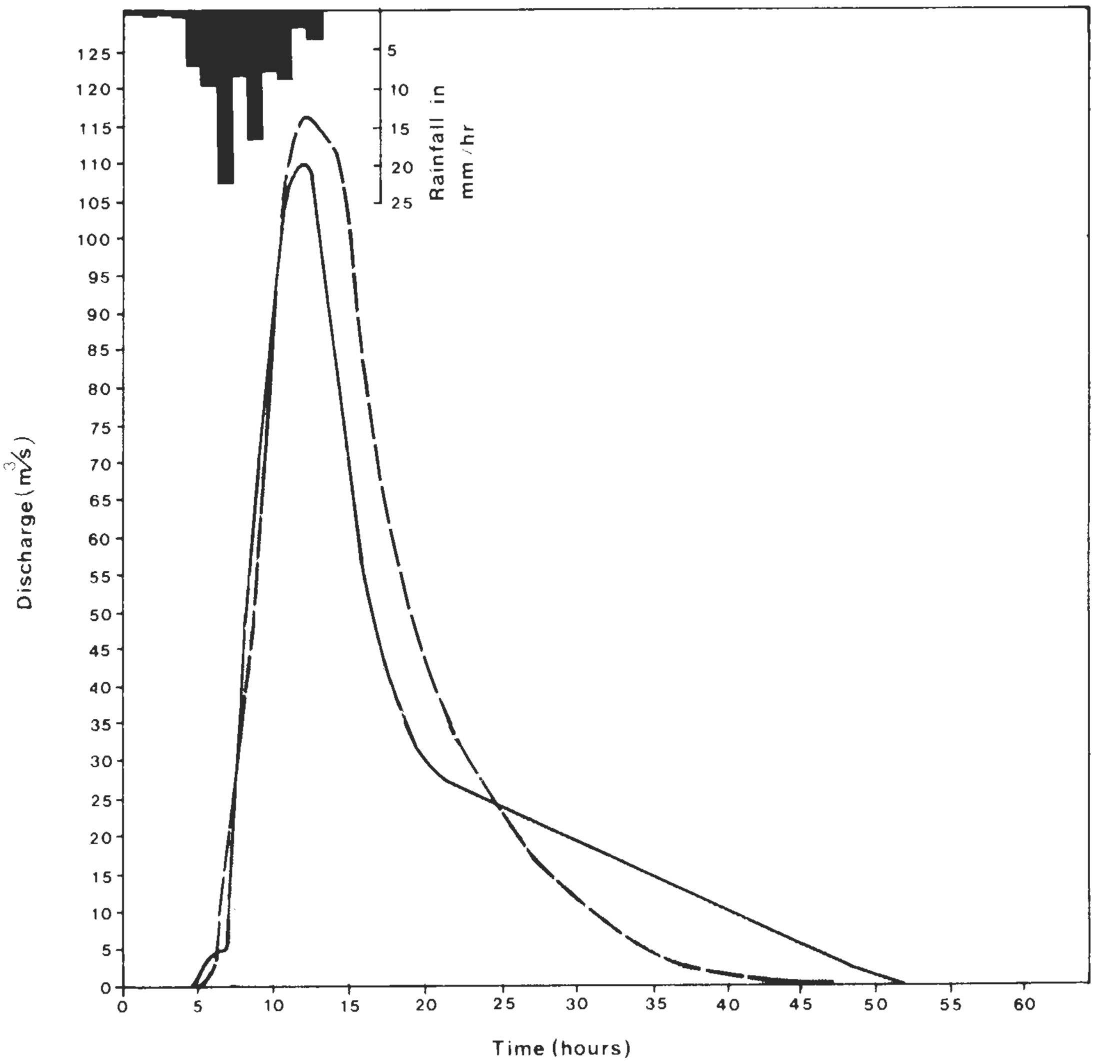

N
St George's Bay
Port Harmon
STEPHENVILLE AIRPORT
Long Gull Pond
0 2km
Scale
Canada - Newfoundland
Flood Damage Reduction Program
Predominant Land Use
Legend
Deforested Areas
Residential
Pasture
Forest
Rock
Nolan Davis & Associates Limited
Cumming-Cockburn & Associates Limited

Figure 3.4




Event of July 17-19, 1979

Canada Newfoundland
Flood Damage Reduction Program

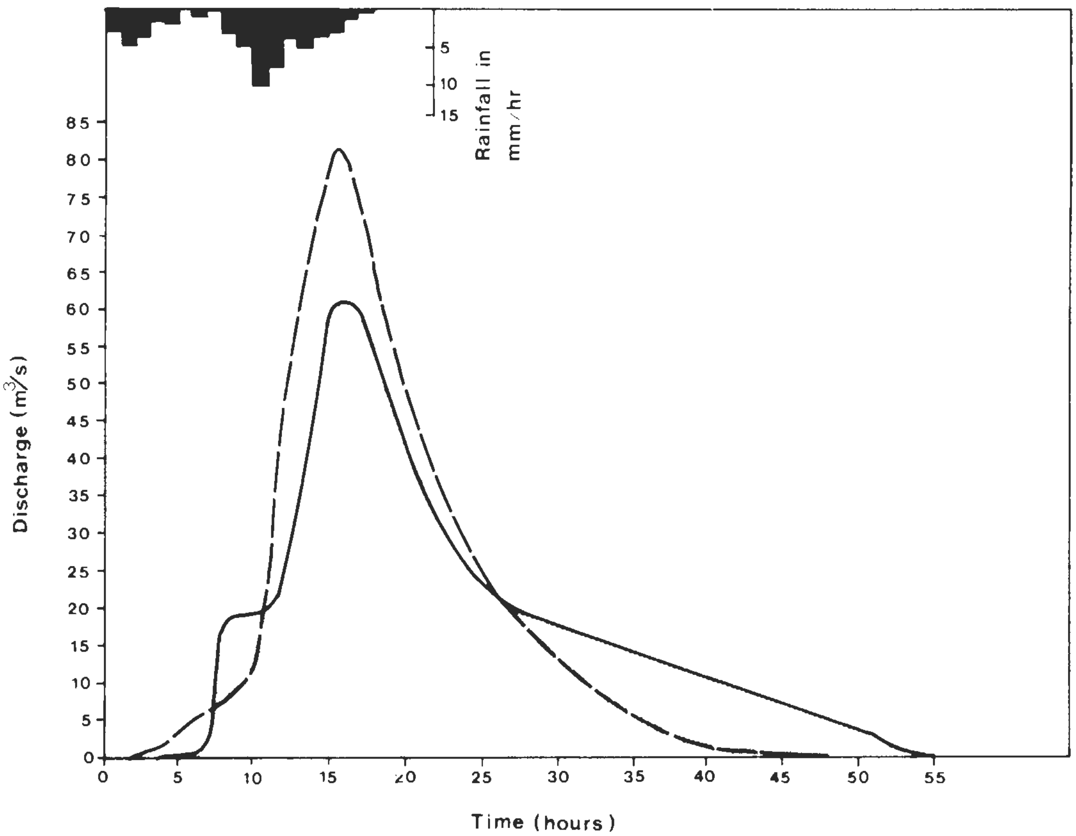
**Comparison of Measured and Simulated Flows**

Legend

——— Recorded Flow
— — — Simulated Flow

Nolan Davis & Associates Ltd. Cumming-Cockburn & Associates Ltd.

**Figure 3.5**




Event of October 11 - 13, 1980

Canada-Newfoundland
Flood Damage Reduction Program

**Comparison of Measured and Simulated Flows**

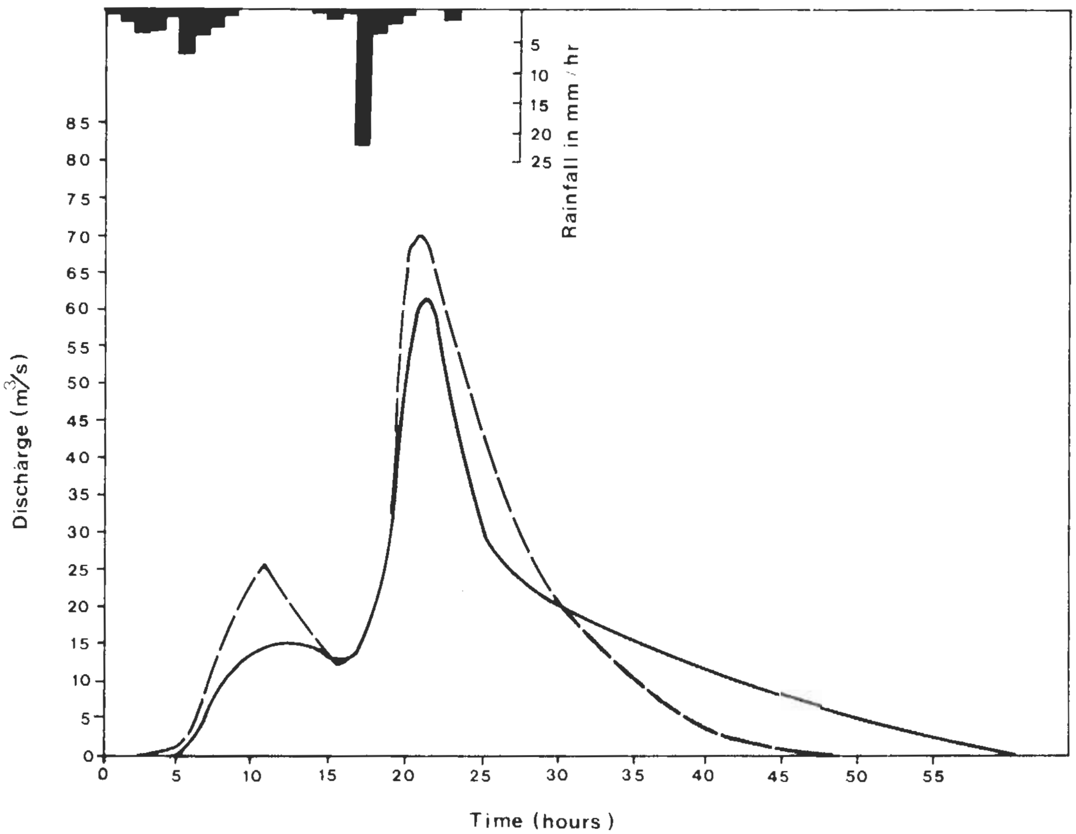
Legend

Recorded Flow

Simulated Flow



Figure 3.6




Event of July 28-30; 1979

Canada-Newfoundland

Flood Damage Reduction Program

**Comparison of Measured and Simulated Flows**

Legend

——— Recorded Flow

— — — Simulated Flow

Nolan Davis & Associates Ltd    Cumming-Cockburn & Associates Ltd

**Figure 3.7**

# Hydraulic Analyses 4.0

# Hydraulic Analyses 4.0

## 4.1 Model Description

The main purpose of the hydraulic analyses was to transform peak discharge estimates into flood profiles along the study reach. This was undertaken by utilizing a mathematical model to simulate water surface profiles corresponding to the 1:20 and 1:100 year flood events. The effects of channel and floodplain storage on flood profiles along the study reach were not considered to be significant with the exception of the Noels Pond area, which is discussed later in this report. In addition, it was postulated that tidal variations were not a significant factor in determining flood levels along the study reach (this was subsequently confirmed by undertaking model sensitivity testing). In such cases it is a standard practice to assume steady state flow conditions in the computation of the backwater profiles.

Where a steady state backwater computation is employed, the appropriate design peak discharge input to the model is the instantaneous peak of the flood hydrograph.

In the case of gradually varied steady flow, the equations of continuity and momentum describing the one-dimensional flow can be simplified to the form of the well-known Bernoulli equation:

$$\frac{\partial h}{\partial x} = (S_o - S_f) \,/\, (1 - v^2/gh) \qquad (4.1)$$

where h = depth of flow (m)

x = distance in direction of flow (m)

$S_o$ = bottom slope (m/m)

$S_f$ = boundary frictional effect (m/m)

v = velocity in direction of flow (m/s)

g = acceleration due to gravity ($m/s^2$)

For natural channels, energy losses occue due to flow resistance. The resulting friction slope can be determined from the Manning's equation:

$$S_f = (nv/R^{2/3})^2 \qquad (4.2)$$

where n = Manning's roughness coefficient
R = hydraulic radius

In order to estimate the flood levels associated with each of the 1:20 and 1:100 year flood peaks, a mathematical model was developed to simulate the hydraulic characteristics along Warm Creek and Blanche Brook.

The HEC-2 Model (45) was selected for use in simulating flood profiles along the study reaches. This model has been successfully used in many similar practical applications. The HEC-2 model was selected since it is a well-proven and well-documented non-proprietory technique which is flexible to use. The model can be applied in the future to evaluate the effects of recommended hydraulic improvements and proposed channelization along the study reach, etc.

The program calculates water surface profiles for flow in natural or man-made channels, assuming that such flow is steady and gradually varied. The simplified one-dimensional equations of continuity and motion are solved using the standard step method with energy losses due to friction evaluated by the Manning's equation.

In addition, the model can calculate critical depth at each cross-section and can compute profiles for supercritical flow, where required. This feature is especially useful for steep stream reaches such as those which intermittently occur along portions of the study reaches. Backwater profiles can be run for subcritical flow conditions by specifying a starting water level at the downstream end of a 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 a given study reach.

The model can take into account the following factors:

1. Channel roughness
2. Floodplain roughness
3. Islands or flow divisions
4. Bends in the stream or floodplain
5. Cross-sectional area of the stream channel and floodplain
6. Slope of the channel and floodplain
7. Energy losses at hydraulic structures, including bridges, culverts, weirs, dams, etc.
8. Channel and floodplain expansion and contraction losses
9. Variation in discharge along the study reach (i.e. due to tributary inflows.)
10. The effect of ice/debris covers on the stream or floodplain.

The model requires input of channel and floodplain cross-sections and associated hydraulic parameters at frequent locations along the study reach. The cross-sections are normally located where changes occur in slope, cross-sectional area or channel roughness, and at bridges or culverts.

A major advantage of the HEC-2 model is tnat the channel and floodplain roughness (Manning's 'n') can be varied for each cross-section in the program. This allows a description of the various factors on which the roughness coefficient depends such as channel morphology, type and extent of vegetation, etc.

Energy losses created at hydraulic structures, such as bridges and culverts, are computed in the program in two parts. First the energy losses due to expansion and contraction of the flow at the cross-section on the upstream and downstream sides of the structure are calculated, and second, the energy loss through the structure itself is computed by either using the special bridge or the normal bridge sub-routine. Energy losses due to expansion and contraction of flow are calculated by employing expansion and contraction coefficients which are multiplied by the absolute difference in velocity heads between cross-sections to estimate the energy loss caused by the transition.

When the normal bridge sub-routine is used the water level is computed at the bridge or culvert section in the same manner as normal river cross-sections, but excluding the cross-sectional area of any existing piers, deck or wingwalls below the water surface. When the water surface elevation exceeds the bottom cord, the wetted perimeter of the section is also adjusted. The special bridge routine computes losses through the structure for low flow or for any combination of weir flow and pressure flow. Similarly, water levels associated with floating ice covers can be modelled with the program, excluding the cross-sectional area of the ice sheet from the channel area. The effect of the ice roughness coefficient and change in hydraulic radius is also taken into account.

The watercourse modelled consisted of the following stream reaches:

i) Blanche Brook beginning at the Brook outlet at St. Georges Bay and extending some 3.6 km upstream to a point north of the Hansen Highway (Route 460).

ii) Warm Creek beginning at its confluence with Blanche Brook in the Town of Stephenville and extending about 8.2 km upstream to a point above Route 460.

In all, a total of nearly 12 km of channel and floodplain was modelled, as discussed in detail later in this report.

The computer model was developed to simulate the existing hydraulic characteristics of the watercourses based on the channel and floodplain configuration as interpreted from the existing topographic mapping and the results of comprehensive field topographic and reconnaissance surveys (see Section 4.2). Subsequent to model development and sensitivity testing, the backwater model was calibrated and verified using measured water levels (see Section 4.4). The flood profiles associated with the peak 1:20 and 1:100 year discharge rates were then established based on the calibrated model. The flood profiles are discussed in Section 4.6. The sensitivity of simulated flood profiles to variations in model input parameters and hydraulic characteristics (including the possible effects of ice and debris jams on flood profiles) are discussed in Section 4.5.

## 4.2 Hydraulic Model Input Data

### 4.2.1 Field Survey

Some cross-sectional data were available as a result of previous field surveys undertaken by Environment Canada (27). However, it was found that special care was required in the use of this information since recent changes to the channel and floodplain have occurred along the study reach. Therefore, field surveys were undertaken as part of this investigation to measure typical channel and floodplain cross-sections.

All topographic information collected during the field surveys was related to geodetic elevation and all sections were located by means of reference to fixed physiographic points located near the flood plain.

Floodplain and channel roughness coefficients were also assessed in the field. An inventory of photographs documenting the present conditions of the floodplain and channel characteristics, as well as the hydraulic structures located therein is available as part of the supplementary report.

i) Cross-sections

A total of 66 cross-sections were field surveyed along the study reaches. The complete inventory of cross-sections, including location and extent is described in detail in the supplementary report entitled, "Physical Surveys and Field Program". Cross-section measurements were obtained at representative locations along the study reach, and located based on changes in the slope, cross-sectional area or channel roughness. Additional measurements were taken near all bridge and culvert crossings along the study reaches. Typical cross-sections for selected stream reaches are shown on Figure 4.1, and plots for each location are given in the supplementary report.

The results of field surveys undertaken for this study were compared to those obtained previously (27). Based on this review, it was noted that the overbank characteristics in most areas along Blanche Brook and Warm Creek have not changed appreciably since the time of Environment Canada's study.

The field surveyed results were also compared to the new 1:2500 scale mapping. Results of this comparison showed that both mapping and survey results give similar elevations for the overbank areas in most areas along much of the study reach. In the more urbanized areas, however, an appreciable degree of difference was noted between the two in some instances, specifically at road and bridge locations. Therefore, while the new mapping could be used to supplement the field surveys, care was required in supplementing the field survey results in the more urbanized areas of the watersheds.

A more detailed discussion of the physical changes which have taken place along the stream channel and floodplain in recent years is given in Section 4.2.2 of this report.

ii) Hydraulic Structures

Each of the hydraulic structures along the study reaches represents a potential flow constriction which may have a pronounced effect on water surface profiles during flood periods. Therefore, the physical dimensions and elevations of all hydraulic structures were field surveyed as described in detail in the supplementary report. These measurements included the size of the opening and the elevations of the soffit and deck of bridges and culverts.

iii) Crest Gauges

In order to collect peak water level data for the purpose of calibrating the hydraulic routing model, a total of eight crest gauges were installed along Blanche Brook and Warm Creek during the week of September 20, 1982 (4 gauges on each watercourse). Subsequent measurements of peak discharge and water levels were undertaken in an attempt to collect data suitable for model calibration and verification. It should be noted that most of these gauges were vandalized several times during the course of the subsequent field monitoring program, resulting in loss of data. The locations of the gauging stations and data collected are discussed in detail in the supplementary report on the physical surveys and the field program.

The results of the field investigations and river and floodplain characteristics are discussed in the following section.

### 4.2.2 Channel and Floodplain Characteristics

i) General Hydraulic Characteristics

In order to determine the hydraulic conditions of the Stephenville area floodplain, a review of the channel and floodplain characteristics of the two study reaches was undertaken by means of field reconnaissance surveys, and interpretation of the available mapping and background information.

It was determined that the study watercourses can best be characterized according to the following four main reaches:

1) Blanche Brook from its outlet at St. Georges Bay to its confluence with Warm Creek
2) Blanche Brook upstream of its confluence with Warm Creek to the study limit near the Hansen Highway
3) Warm Creek from its confluence with Blanche Brook upstream to Noels Pond

4) Warm Creek above Noels Pond

Typical channel and floodplain characteristics along Blanche Brook and Warm Creek are summarized in Tables 4.1 and 4.2 respectively. Manning's roughness coefficients were selected dependent on relative description and density of a particular type of land use and soil cover, type and amount of vegetation, channel configuration and natural physical constraints relative to the channel and overbank reaches along the watercourse. Typical roughness coefficients were determined based on field observations of channel and overbank characteristics and with reference to the classification derived by Chow (14). A summary of typical Manning's roughness coefficients determined for various reaches of the study area are given in Tables 4.1 and 4.2.

A brief description of the general characteristics of the four main reaches with respect to their hydraulic characteristics is provided below:

1) Blanche Brook : Outlet to Warm Creek confluence

The Blanche Brook floodplain, from its confluence with Warm Creek to its outlet at St. Georges Bay, can be characterized as being a wide floodplain with a relatively flat longitudinal gradient (average slope of 0.0028 m/m). The overbank areas are sparsely vegetated with brush and debris to the north, while being primarily composed of groomed grass lands to the south. Development within the floodplain is limited in this area with some residential areas located to the northwest, the Stephenville Airport and Town Recreation Centre located to the south. An abandoned brewery site is located on the floodplain just downstream of the confluence of Blanche Brook and Warm Creek.

The channel in this reach is winding and relatively steep with an average slope of 0.016 m/m. The channel banks are generally clean and well defined while the channel bed is comprised primarily of stones and cobbles. There is little channel vegetation present due to the high flow velocities common to the watercourse in this area. Localized evidence of bank erosion can be noted in areas along this reach. Erosion of the banks and sedimentation in the lower portions of this reach near the Kin Place Bridge have significantly altered the characteristics of this reach in recent years. While short term natural effects of erosion and sediment movement have altered the hydraulic characteristics of this area, little resulting effect on flood levels in this region is expected due to corrective man-made influences to the region in recent years.

The majority of man-made influences on the channel and overbanks (quite extensive through this region) are comprised of rip-rap protection and construction of earth and roadway berms. Other modifications have been made to about 90% of the banks along this reach. These account for the most extensive changes to this area since the time of the Environment Canada study (27).

2) Blanche Brook : Warm Creek confluence to study limit:

This reach is characterized by overbank areas with an average slope of 0.014 m/m and consisting of a mixture of heavily vegetated regions of trees, brush and residential usage. This mixture combined with the configuration of the floodplain might tend to constrict the passage of flow during peak storm events. The width of the floodplain is relatively broad in the lower portions of the reach with typical widths ranging from 300 m to 500 m. The floodplain then gradually tapers off to a narrower width (20 to 30 m) in the vicinity of the Hansen Highway as the valley section becomes more pronounced and well defined. Residential land use is limited to the fringe areas of the floodplain in this region and the overbanks are comprised mainly of heavy stands of timber and vegetation. Overbank Manning's roughness values were found to range in the order of 0.070 to 0.100 in the areas upstream of the Hansen Highway where there are heavy stands of forest, with medium undergrowth. In localized areas where alder growth is common, the roughness parameter was adjusted to account for constrictions to flow passage (see Table 4.1 for summary of typical Manning's roughness values).

With respect to the channel characteristics, flow is confined to the Blanche Brook channel along the majority of this reach. This is partially accountable due to the extensive areas of man-made influences on the Brook which have occurred in recent years. About 250 m of gabion bank protection are located in the vicinity of Mill Place. The channel in this reach is steep with an average slope of 0.033 m/m, with the channel bed comprised of cobbles, stones, and the occasional boulder. Evidence of high flow velocities can be noted along the reach with scouring and erosion along the channel banks (especially on the overbanks behind the gabion protected areas). Similarly, deposition areas can be found in the channel in the lower, flatter portions of this reach, primarily in the areas of the Main Street bridge and near the confluence with Warm Creek.

With the exception of the extensive amounts of channel protection works and berming which are evident along virtually 100% of the channel length, little appreciable change has occurred to overbank areas since the previous study throughout this reach. The overbank characteristics were found to compare well with those previously noted by Environment Canada, mentioned earlier in this report.

3) Warm Creek : Confluence with Blanche Brook to Noels Pond

This reach is characterized by a moderately wide floodplain, ranging in width from 150 m to 300 m in the Town of Stephenville and narrowing to 60 m to 100 m from Mississippi Drive to the railway spurline. The floodplain then broadens again in the vicinity of Noels Pond to an average width of 170 m. The overbank areas consist of predominantly residential and commercial use in the lower portions of the reach where the slope on the floodplain averages 0.0038 m/m (downstream of Mississippi Drive).

A mixture of groomed lands and densely vegetated regions characterize the upstream areas with an average slope of 0.0035 m/m. Manning's roughness values associated with these densely vegetated areas were found to be high, specifically in the floodplain areas between the transformer sub-station weir and Noels Pond, where there are heavy alder stands and scrub growth and moderate forest cover. This reach is also characterized by some localized areas of swamp and bog in the vicinity of the railway spur line.

Man-made influences such as dykes and berms are predominant along the channel banks below Mississippi Drive, comprising approximately 60% of the channel length through this region. Upstream of this point the natural channel is more well defined and man-made influences are less predominant.

The channel reach is relatively steep throughout with an average slope of 0.024 m/m. In general, the channel can accommodate the occurrence of peak flows through this area while the western overbank has a relatively higher potential flow capacity than does the eastern overbank. This is due to the higher flow conveyance area associated with the eastern overbank.

4) Warm Creek : Noels Pond to the study limit
(see sub-section iii for a discussion of Noels Pond)

Upstream of Noels Pond, the Warm Creek channel is in a natural state and is relatively straight and steep with channel slopes averaging in the order of 0.019 m/m. The channel banks, while relatively clean, are poorly defined in the majority of areas while the channel bed is comprised of stones and cobbles. While there is some channel vegetation in the lower portions of the reach, channel velocities are high enough to prevent growth of any significant vegetation over the channel reach. There are many areas of local tributary inflow to Warm Creek in this area, and areas of ponding water are not uncommon to the lower portions of the reach.

> The overbank and floodplain characteristics in this region are very broad (widths predominantly in in excess of 500 m) and differ substantially from the other reaches in the study area. They are characterized by a mixture of open field and densely vegetated areas of trees and brush (the predominant characteristic of this reach). Furthermore, areas of swamp and bog are common to the reach immediately upstream of Route 490. Similarly, the floodplain in the area of the community of Noels Pond is fairly extensive, with the western overbank having a higher potential flow capacity than the eastern areas. The floodplain throughout this reach is relatively flat, with an average slope of 0.0017 m/m.

In terms of an overview of the entire study area, it is noted that through the Town of Stephenville, the overbank areas are characterized by extensive residential and commercial development. The floodplain in this area is relatively wide, while along the middle and upper reaches of the study area, the floodplain is narrower (with the exception of the area upstream of Noels Pond) with the overbank areas predominantly composed of dense trees and brush. Tables 4.1 and 4.2 and Figure 4.2 summarize the typical channel and floodplain characteristics along the study reaches.

Rapidly varying consecutive sections, bridge and culvert entrances and outlets, floodplain structures, etc. were accounted for through adjustments in the expansion and contraction coefficients, following recommended values found in the HEC-2 manual (45). The general criteria for determining these coefficients are summarized in Table 4.3, together with typical values.

ii) Hydraulic Structures

The discharge and flood levels during peak flows are also controlled in part by some 12 bridges and culverts and two weirs at various locations along the study area. Many of these structures create some constriction to flow and are frequently surcharged, thus increasing the flood risk to residences and businesses in the adjacent floodplain. Table 4.4 provides a summary of the bridge and culvert characteristics for each structure, which can be located by reference to Figure 4.3. The culvert and bridge capacities summarized in Table 4.4 were estimated assuming that there is overflow of the roadway where pertinent. The following briefly describes the hydraulic characteristics of the most potentially serious constrictions on the watercourse:

1) Minnesota Drive Bridge (B2):

The overbank in this area is characterized as being broad and relatively flat. The bridge opening is effectively perched, thus flow may occur over the roadway approach before the actual capacity of the bridge opening is reached. The degree of sedimentation of the structure combined with the effect of the confluence of the two study watercourses immediately downstream of this location, reduces the flow conveyance capacity of this structure. The backwater created by this structure will likely create a broad floodplain with depths in the order of 1 to 2 metres. A high potential for ice and debris jamming at this location also increases the severity of the flood risk in this area, as a review of historical records has indicated.

2) Main Street Bridge (B3):

The existing road approach and opening capacity at this structure is low, thus the road approaches would be severely surcharged during many high flow events, as historical records have indicated. Flood waters back up until surcharging the road, thus increasing the flood hazard to the adjacent low lying overbank areas. This structure is perhaps the most susceptible to ice and debris jamming of any in the study area (see Table 2.1). This structure affects flooding in what is potentially the highest flood risk area along Blanche Brook.

3) Mississippi Drive (B6):

As historical flood records indicate, this structure is also susceptible to surcharging. Low culvert conveyance capacity combined with a high potential for ice and debris jamming at this location have resulted in numerous flood problems to the adjacent lands. Scarring of the overbanks and rip rap protection at the culvert inlet also gives evidence of the high velocities associated with the flooding in this area, and the potential for occurrence of ice and debris jams at this structure.

4) Connecticut Drive Railway Trestle (B7A):

This structure has a low conveyance capacity, resulting in a severe flow constriction. This structure would control outflows from Noels Pond (located just upstream) for most high flow events by dampening out the effect on discharge of the Noels Pond outlet weir. The relationship of this structure on the outflow of Noels Pond is discussed in section 3.3.6.

5) Route 490 culverts at the inlet to Noels Pond (B7B):

While this is a relatively new structure, the flow capacity appears to be low relative to design peaks. It is possible that backwater from this structure may influence flood levels within the Community of Noels Pond (some 2 km upstream). The low conveyance capacity of the culverts, coupled with the accumulation of sediment and debris during flood events have also resulted in partial blockage of the inlet to the structure in the past, which add to the severity of the flood risk in this area. At the time of the field surveys, two of the culverts were effectively blocked by debris (approximately 90% blocked in September, 1982, 40% blocked in July, 1983). The partial blockage of these culverts was found to have some effect on related upstream water levels (±0.4 m) as noted subsequently in Section 4.5.7.

Surcharging at other culverts and road crossings also occurs to a lesser extent at other locations, as identified in Section 4.6 and Appendix D. Additional details on bridges and culverts, including field sketches made at each location are available in the supplementary report.

iii) Channel and Floodplain Storage:

As discussed in Section 3.3.6, the storage available in Noels Pond affects flood levels around the pond and immediately downstream. The Pond has a length of some 1800 m, is 700 m wide on average and has an associated normal surface area of some 130 ha. The inflow/outflow relationships from the reservoir are in part controlled by backwater from downstream structures and by a flow constriction at the inlet.

Inflow is controlled for low frequency storm events by the Route 490 culvert. As noted in the previous section, this structure restricts the passage of contributing upstream flows to the Pond. While this increases the efficiency of Noels Pond in truncating the peak storm discharge and associated downstream water surface elevations, it also increases the flood risk to the upstream reaches.

Similarly, the discharge from Noels Pond is controlled in part by backwater effects from downstream structures. Due to the constrictions to flow and resulting backwater effect of the railway trestle near Connecticut Drive, the effective capacity of the outlet weir is reduced somewhat for high flow events.

The Town of Stephenville maintains the operation of stop log settings at the weir. The pond's primary function is as the municipal water supply source for the Town. Therefore, the pond levels are kept high by maintaining the logs in place in the weir until mid to late October, when they are removed and the pond is allowed to drain. The log settings are replaced in early spring to ensure an adequate water supply will be available for the summer months and to ensure maximum storage potential is available in the pond during the spring freshet.

While no records are maintained on the operation of Noels Pond, discussions with the Town of Stephenville Engineering Department concluded that 8 logs are normally maintained in the weir during the late spring, summer and fall months.

Downstream of Noels Pond, near the railway spur line, there also exists areas of local channel storage as this area is comprised mainly of swamp lands. Similarly, upstream of Route 490 at the inlet to Noels Pond, there is a localized area of swamp and bog. The effect of these storage areas has been incorporated in the channel routing components of the HYMO model.

## 4.3 Structure of Hydraulic Model

In order to simulate the flood levels associated with the 1:20 and 1:100 year peaks, the available background and field data was input to the HEC-2 program. Figure 4.2 shows in schematic form the HEC-2 model structure developed for the hydraulic analysis.

With respect to input of available data, the following criteria were established in order to define cross-section locations and characteristics.

i) All sections are coded as if looking upstream along the watercourse.

ii) Field measured cross-sections as applied in the hydraulic model are referenced to the supplementary field report according to the following numbering system:
   - For Warm Creek cross-sections, the alphabetic characters "WC" are substituted with the number "22". Thus, for example, field survey cross-section WC-18 is identified in the backwater model as section number 2218.
   - For Blanche Brook cross-sections, the alphabetic characters "BBK" are substituted with the number "88". Thus, for example, field surveyed cross-section BBK-12 is identified in the backwater model as section number 8812.

iii) In some cases, field measured cross-sections were used more than once as typical cross-sections along particular reaches. This is to facilitate the accurate coding of bridges and other such constraints at various locations on the watercourses, as described in the program documentation (45). Elevations were adjusted by applying the average slope between measured sections. Repeated sections are denoted in the backwater model by a number to the right of a decimal point following the original section number identification (e.g. section 888.2 would be a repeated section based on the surveyed section number 888, or BBK-8).

iv) All hydraulic structures are referenced to the field survey report through the use of comment cards.

The layout of cross-sections is shown on Figure 4.2 which illustrates in schematic form the application, extent and relative location of the field measured sections as input to the HEC-2 model. The approximate location of the surveyed cross-sections is also given on Figure 4.3.

With respect to the modelling of the hydraulic structures on the watercourse (see Table 4.4), most hydraulic structures were simulated using the special bridge method, as documented in the HEC-2 Users Manual (45). (This option allows a combination of pressure and weir flow to be modelled.) However, alternative techniques were used to model the following structures:

i) Kin Place Bridge (B1)

This structure is perched above the floodplain. Therefore, the normal bridge method was applied (as documented in the HEC-2 Users Manual) since discharge would occur over the approaches of the bridge before a condition of pressure flow through the culvert could be attained.

ii) Main Street Bridge (B3)

The roadway approaches to this structure are lower than the maximum soffit elevation. Therefore, this structure is classified as being perched, and was also modelled utilizing the normal bridge method.

iii) Carolina Avenue Bridge (B5)

This structure was also modelled as a perched bridge utilizing the normal bridge method since the roadway approach is lower than the maximum soffit elevation.

iv) Connecticut Drive Railway Trestle (B7A)

This structure was modelled as a normal bridge for two reasons, firstly, as it is a perched structure and this is generally the most representative approach for modelling this condition given a large difference in road deck elevation versus that of the soffit. Secondly, as the Connecticut Drive bridge is located immediately downstream of the trestle bridge, it was found that utilization of the special bridge method for both structures yielded unrepresentative energy losses over this area. Thus the normal bridge method was utilized in computing water surface elevations at this structure.

v) Route 490 Culvert (B7B)

The multiple opening configuration of this structure is such that the special bridge routine overestimated the flow conveyance capacity of the structure under pressure flow conditions. In order to better represent the hydraulic efficiency of the structure, the normal bridge method was utilized.

Similarly, as two of the culverts are frequently blocked by debris, this was found to most accurately reflect the stage-discharge capacity of the structure under a variety of peak flow conditions.

As stated previously, the special bridge method was applied to all other structures. Thus, the most accurate method for calculating hydraulic losses through each structure was utilized. (Subsequent model calibration simulations substantiated the use of the modelling procedures as described above).

Preliminary HEC-2 simulations indicated that the flow regime was supercritical along part of the Warm Creek study reach. Therefore, backwater simulations were undertaken assuming both subcritical and supercritical flow conditions at this area. That is, a backwater model was developed beginning at the upstream section of the relevant reach and supercritical runs executed. This was done by following procedures which are outlined in the HEC-2 modelling documentation. For a subcritical profile computation, if a channel reach consisting of three or more consecutive sections passes through critical depth, then the supercritical flow profile should be applied. Therefore, given this condition, a combination of both normal and supercritical backwater model simulations was utilized for that section of Warm Creek extending from the upstream face of the Route 460 Bridge to the study limit. It was found that for all practical purposes, subcritical flow profiles were not significantly different from the supercritical profiles in this reach. Therefore, subcritical simulations were used in all subsequent analyses.

Boundary conditions and other model input parameters were determined as follows:

i) All computations were initially performed assuming a condition of free flow with the exception of the Route 490 culverts (as discussed in Section 4.2.2). That is, all hydraulic structures were considered to be free from any temporary obstructions which would reduce their effective discharge capacity during the passing of peak storm flows. The model was subsequently revised during the assessment of ice and debris jam potential to account for blockage at various culverts on the system.

ii) The hydraulic coefficients were derived as previously outlined in Section 4.2.2 and subsequently applied in the sensitivity analysis as discussed in Section 4.5.

iii) The downstream starting elevation at the confluence with St. Georges Bay was assumed to be the maximum recorded tide with a corresponding elevation of 1.5 m, and was used for all flood profile simulations.

## 4.4 Model Calibration and Verification

### 4.4.1 General

In order to most accurately reflect the potential flood conditions along Blanche Brook and Warm Creek, the HEC-2 model was calibrated and verified by comparison to field measured flood levels. The monitoring program and data collected are discussed in the Supplementary Report. Observed water levels were utilized as targets for refining sensitive backwater model parameters.

The general procedures for calibration of the HEC-2 model are summarized as follows:

1) Collect suitable water level and discharge measurements by field monitoring of high flow/stage events at predetermined locations along Blanche Brook and Warm Creek (refer to Figure 4.3 for gauging locations).

2) Isolate the sensitive hydraulic model parameters in order to simulate the documented water levels obtained through step 1).

3) Compare computed water levels (using mean hydraulic coefficients determined in sensitivity analysis) to those recorded.

4) Vary the appropriate hydraulic parameters, as required, and iterate until a suitable comparison between measured and computed water levels is achieved.

5) Validate the refined model using a split sample technique and observed water levels.

As a result of the field monitoring program carried out during this investigation, the following events were documented for subsequent application in the HEC-2 model calibration and verification analysis:

1) A winter rainfall event which occurred during the period of January 11-13, 1983 deposited 59.4 mm of rain at the Stephenville meteorologic station. This, combined with the melt of some 11 cm of snow cover,resulted in a significant stage rise on Blanche Brook and Warm Creek of approximately 1.3 m. A corresponding peak discharge of 64.9 $m^3/s$ was recorded at the Water Survey of Canada gauge. Figures 4.4 and 4.5 show the water levels recorded for this event about the channels. Further documentation on the data collection and results can be found in the supplemental report, "Physical Surveys and Field Program".

2) A summer rainfall event which occurred during the period June 1-3, 1983 deposited 86.2 mm of rain at the Stephenville meteorologic station. The resultant peak discharge, recorded at 2:00 a.m. on June 2, 1983, was noted as being 50.9 $m^3/s$. The recorded water level measurements obtained as a result of this storm are given in Figures 4.4 and 4.5. A more detailed documentation of this event can be found in the supplemental report.

No other events suitable for model calibration and verification were observed during the course of the investigations.

### 4.4.2 Model Calibration

It was evident from the sensitivity analyses that the water levels are most sensitive to the Manning's "n" roughness coefficient and discharge applied in the model. Since discharge is observed, calibration was undertaken by modifying the channel and floodplain roughness coefficients as required. Water level and discharge observations recorded on January 13, 1983 were used to calibrate the HEC-2 backwater model.

The following summarizes the calibration results:

1) The initial uncalibrated backwater model applied in this analysis reflected the mean hydraulic parameters as determined from the sensitivity analysis. It was found that the mean Manning's roughness

values yielded water levels slightly lower than those recorded. The calibrated model reflected roughness values in the order of 10% to 15% higher on average than the mean values derived.

2) The uncalibrated model yielded water levels 0.21 m lower on average than the recorded event.

3) The calibrated model resulted in water levels averaging 0.1 m lower than the recorded water levels.

4) Due to damaged or missing crest gauges at gauge locations 2, 5 and 6 (see Figure 4.3), these areas could not be calibrated directly. However, the mean roughness coefficients at locations 2, 5 and 6 were modified to reflect the results of the calibration results on the other parts of the study reaches.

5) The calibrated model substantiated the flow split upstream of the confluence of Blanche Brook and Warm Creek. The measured discharge at the gauge was transferred along the study reach by ratios as determined in the hydrologic analysis of the watersheds (see Section 3.3). This flow split was found to accurately represent discharge and associated flood level conditions for reaches upstream of the confluence of Blanche Brook and Warm Creek.

Figures 4.4 and 4.5 summarize the results of the backwater model calibration, on Blanche Brook and Warm Creek respectively, to the January 13, 1983 event.

### 4.4.3 Model Verification

The calibrated backwater model was subsequently verified utilizing the discharge and water level observations collected on June 2, 1983. The base hydraulic parameters used in this analysis reflected those determined in the calibrated model.

The results of the hydraulic model parameter verification are shown on Figures 4.4 and 4.5 and are summarized as follows:

1) Due to vandalization of the crest gauges, only those gauges located at gauging stations Nos. 7 and 8 (refer to Figure 4.3 for location) were operative during this event.
2) The verified model yielded water levels within 0.05 m of observed levels.

Additional observations on peak flows and water levels should be collected during severe future flood events in order to undertake further model verification.

Based on the findings of the model verification analysis, it was determined that the backwater model suitably represents the hydraulic characteristics of Blanche Brook and Warm Creek. The 1:20 and 1:100 year flood profiles were then computed utilizing this model, as discussed in Section 4.6.

### 4.4.4 Summary of Backwater Model Testing

The following summarizes the main findings and conclusions of the HEC-2 model calibration and verification:

1) The model calibration and verification has indicated a good comparison (within 0.1 m on average, including tidal variations downstream of Kin Place Bridge) of simulated and observed water levels using the existing (limited) data base.

2) The model testing confirmed the flow split at the confluence of Blanche Brook and Warm Creek (see Section 3.3).

3) The recorded peak flow and water levels are of relatively small magnitude compared to the 1:20 and 1:100 year flood levels. Also, the data collection was incomplete due to vandalism of the gauge network. Additional measurements for severe events should be collected for the purpose of further model testing.

4) It is believed that the calibrated backwater model adequately reflects the hydraulic characteristics of Blanche Brook and Warm Creek and can be used to simulate the 1:20 and 1:100 year water surface profiles for the purposes of this investigation.

## 4.5 Sensitivity Testing

### 4.5.1 General

In order to assess variations in the magnitude of various input parameters on flood profiles along the study reaches, various sensitivity simulations were undertaken.

Based on a review of the initial model simulations, the field reconnaissance survey, and on previous results of backwater modelling on this watercourse (27), and other similar watercourses in Ontario and New Brunswick, the following parameters were determined to be of most importance with respect to definition of flood levels in the study area:

- peak discharge rates along the watercourse
- definition of channel and floodplain roughness coefficients (Manning's 'n')
- the influence of tidal variations at the confluence of Blanche Brook and St. Georges Bay
- expansion and contraction coefficients
- the degree of cross-sectional representation along the study reaches
- presence of ice and debris in the watercourse.

The following sections outline the methodology and results of the sensitivity testing.

### 4.5.2 Sensitivity to Extent of Field Surveyed Cross-sections

In order to assess the sensitivity of the hydraulic model to the degree of cross-sectional data input into the model, simulations were conducted as follows:

1) With a model comprised only of the field surveyed cross-sectional data

2) With the above model revised to incorporate 6 of the sections measured during previous studies (27), as well as having measured cross-sections repeated and adjusted by bedslope (maximum change in water surface elevations of 0.5 m between consecutive cross-sections and a more gradual computation of energy losses)

3) Finally, the initial model (containing only the surveyed cross-sections) was constructed with the provision for the HEC-2 model to compute internally interpolated sections based on a maximum allowable loss in velocity head of 0.25 m.

The resulting water levels computed for each of these cases were then compared to determine the relative sensitivity of changes to the flood profile.

It should be noted that in all cases, the mean values for discharge and roughness coefficient as given in Tables 4.5 and 4.6 respectively were used in the sensitivity testing on the extent of field measured cross-sections.

By assuming the first condition as the base (that is, only field measured cross-sections applied), it was found that on average the second condition yielded slightly lower water levels compared with a mean variation in computed water levels of about 0.02 m and a range of about 0 m to 0.22 m. Similarly, it was noted that the third condition also yielded results slightly lower than field measured only, with an average difference in water levels of about 0.01 m over a range of approximately 0 m to 0.15 m.

In comparing only the second and third condition, it was noted that the range of difference in water levels was about 0 m to 0.18 m and averaging about 0.01 m, with the third condition yielding slightly higher results on average.

Based on this analysis, it was concluded that the original model schemetization and cross-section locations provides an acceptable degree of accuracy in simulating backwater profiles. Interpolated cross-sections were not found to be necessary for the purposes of this study.

### 4.5.3 Sensitivity to Computed Peak Discharge

Sensitivity simulations were conducted utilizing the computed 1:100 year peak discharge versus the 1:100 year peak discharge at the upper and lower 95% confidence limits, according to the values summarized in Table 4.5.

As expected, the peak discharge had some effect on variation of water levels, with the average difference being about 0.48 m, above the mean at the +95% confidence level with differences ranging from 0.09 m to 2.91 m. Similarly, peak flows at the -95% confidence level resulted in an average decrease of about 0.74 m in water levels compared to those computed for the mean, with a range of 0.17 m to 1.98 m. The smallest differences were found to be associated with the steeper stream reaches while the largest differences were found to be in the downstream floodplains. The more extreme differences are found at hydraulic structures, where the effective discharge capacity of the structure directly affects the sensitivity to discharge.

In summary, it was found that the average difference of the median profile from the ±95% ("upper and lower profiles") was found to be approximately 0.62 m along the study reach.

Comparing these analyses to sensitivity tests discussed in subsequent sections, it was evident that changes in flow had the most effect on water levels computed for the entire length of both watercourses, with Blanche Brook found to be the most sensitive of the channels to variation in discharge. However, the large variations in discharge resulted in relatively smaller variations in water level profiles along the study reach.

### 4.5.4 Sensitivity to Tidal Influences

The HEC-2 model requires the definition of initial starting levels along the study reach. This, in turn, accounts for possible backwater effects on water levels in the lower portions of a channel reach. Sensitivity simulations were undertaken to determine the effect of variations in the initial water levels on flood levels in the lower portion of Blanche Brook which may result from the tidal influence at St. Georges Bay.

For the purpose of this investigation, the range of water levels at the Brook outlet was chosen as 0.55 to 1.5 m. This represents a range from the mean high tide to the maximum recorded tide at the outlet of the Brook. As little tidal information is available for Stephenville, the values chosen were derived through transposed recorded tide levels for the Harrington Harbour station, as supplied by Fisheries and Oceans Canada (30). Approximate geodetic tidal elevations were obtained by adjusting the tidal levels by the mean water level for the secondary port at Port Harmon.

It was evident from this analysis that tidal influence is not a significant factor with respect to water levels along the Blanche Brook or Warm Creek watercourses. The water surface profile below the Kin Place Bridge (Recreation Centre) is the only area subject to any tidal influence. The average difference in water levels downstream of the bridge was about 0.01 m, and ranging from 0 to 0.03 m (excludes starting level at downstream section). This reach is characterized by a relatively broad floodplain and a channel with high banks. (It was also found that during peak flow conditions tidal variations have very little effect on the areal extent of the floodplain associated with Blanche Brook.)

Based on the sensitivity backwater profiles, it is noted that the Kin Place Bridge partially controls upstream water levels under high flow events, since the flow goes through critical depth at this location. Tides do not affect flood levels upstream of the Kin Place Bridge.

### 4.5.5 Sensitivity to Roughness Coefficient

Manning's roughness coefficients for the channel and floodplain were determined as described in Section 4.2.2. The sensitivity of the flood profile computations to variations in roughness coefficient was undertaken as a means of substantiating the accuracy of the backwater model prior to subsequent model calibration and verification.

The range of Manning's "n" values applied in the analysis are summarized in Table 4.6. The discharge value used in the sensitivity testing corresponded to the median 1:100 year estimates of peak flow as given in Table 4.5.

The range of "n" values given in Table 4.6 corresponds to the range of potential values as described by Chow (14) applied to the channel characteristics of the study reach. For the purposes of these sensitivity tests, it was determined that the roughness coefficients could vary ±20% about the "mean value" previously discussed.

The average difference in water levels from the mean were found to be about +0.09 m and -0.08 m corresponding to +20% and -20% changes in the roughness coefficients respectively. The corresponding range of differences were found to be about 0.01 m to 0.45 m and 0.00 to 0.36 m respectively. Similarly, the corresponding average difference between the upper and lower range was about 0.17 m along the study reaches, with differences of from 0.03 m to 0.79 m. The largest differences were noted in the lower reaches where the stream channel and floodplain slopes are somewhat flatter, for example, from Kin Place Bridge upstream to Minnesota Drive and from Minnesota Drive upstream on Blanche Brook to the vicinity of the shopping mall. Along Warm Creek the most sensitive areas were found to be downstream of Carolina Avenue.

On the basis of these tests, it was concluded that there is some variation in peak flood levels along the watercourses as a result of possible variations in roughness coefficients, although not as significant as variations in discharge. The sensitivity analysis verifies the need for accurately selecting roughness coefficients for application in the backwater modelling of the 1:20 and 1:100 year design flood events.

#### 4.5.6 Sensitivity to Expansion and Contraction Coefficients

Sensitivity simulations were also undertaken in order to assess the effect of variations in expansion and contraction coefficients on the accuracy of the backwater model. Simulations were conducted by varying the expansion and contraction coefficients about a range of ±50% from the typical mean values given in Table 4.3. From this analysis, it was found that negligible differences in water levels resulted (variations less than ±0.10 m resulting about the mean). It was, therefore, concluded that flood profiles along the study reaches are relatively insensitive to variations in expansion/contraction coefficients. The mean values given in Table 4.3 were, therefore, utilized for the purposes of this investigation.

#### 4.5.7 Sensitivity to Ice and Debris Jams

Ice jams generally form at constricted sections and locations where irregularities occur. Some characteristic locations where ice jams could initiate include:

1) At a transition zone between a rapidly flowing stream reach and a section of more tranquil flow. On Blanche Brook this occurs at the confluence with Warm Brook and at the confluence with St. Georges Bay;
2) At channel singularities such as shoals, changes in alignment, constrictions in the flow and other channel obstructions (see Section 4.2.2 for a discussion of channel obstructions affecting ice jams);
3) At locations of hydraulic structures such as bridges, weirs, and dams along the stream (see Section 4.2.2);
4) At locations where significant accumulations of anchor ice have formed along the stream. (No specific sites have been identified based on existing information).

If the flow velocity and Froude number are low (say below 0.75 m/s and 0.08 respectively) (16, 34, 42), then static ice jams will form and remain in place, allowing water passage beneath it. Such jams would have relatively low flood damage potential, being characterized by fairly uniform increases in water level along their length. (Such flow conditions exist in Noels Pond.)

However, "dynamic" ice jams may evolve from unstable forms of simple or static ice jams. Dynamic jams may be formed at a flow obstruction or channel irregularity where a heavy run of ice is suddenly stopped, such as undersized bridges or culverts. Higher flow rates and Froude number may also cause ice floes to underturn at the leading edge of a downstream ice cover, thickening the jam by shoving, breaking and crushing. The movement of the jam might continue until balanced by internal forces or until the jam catches on bottom irregularities, resulting in the formation of a so-called "dry jam". Both types of jams may remain in place until the river discharge changes significantly, or until the strength of the ice is weakened due to warm weather.

The flow velocity and depth values calculated from the HEC-2 model were utilized to estimate Froude numbers in order to assess the potential for ice jam formation. In general, it was found that the average velocity and Froude No. along the study reach at historical ice jam locations was found to be over 2.0 m/s and 0.3 respectively. This indicates a high potential for severe ice jam formation and growth at the historical ice jam locations.

In order to assess the degree of sensitivity of the watercourses to the presence of ice and debris in the channel, the hydraulic model was revised to incorporate the following conditions:

i) The conveyance area of the several structures was assumed to be reduced by 50% to simulate ice or debris blockage. (This assumption was verified by comparative analyses - see subsequent discussion). It was assumed that blockage could occur in the future at the following locations where historical jams have been observed:

- Minnesota Drive Bridge
- Main Street Bridge
- Mississippi Drive Bridge
- Culverts under Route 490
- Route 460 Bridge

ii) An ice cover of 0.15 m thickness was also assumed for a short reach upstream of each structure. A continuous cover was assumed upstream of Route 490 since ice movement would be restricted by the flatter gradient of the channel and floodplain.

The selection of jam locations and degrees of ice cover were based on discussions with local residents and on historical data collected as part of this study, as well as through utilization of several theoretical techniques.

It was assumed that a stable ice cover would form on Noels Pond and that severe jams at the outlet are not likely. (Flow velocities through Noels Pond are very low.)

Alternative calculations of ice related water levels to those associated with free flow conditions were also conducted for comparison purposes and for various flow events using Froude number analyses and techniques developed by Beltaos (8), and through utilization of the HEC-2 ice subroutine. These levels were subsequently compared to those obtained by the backwater modelling. Similarly, the analysis was compared to the associated conditions resulting from the March, 1979 ice related flood on Blanche Brook. Based on this assessment, it was found that all methods yielded comparable results, thus substantiating the assumptions made in the sensitivity analyses on ice and debris jams. It was also noted that the theoretical calculations yielded higher results than the 50% assumed blockage backwater results (but may not be valid due to floodplain relief).

Based on this analysis it was noted that flood levels on both watercourses are sensitive to the presence of ice and debris in the channel. Water surface elevations over the affected reaches were increased on average by 0.9 to 1.2 m from those associated with free flow conditions. In some cases (where weir flow over the roadway approaches occurs), the effect of blockage on the upstream flood levels is relatively small since a small increase in water level results in a large increase in flow capacity. Also, the increase in water level due to blockage is generally a local effect at the culverts and bridges, which usually extends upstream only 150 m to 200 m. depending on local channel and floodplain characteristics. The exception to this is upstream of Route 490 where the influence of blockage at the culverts extends upstream to the Community of Noels Pond. The profiles for the 1:100 mean condition; and the 1:100 upper 95% with and without blockage are compared on Figures 4.6 and 4.7.

### 4.5.8 Summary of Results and Conclusions of Sensitivity Testing

The following points summarize the main findings and conclusions of the sensitivity analyses on computed flood profiles along the study watercourses:

1) The flood profiles along the watercourses were relatively more sensitive to variations in peak discharge, as represented by the following confidence limits:
   - average difference above the mean for +95% confidence limit was 0.48 m (range 0.09 to 2.91 m)
   - average difference below the mean for -95% confidence limit was 0.74 m (range 0.17 to 1.98 m)

2) Flood profiles along Blanche Brook are more sensitive to variation in discharge than are levels along Warm Creek

3) The sensitivity analysis on the flood profiles along the watercourses to variation in roughness coefficient resulted in relatively smaller changes in flood elevations (average of 0.17 m over ±20% change in 'n' values). The flood profiles along the watercourses are, therefore, somewhat sensitive to variations in roughness coefficient.

4) The influence of tidal variations are felt only in the extreme lower portion of the watercourse; specifically downstream of the Kin Place Bridge. This has almost no effect on the areal extent of the floodplain along the lower reaches of Blanche Brook.

5) The flood profiles were not sensitive to variations in the expansion and contraction coefficients.

6) The field measured cross-sections were found to provide an accurate representation of the flood profile along the study reach, and hence the use of interpolated sections was not considered to be necessary.

7) The flood profiles are sensitive to the presence of ice and debris jams in the channel, with related water surface increases of as much as 0.9 to 1.2 m. However, the effect of ice and debris jams is fairly localized due to the steepness of the stream channel and floodplain. (See Figures 4.6 and 4.7 for the location of areas affected by ice and debris jams.)

## 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 a recurrence interval of 1:20 and 1:100 years.

The hydrologic analyses described in Section 3.0 resulted in the determination of 1:20 and 1:100 year instantaneous peak discharge values for the study area (see Table 3.6). The sensitivity testing described in Section 4.5 confirmed the importance of accurate discharge and hydraulic parameter estimates.

The HEC-2 backwater model was developed as discussed in Section 4.3 and a schematic of the overall model structure is given on Figure 4.2. Channel and floodplain characteristics were determined as discussed in Section 4.2.2. The calibration and validation undertaken has increased the level of confidence in the ability of the backwater model to accurately simulate flood profiles.

The boundary conditions and model structure are discussed in detail in Section 4.2. The following briefly outlines the main assumptions in the application of the calibrated model for the simulation of the 1:20 and 1:100 year flood profiles:

1) Water level profiles were computed assuming a subcritical flow condition. While supercritical flow was encountered for some short reaches of Warm Creek (at the upstream study limit), sensitivity testing demonstrated insignificant differences when supercritical vs. subcritical were compared in these reaches.

2) Water levels in Noels Pond were modelled by reservoir routing techniques. The results of the HYMO routing simulations were used to establish the water levels in Noels Pond for the 1:20 and 1:100 year peak discharge (found to be 24.35 m and 24.5 m respectively). These water levels were used as the starting elevations for calculating the flood profiles upstream of Noels Pond on Warm Creek.

3) The hydraulic coefficients used in the development of the 1:20 and 1:100 year flood profiles were those as calibrated and verified to recorded events.

4) All bridges, culverts and hydraulic constraints were assumed free of any temporary obstruction which may reduce the hydraulic discharge capacity. The effect of such obstructions on flood profiles is discussed in Section 4.5.7. (Flood profiles with blockage for the 1:100 year upper 95% confidence limits were also determined for comparative purposes.

5) Peak flows summarized in Table 3.6 were used in determining the 1:20 and 1:100 year flood profiles.

Upstream of Main Street along Blanche Brook, the flood hazard is relatively low, as the 1:20 and 1:100 year flood lines do not encroach on existing urban developments. The only exception to this being in the area south of White's Avenue and near the Jehovahs Witness Hall, where a moderate flood risk is associated with the 1:100 year event. Few buildings are directly affected by flooding, however, access routes would be affected during flood conditions. It should be noted that this region is sensitive to ice and debris jamming. The associated flood risk in this area is greatly increased under these conditions.

ii) Warm Creek

A moderate to high flood hazard exists in the reach along Warm Creek from Carolina Avenue to Noels Pond. While high magnitude floods would primarily affect the access routes and recreational facilities located in this area, localized urban pockets such as near Mississippi drive may be inundated as a result of the 1:20 and 1:100 year floods. This, coupled with the presence of ice or debris in the channel, creates a flood hazard to the area, especially with respect to potential future development in the region.

A potential for a small amount of spill from Noels Pond was found to exist for the 1:100 year event (see Figure 5.1). Upstream of Noels Pond, the flood hazard is higher since overtopping of the two main highways to the Town of Stephenville would likely occur under both the 1:20 and 1:100 year events. Residential buildings located on the eastern fringe of the community of Noels Pond would be subject to moderate to high flood risks. Similarly, large areas of agricultural lands would be inundated under most high discharge events. Thus, the flood risk to the downstream reaches would increase as debris such as downed trees or fences may be washed downstream and block culvert or bridge openings.

The flooding of developed areas as described above relates to overland flow in the floodplain areas. In addition, some basements adjacent to the floodplain may also be flooded due to infiltration. However, the determination of this type of basement flooding was beyond the scope of the present study. Also, the first floor elevations and structure openings of all potentially flood prone structures must be field surveyed should it be necessary to determine flood damages in any possible future flood control investigations.

As an indication of the areal extent of flooding, the average top width of flooding (in those areas where the channel capacity was exceeded) was found to be about 175 m, compared to an average channel top width of 24 m. The main flooding hazards along the upstream reaches would appear to be related to inadequate discharge capacities of the hydraulic structures in the areas, while flooding in the downstream reaches is due to both under-designed bridges or culverts (with respect to flow capacity) and the limited discharge capacity of the channel. Similarly, the presence of ice and debris accumulating in the channel at the entrances to these structures will substantially increase the local flood risk by increasing free flow water levels by as much as 1.5 m.

## 4.7 Main Conclusions and Recommendations of Hydraulic Analyses

The HEC-2 backwater model was successfully utilized to calculate flood profiles along the study reach using channel and floodplain characteristics determined from the field surveys, and subsequently calibrated/ verified by the field monitoring program.

Based on the results of the foregoing on the study watersheds, the following main conclusions and recommendations are noted:

Conclusions:

1) Flood profiles along Blanche Brook and Warm Creek are primarily sensitive to the following parameters:
   - variation in discharge
   - variation in channel and floodplain roughness coefficient
   - presence of ice or debris jamming in the watercourse during the occurrence of peak flows.

2) Testing of the backwater model by comparison to observed flood levels has confirmed the accuracy of the backwater simulation.

3) Supercritical flow profiles were encountered at various locations along the study reach. However, the difference between subcritical and supercritical flow simulations was found to be relatively insignificant along these reaches (differences less than 0.1 m on average). Therefore, the subcritical flow simulations accurately reflect the design flood levels about the study reaches.

4) The water surface elevations through Noels Pond should be taken from the HYMO simulations on the watercourse. The 1:20 and 1:100 year water levels in Noels Pond are 24.35 m and 24.5 m respectively.

5) The computed 1:20 and 1:100 year hydraulic profiles utilizing the calibrated parameter values input in the model are given in Figures 4.6 and 4.7 for Blanche Brook and Warm Creek respectively. The water surface elevations for these events are also tabulated in Appendix D.

6) Man-made flow restrictions exist at several stream crossings. This includes the stream crossings at Main Street and Carolina Avenue, the bridge at Mississippi Drive, and the Route 490 culverts.

Recommendations:

1) The potential for significant ice jams along Blanche Brook and Warm Creek exists mainly at man-made flow restrictions. The structures located at Main Street, Carolina Avenue, Mississippi Drive and Route 490 should be enlarged to reduce the potential for future ice and debris jam formation. In the interim, a maintenance program should include removal of any accumulated ice or debris at these and any other similar locations where such accumulations may occur from time to time.

2) Additional hydraulic information should be collected for the purpose of further model verification. This includes data on ice jam formation and ice thickness, and degree of blockage at jam susceptible structures during future high flow events.

3) Additional field measurements of discharge and water levels associated with severe floods should be collected for the purposes of further model verification. This will become more important as changes to the watercourse continue to be made.

4) The 1:20 and 1:100 year flood profiles for Blanche Brook and Warm Creek are summarized in Figures 4.6 and 4.7 and the water levels are given in a tabular summary in Appendix D. It is recommended that these profiles be adopted as the 1:20 and 1:100 year water surface elevations for the study reaches, and thus utilized for future regulation of development along Blanche Brook and Warm Creek. Similarly, the aerial extent of flood hazard lands as shown on the 1:2500 scale mapping should be adopted as identifying the potential extent of flooding within the study area.

5) A potential for "spill" from the floodplain exists on Blanche Brook and Warm Creek in the reaches downstream of Main Street (Carolina Avenue). Similarly, local areas on Warm Creek such as Mississippi Drive and at Noels Pond near the airport runway are susceptible to "spill" during high flood stage events. A detailed assessment of the spill patterns in these areas, however, was beyond the scope of this study. It is, therefore, recommended that such an analysis be carried out to accurately define the areal extent and possible flood damages associated with the potential spill of flood waters at these locations.

6) The HEC-2 model can be utilized to determine the effect of proposed remedial measures on the 1:20 and 1:100 year flood profiles and the associated areal extent of flooding.

TABLE 4.1

Typical Channel and Floodplain Characteristics
Along Blanche Brook

| | Reach | Typical Channel Characteristics: Depth (m) | Width (m) | Slope (m/m) | Material | Typical Floodplain Characteristics: Width (m) | Slope (m/m) | Vegetation Conditions | No. of Bridges & Structures | *Typical Manning's "n": Left Overbank | Channel | Right Overbank |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | Outlet to Minnesota Dr. (Warm Creek Confluence) | 4-5 | 30 | 0.016 | - stone & cobbles<br>- some pools & weeds<br>- some sand | 100 | 0.0038 | - sparse vegetation<br>- grass land, some scrub, bush and debris | 1 | 0.040 | 0.040 | 0.040 |
| 2a | Minnesota Dr. to Main St. | 2.5-3 | 30 | 0.009 | - stone & cobbles | 340 | 0.0114 | - sparse vegetation<br>- grass & scrub<br>- some commercial landuse | 2 | 0.050 | 0.045 | 0.050 |
| 2b | Main St. to Hansen Hwy. | 2.5-3 | 25 | 0.033 | - stone & cobbles<br>- some boulders<br>- some rip rap & gabion protection on banks | 30 | 0.0114 | - grassland & pavement<br>- scrub & moderate to dense vegetation<br>- some debris<br>- residential land use on fringe | 1 | 0.055 | 0.040 | 0.070 |
| | Upstream of Hansen Hwy. | 2-4 | 30 | 0.013 | - stone & boulder<br>- some cobbles | 35 | 0.021 | - dense timber stands<br>- dense brush, scrub | 0 | 0.100 | 0.055 | 0.100 |

* For detailed description of Manning's "n" over the reach, refer to hydraulic model computer listing.
(Note: These values also reflect the results of model calibration and verification)

TABLE 4.2

Typical Channel and Floodplain Characteristics
Along Warm Creek

| Reach | Typical Channel Characteristics | | | | Typical Floodplain Characteristics | | | No. of Bridges & Structures | *Typical Manning's "n" | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | Depth (m) | Width (m) | Slope (m/m) | Material | Width (m) | Slope (m/m) | Vegetation Conditions | | Left Overbank | Channel | Right Overbank |
| 3a Blanche Bk confluence to Mississippi Drive | 2-2.5 | 20 | 0.024 | - stone & cobbles<br>- some sand | 200 | 0.0038 | - grassland & pavement<br>- moderate vegetation scrub<br>- Alder growth on banks | 2 | 0.040 | 0.040 | 0.045 |
| 3b Mississippi Dr. to Railway Spur line | 1-2 | 15 | 0.015 | - stone & cobbles | 70 | 0.0035 | - grassland, pavement (west)<br>- dense veget. (east)<br>- heavy Alder growth on banks | 2 | 0.050 | 0.045 | 0.060 |
| 3c Railway Spur Line to Noels Pond | 1.5-3 | 20 | 0.010 | - stone & cobbles<br>- some weeds and sand | 130 | 0.0035 | - grassland & scrub<br>- dense timber stands<br>- swamps & sluggish reaches | 3 | 0.050 | 0.045 | 0.075 |
| 4a Noels Pond to Rte 460 (Hansen Hwy) | 1-2 | 15 | 0.021 | - sand & cobbles<br>- some stones | 500 | 0.0017 | - high grass & brush<br>- some trees, debris | 3 | 0.045 | 0.040 | 0.065 |
| 4b Upstream of Route 460 | 1-1.5 | 22 | 0.021 | - sand & cobbles<br>- some stones | 200 | 0.011 | - pasture type grassland<br>- areas of moderate vegetation | 0 | 0.080 | 0.045 | 0.035 |

* For detailed description of Manning's "n" over the reach, refer to hydraulic model computer listing.
(Note: These values also reflect the results of model calibration and verification)

TABLE 4.3
Summary of Typical Expansion
and Contraction Coefficients *

| Parameter | Range of Typical Values |
| --- | --- |
| Expansion Coefficient: | |
| i) Gradually varying sections | 0.3 |
| ii) Rapidly varying sections and hydraulic constraints | 0.5 |
| iii) Abrupt variations between sections and severe hydraulic constraints | 0.6 - 1.0 |
| Contraction Coefficients: | |
| i) Gradually varying sections | 0.1 |
| ii) Rapidly varying sections and hydraulic constraints | 0.3 |
| iii) Abrupt variations between sections and severe hydraulic constraints | 0.5 - 0.8 |

NOTE: * Source (45)

TABLE 4.4

Bridge and Culvert Characteristics

| Location | Type of Structure | Elevations *Road (m) | Elevations Soffit (m) | Area of** Opening ($m^2$) | Approximate Capacity ($m^3/s$) |
|---|---|---|---|---|---|
| B1 - Kin Place | Conc. Box Culvert | 3.74 | 3.86 | 45.6 | 175 |
| B2 - Minnesota Drive | Bridge | 6.82 | 7.29 | 58.7 | 54 |
| B3 - Main Street | " | 9.94 | 10.68 | 42.0 | 52 |
| B4 - Hansen Hwy. | " | 30.78 | 29.74 | 62.4 | 200 |
| B5 - Carolina Ave. | Conc. Box Culvert | 8.35 | 8.58 | 20.1 | 50 |
| B6 - Mississippi Dr. | Conc. Box Culvert | 10.09 | 9.98 | 15.1 | 39 |
| W1 - Transformer Sub-Station | Weir | 14.47 | N/A | N/A | N/A |
| C1 - Railway Spur Line | 4 CMP Culverts | 22.47 | 21.05 | 4.7 | 18 |
| B7A - Connecticut Dr. Railway Trestle | Bridge | 22.93 | 23.15 | 19.6 | 32 |
| B7 - Connecticut Dr. | Bridge | 24.04 | 24.07 | 65.1 | 50 |
| W2 - Noels Pond Outlet | Weir | 21.97 | N/A | N/A | f(t/water)*** |
| B7B - Route 490 | 3 CMP Culverts | 25.81 | 23.48 | 31.9 | 137 |
| B7C - Route 490 Railway Trestle | Bridge | 24.9 | 24.4 | 18.9 | 96 |
| B8A - Route 460 Railway Trestle | Bridge | 26.20 | 26.10 | 20.8 | 98 |
| B8 - Route 460 | Conc. Box Culvert | 26.20 | 25.87 | 28.6 | 29 |

NOTES:

* Denotes the minimum elevation of the roadway approach or bridge deck, dependant on which controls the potential for surcharge over the roadway

** Assuming flow not contained (flow over roadway)

*** Denotes that capacity is significantly affected by tailwater

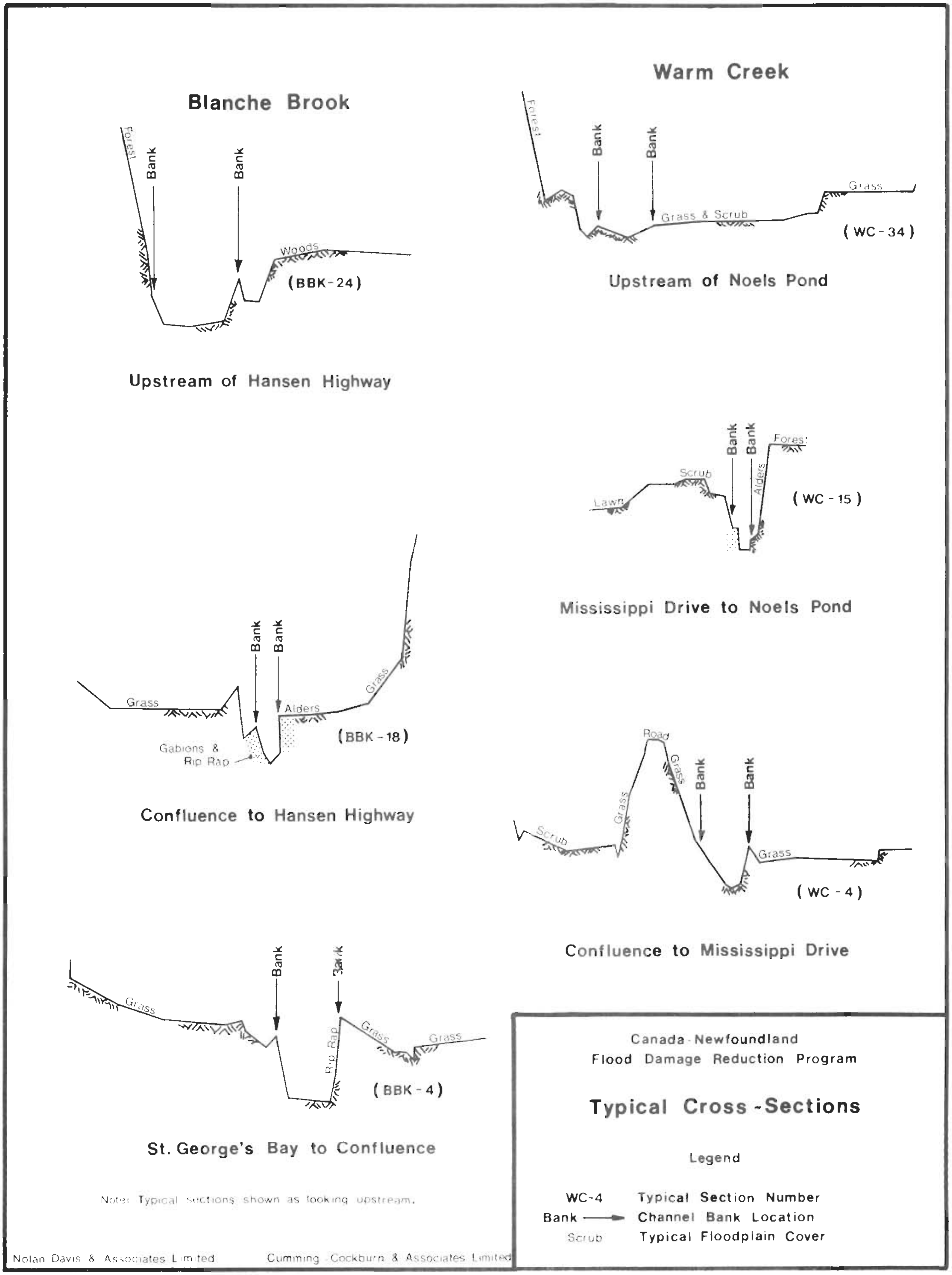
TABLE 4.5
Maximum Instantaneous Discharge Applied
in Sensitivity Analysis

| Section No. | Location | 1:100 Year Discharge (-95% C.L.) | 1:100 Year Discharge $QP_{100}$ ($m^3/s$) | 1:100 Year Discharge (+95% C.L.) | 1:20 $QP_{20}$ ($m^3/s$) |
|---|---|---|---|---|---|
| Blanche Brook: | | | | | |
| 881 | Outlet | 87.9 | 166.5 | 245.0 | 112.0 |
| 888 | Warm Creek Confluence | 86.9 | 165.6 | 244.8 | 110.8 |
| 887.1 | Minnesota Drive | 61.1 | 106.8 | 152.7 | 79.4 |
| 8822.1 | Hansen Hwy. | 44.6 | 78.4 | 113.3 | 58.1 |
| Warm Creek: | | | | | |
| -888 | @ Confluence with Blanche Brook | 33.4 | 68.1 | 98.8 | 48.6 |
| 2224.3 | Noels Pond Outlet weir | 33.7 | 68.6 | 98.9 | 50.6 |
| 2225.1 | Noels Pond Inlet (Route 490) | 46.9 | 83.5 | 126.9 | 59.5 |
| 2231 | @ Tributary d/s of Route 460 | 48.1 | 82.7 | 118.3 | 62.0 |
| 2231.1 | Route 460 | 39.8 | 68.2 | 98.4 | 51.2 |

TABLE 4.6

Summary of Roughness Coefficients Used in Sensitivity Analysis

| Reach | Mannings Roughness Coefficient | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | Left Overbank | | | Channel | | | Right Overbank | | |
| | -20% | n | +20% | -20% | n | +20% | -20% | n | +20% |
| **Blanche Brook:** | | | | | | | | | |
| Outlet to Minnesota Dr. (Warm Creek Confluence) | 0.030 | 0.040 | 0.050 | 0.025 | 0.035 | 0.045 | 0.030 | 0.040 | 0.050 |
| Minnesota Drive to Main Street | 0.030 | 0.040 | 0.050 | 0.030 | 0.040 | 0.050 | 0.030 | 0.040 | 0.050 |
| Main Street to Hansen Hwy. | 0.035 | 0.045 | 0.055 | 0.030 | 0.040 | 0.050 | 0.050 | 0.060 | 0.070 |
| Upstream of Hansen Hwy. | 0.095 | 0.100 | 0.120 | 0.045 | 0.055 | 0.065 | 0.095 | 0.100 | 0.120 |
| **Warm Creek:** | | | | | | | | | |
| Blanche Brook Confluence to Mississippi Drive | 0.030 | 0.040 | 0.050 | 0.030 | 0.040 | 0.050 | 0.035 | 0.045 | 0.055 |
| Mississippi Drive to Railway Spur Line | 0.050 | 0.060 | 0.070 | 0.035 | 0.045 | 0.055 | 0.060 | 0.070 | 0.080 |
| Railway Spur Line to Noels Pond | 0.040 | 0.050 | 0.060 | 0.035 | 0.045 | 0.055 | 0.075 | 0.095 | 0.115 |
| Noels Pond to Route 460 (Hansen Hwy.) | 0.040 | 0.050 | 0.060 | 0.025 | 0.035 | 0.045 | 0.050 | 0.060 | 0.070 |
| Upstream of Route 460 | 0.080 | 0.100 | 0.120 | 0.025 | 0.035 | 0.045 | 0.025 | 0.035 | 0.045 |




Canada-Newfoundland
Flood Damage Reduction Program

**Typical Cross-Sections**

Legend

| | |
|---|---|
| WC-4 | Typical Section Number |
| Bank ⟶ | Channel Bank Location |
| Scrub | Typical Floodplain Cover |

**Figure 4.1**

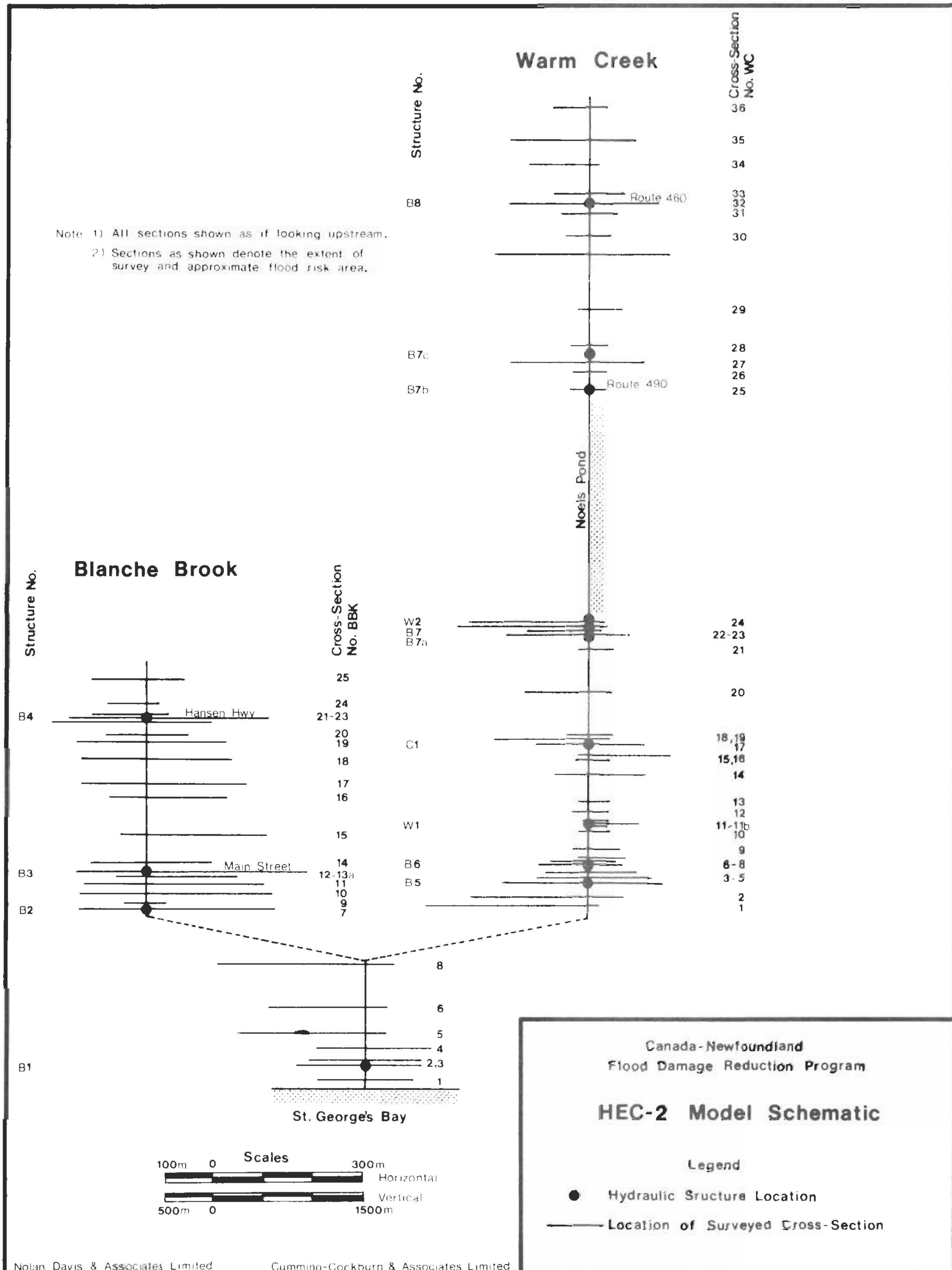

Warm Creek
Structure No.
Cross-Section No. WC
Route 460
Route 490
Noels Pond
Note 1) All sections shown as if looking upstream.
2) Sections as shown denote the extent of survey and approximate flood risk area.
Blanche Brook
Structure No.
Cross-Section No. BBK
Hansen Hwy
Main Street
St. George's Bay
Scales
100m 0 300m Horizontal
Vertical 500m 0 1500m
Canada-Newfoundland
Flood Damage Reduction Program
HEC-2 Model Schematic
Legend
Hydraulic Sructure Location
Location of Surveyed Cross-Section
Nolan Davis & Associates Limited
Cumming-Cockburn & Associates Limited

Figure 4.2

# Remedial Measures 5.0

# Remedial Measures 5.0

## 5.1 General

On the basis of the identification of historical flooding and utilizing the results of the 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 Blanche Brook and Warm Creek.

Broadly speaking, the basic elements for a flood damage reduction plan can be classified as:

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 a number of alternative remedial measures for further future detailed consideration, as shown on Figure 5.1 and discussed in the following sections.

## 5.2 Identification of Structural Measures

The following structural flood control measures should be considered in order of priority:

1) Replacement of Route 490 Culverts:
The feasibility of reconstructing the Route 490 crossing over Warm

Creek should be considered. The existing alignment of three corrugated metal pipes should be replaced by a wide span bridge to improve the hydraulic characteristics and increase the flow conveyance capacity.

The hydraulic efficiency of the structure would be increased such that the potential for debris jamming at this location would be greatly reduced. This, in turn, would reduce the potential flood hazard in the community of Noels Pond.

Alternatively, further hydraulic investigations could be undertaken to determine if it is possible to modify the existing structure in order to increase hydraulic efficiency and reduce the potential for debris jamming.

These investigations should also include analysis of any possible adverse effects at downstream locations.

2) Replacement of flow obstructions:
The following should be considered with respect to the increased flow conveyance capacity and the potential for reduction in future occurrences of ice and debris jams and floodplain spill at these locations.
   a) The replacement of the culvert and/or raising of the road at Mississippi Drive on Warm Creek
   b) The replacement of the bridge crossing at Carolina Avenue on Warm Creek
   c) Replacement of the Minnesota Drive bridge at the confluence of Blanche Brook with Warm Creek.

3) Upstream detention on Blanche Brook:
This would consider the installation of a dyke and outlet control weir over the natural valley section upstream of the Hansen Highway. The installation of such a structure would serve to attenuate the peak flows experienced along the downstream reaches of Blanche Brook. In assessing the feasibility of this scheme, the following should be examined:

a) The effect on the timing arrival of the peak flows from Warm Creek to Blanche Brook with respect to the present condition.
b) An assessment as to the potential flood hazard should such a structure fail under high runoff events.
c) The suitability of the proposed site with respect to environmental concerns.
d) The maximum level of control provided to attenuating peak inflows and subsequent downstream water levels.

4) Dykes or flood berms along selected reaches of the channel:
The feasibility of constructing a flood control berm along the stream reach immediately downstream of Main Street on Blanche Brook and to the west of Warm creek downstream of Mississippi Drive should be considered.
- About 900 m of berm might be required along the southwest bank of Blanche Brook down to just east of West Street.
- Some 320 m of berm to the east of Blanche Brook from Main Street to the Warm Creek confluence.
- Some 550 m of berming would be required on the east bank of Warm Creek downstream of Mississippi Drive.

5) Raising of Minnesota Drive:
The possibility of raising Minnesota Drive to allow the road embankment to act as a dyke (from Mississippi Drive to the confluence) to minimize flooding of the downstream residential areas should also be investigated together with the possibility of enlarging or twinning the Mississippi Drive and Carolina Avenue culverts. The raising of Minnesota Avenue from the confluence with Warm Creek to east of West Street to act as a dyke should also be considered.

6) Additional stream channel improvements such as:
a) Clearing of vegetation and debris from the channel and floodplain in the areas upstream of the Hansen Highway on Blanche Brook and Route 490 on Warm Creek.

b) Dredging of gravel and cobbles from the channel from above Minnesota Drive to St. Georges Bay on Blanche Brook.

c) Channel enlargement (e.g. in the area just downstream of Main Street and Carolina Avenue to St. Georges Bay).

d) Replacement of gabion works on Blanche Brook near Cornwall Heights to curtail erosion of stream banks and and prevent failure of existing baskets.

Figure 5.1 summarizes the locations most feasible for the structural measures as noted above.

## 5.3 Non-Structural Flood Control Measures

In developing areas such as along the Warm Creek and Blanche Brook 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 of hazard and the implementation of flood proofing measures to protect these developments. Other non-structural measures which might be considered include:

1) Maintain a program of debris clearing every fall to reduce the potential for debris blockage at sensitive hydraulic structures during the spring freshet.

2) Review the operational policies of the Noels Pond reservoir, such that the usefulness of this structure to reduce downstream flood peaks is improved.

3) Develop a flood warning system to reduce the potential for flood losses during peak runoff events.

4) Relocation of flood prone structures. This could include expropriation of high hazard lands and restricted development in these areas.

## 5.4 Summary

The most attractive structural alternatives presently appear to be:

1) The replacement of the Route 490 culverts on Warm Creek.

2) A system of berms or dykes along both watercourses to the south of Main Street (Carolina Avenue).

The most attractive non-structural measures include:

1) Implementation of a system of debris clearing to reduce ice/debris jamming potential at the various hydraulic structures along the study reaches.

2) Implementation of regulations to restrict future development in flood prone areas along Blanche Brook and Warm Creek. A two-zone floodway, flood fringe concept is recommended.

These measures would potentially serve to reduce the high flood risk presently occurring in the Town of Stephenville. The benefits of flood damage reduction should be assessed by estimating the potential flood damages associated with the occurrence of various flood events and calculating the associated cost/benefit ratios.

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# Appendix A

# Historical Flooding in the Stephenville Area

SOURCE: REFERENCE (25)

# APPENDIX A
# HISTORICAL FLOODING IN THE STEPHENVILLE AREA

FLOOD OF SEPTEMBER 1 - 2, 1948

Cause: Heavy rains from tropical storm

Description:

All regions of the province felt the force of the tropical storm. Railway schedules were interrupted as a result of minor washouts along the rail line.

At Stephenville (Earnest Harmon Air Force Base), rain driven by maximum winds of 90 m.p.h. (144.8 km/hr) blanketed the area with lakes of water forcing everyone under shelter. Road damage was extensive in the Port au Port Peninsula area.

In St. John's, a gulley near the West End Fire Hall was choked with debris and water overflowed onto LeMarchant Road and flooded the fire hall basement.

Magnitude:

A 3.25 inch (82.6 mm) rainfall was recorded at Corner Brook on September 1st, with 2.41 inches (61.2 mm) falling at Deer Lake and 2.07 inches (52.6 mm) at Gander. At St. John's Torbay Airport, a rainfall of 1.89 inches (48.0 mm) was recorded.

Damages: Damage to local roads in the Port au Port area was estimated at $1,500.

FLOOD OF MAY 19-21, 1969

Cause: Heavy rainfall, snowmelt

Description:

The C.N.R. reported flood damages to their tracks at Harry's River, Black Duck and Codroy Pond in southwestern Newfoundland, disrupting rail service. Highway traffic was also disrupted by washouts. Traffic was re-routed to Stephenville Crossing through Carter's Road near St. George's.

In Stephenville, high flows were experienced on Warm Creek and Blanche Brook. Ice was apparently not a large factor in the flooding which occurred.

On Blanche Brook, the residential area upstream of Main Street was flooded as the banks were overtopped at bends in the river located near Mill Place and Maxwell Avenue. Flood waters flowed through local depressions approximately parallel to the river, flooding several basements and other property.

Most of the flood waters returned to the brook just above the Main Street Bridge, but Main Street was also overtopped near the present Federal Building allowing some of the overbank flow to rejoin Blanche Brook, several hundred feet below the bridge.

A backup of water at the Mississippi Drive bridge on Warm Creek caused erosion of the footing and overtopping of the roadway. Overbank flow occurred west of the bridge between the church and overpass embankment, then under the overpass to the area below the Carolina Avenue Bridge.

Near the confluence of Warm Creek and Blanche Brook, flood waters covered the field near the curling rink and the brewery. Another area flooded was on the airport side of Blanche Brook above the present St. Stephen's Recreation Centre Bridge, but no damage was reported at this location.

Magnitude:

A rainfall of 1.31 inches (33.3 mm) was recorded at Stephenville Airport on the 20th.

Damages:

No estimates of resultant damage were available. However, various municipal agencies assisted in flood relief, and the town provided fill at no cost to residents who sustained erosion damage.

FLOOD OF MAY 15-18, 1972

Cause: Spring runoff accompanied by rainfall

Description:

On the Port au Port Peninsula, flooding cut road connections, isolating several communities.

Near Lourdes, a 55-foot (17 metres) gap occurred in the highway when the water had risen six feet. The road between Point au Mal and Port au Port East was also washed out by the overflow from a culvert incapable of handling the amount of water which had been collecting.

A 75-foot (23 metre) section of highway was washed out between West Bay and West Bay Center and water was reported to be 10 feet (3 metres) deep through the gap. Another section of road between Piccadilly and West Bay was washed out. The washout was said to be in the order of 30 feet (9 metres) wide and fishermen were reported to be transporting their lobster by dory across the washout.

Telephone service to portions of the Port au Port Peninsula was disrupted when the flooding between Stephenville and Stephenville Crossing caused damage to the lines.

In Stephenville, debris carried by the flood waters of Warm Creek blocked culverts on Mississippi Drive causing water to flow over the bridge and pavement at both ends until the road was washed out. Flood waters also threatened the bridge on Carolina Avenue and temporary dykes were constructed in this area to protect the low land around the Harmon Gymnasium and the brewery.

On Missouri Drive, near Massachusetts Drive, water backed up behind a culvert bridge, overflowed onto the pavement and washed out a section of earthfill (about 10' - 15'). The remains of the structure were later removed to clear the channel and the road was never rebuilt.

A youth fell into the flood swollen waters of Warm Creek but was rescued.

FLOOD OF FEBRUARY 2-3, 1973

Cause: Rain, mild temperatures, melting snow and ice jams

Description:

In Corner Brook, minor washouts were reported to have occurred on several gravel roads throughout the city.

In Stephenville, on Blanche Brook, ice piled up across the Main Street Bridge. The wooden sidewalk on the bridge had to be replaced, but the bridge itself was not damaged. The Canada Manpower Office and Building 450 on the Harmon Complex were slightly damaged when ice was forced against them.

All buildings close to the Main Street Bridge had their parking lots blocked with ice which was piled 10 feet high in places. Water rose almost to floor level of the Indian Head Co-op, and also flooded Minnesota Drive opposite the brewery.

A tractor, engaged in removing the ice jam, was reported as having slipped through the ice into the brook.

Magnitude:

It was reported as being the worst ice condition in the history of the town.

Flooding of the brewery was prevented by the temporary dykes which had been built in the spring of 1972 when similar, but less extensive flooding occurred in the area.

Damages: No estimates of the damages were published.

FLOOD OF AUGUST 1-4, 1973

Cause: Heavy rainfall for a 3-day period

Description:

On the Port au Port Peninsula damage was reported to roads. At Fox Island River, a landslide occurred upstream bringing 30-40 cords of pulpwood and trees down the river with the flood which dragged anchors taking several boats out to sea. The highway was also reported to be closed near Piccadilly, Port au Port, because of a washout.

In the Stephenville area, Highway No. 47 (Hansen Highway) was under water in at least two places and a bridge, near Wheeler's Night Club above Noel's Pond, was washed out.

Basements, in the Mill Place area, north of Main Street, were flooded as Blanche (Cold) Brook swelled to several times its normal size.

Many residents were without water for about two days when a water line was broken because of the high water and debris.

Town equipment was kept busy clearing debris (at least 15 truck loads) from the bridges on Blanche Brook, thus averting serious damage. However, concerns were expressed as to their safety, owing to the remaining blockage.

The flood waters eventually broke through the temporary dykes protecting the brewery and reached a height of about 1.5 feet (0.45 metres) above the floor level. Three private vehicles in the parking lot were also inundated. Telephone and electrical services to the building had to be cut off.

At nearby Noel's Pond, at least four families were forced to leave their homes because of the flood waters.

In Corner Brook, a number of basements were flooded as Bell's Brook rushed through and around one of two culverts intended for re-alignment of the road below Blackwood's Hill. At the confluence of Bell's Brook and Corner Brook Stream, water was reported to be washing the Bowater's oil supply line across the brook. Some water also backed up as a result of a partial blockage on Corner Brook Stream by the temporary road near the entrance to Bowater's Pulp Mill. The parking lot behind the Western Star Building on Brook Street was also inundated.

A bailey bridge had to be erected over Blow-me-Down Brook (Blomidon Brook) to replace the one washed out between Frenchman's Cove and Lark's Harbour, Bay of Islands.

Near Pasadena, a water line crossing South Brook was severed by flood waters. A property adjacent to Blue Gulch Brook was eroded 6-8 feet (1.8 - 2.4 metres) when the brook rose to very high levels (photograph published*). At least one home in Pasadena was inundated.

Magnitude:

Rainfall for the three day period totalled 3.61 inches (91.7 mm) at Corner Brook, 2.92 inches (74.2 mm) at Stephenville Airport and 2.75 inches (69.9 mm) at Deer Lake.

The flooding at Stephenville was reported to be the worst in the history of the town.

The C.N.R. files stated that the following houses at Noel's Pond (Cormier Village) were inundated to the following depths:

Residence of Donald Cormier - ±6 in. on main floor

Residence of Eric Cormier - ±4-6 in. on main floor

Residence of Tom Cormier - ±4 feet on main floor

Residence of Mrs. Mildred Hynes - ±6 in. on main floor

Mrs. Vincent Cormier had no damage to her dwelling; however, she had been compensated for loss of hens and income from the sale of eggs.

Damages:

At Stephenville, the broken water main had been damaged during the 1969 flood and had been left dangerously exposed by ice gouging during the spring flood of 1973. After this, the town had spent approximately $21,000 on encasing it in concrete and burying it deep enough to avoid future problems from ice gouging. However, debris was the cause of this break.

Damages to interior and equipment as well as inventories of both goods in process and finished goods at the Brewery was said to be in the order of $70,000.

No estimates were given for the damages inflicted on the residential area north of Main Street.

At Noel's Pond, the C.N.R. was blamed for the damages in that area. A brief was submitted to the C.N.R claiming damages in the order of $2,000.

Damages to the C.N.R. lines were in the order of $46,000.

FLOOD OF NOVEMBER 1-2, 1974

Cause: Heavy rainfall

Description:

In Stephenville, Blanche Brook was swollen to several times its normal size. Work crews were kept busy attempting to clear debris from structures on Carolina Avenue and the Hansen Highway in order to prevent overflow onto the roads.

At Noel's Pond, residents were also affected by flooding but no reports of damages were obtained.

Cold Brook, about five miles from Stephenville, was cut off from the Hansen Highway due to the flooding of the bridge and the washout of the fill at its approaches.

A bridge to the Labrador Linerboard Ltd. camps was reported as having been washed out.

Magnitude:

A total rainfall of 1.68 inches (42.7 mm) was recorded at the Stephenville Airport on the 1st. During a 12-hour period on the 1st, 1.29 inches (32.8 mm) of the total was reported to have fallen.

Damages: No estimates of the damage were published.

FLOOD OF DECEMBER 22-25, 1975

Cause: Rain, snow and unseasonable temperatures

Description:

In Stephenville, flooding of streets was said to be "extensive" as storm sewer systems were not able to handle the amount of water.

In Corner Brook, storm damage was reported to be quite extensive to the streets. Maple Valley Road and Blackwood Hills were reported to be hit the hardest. Road shoulders were reported as being damaged severely. Peddle's Lane, where storm sewers had been installed after the street was washed out by flooding in September, was not seriously affected.

In the Exploits River Valley, a section of the Trans Canada Highway four miles west of Grand Falls was closed because of flooding. The road was flooded to a depth of about two feet (about 0.6 metres) at 8 a.m. on December 24th. On January 8th, almost two weeks after the peak flow, part of the highway was still under water. Traffic was diverted by way of the Old Badger Road.

Magnitude:

A total of 2.05 inches (52.1 mm) of precipitation, including snow and rain was reported at Stephenville.

On December 22nd, a rainfall of 0.98 inches (24.9 mm) was recorded at Corner Brook. A rainfall of 1.79 inches (45.5 mm) was recorded on the 22nd at the Exploits Dam (Red Indian Lake).

On the Exploits River ice began to move downstream in stages as a result of the rainfall and high temperatures until it reached the area of Goodyear's gravel pit. At this point in the river, the ice was reported to have accumulated and partially blocked the channel. Increased water levels resulted upstream and subsequently began to flow through the gravel pit adjacent to the north bank until reaching the Trans Canada.

The gauging station on the Exploits River below Stony Brook (Station No. 02YO005) indicated that the flow began to increase about 7 a.m. on December 23rd and reached a value of 13,000 cfs (368 $m^3/s$) by midnight and about 22,800 cfs (645 $m^3/s$) at 4 p.m. on December 24th. The flow receded to about 6,000 cfs (170 $m^3/s$) by midnight on December 26th.

The Exploits River drainage area, above Red Indian Lake, was reported as not having contributed to the flooding as Price Newfoundland Ltd. had closed the Exploits Dam in November 1975; to increase the storage which had been depleted because of insufficient runoff. As well, the company did not require any water for the power plant because of an on-going strike at the mill.

Damages:

The highway remained closed throughout most of the winter. No estimates of damages to the highway were given. This section of highway was reported as having been vulnerable to flooding due to the low lying land areas adjacent to the river and minor flooding had been noted by highway personnel "especially in the spring of the year".

It was also noted that approximately two miles of highway would be raised several feet to prevent the recurrence of flooding in the future.

FLOOD OF DECEMBER 27-29, 1977

Cause:

Combination of rain and high temperatures on Christmas Day and Boxing Day melted the accumulation of snow

Description:

Flooding was reported across insular Newfoundland, resulting in disruption of transportation and inconvenience to many home owners.

Trains were halted because of washouts along the C.N.R. rail lines across the Province. C.N.R. officials said some 40 washouts occurred and reported that the main problems were between Clarenville and Port aux Basques. Some of the damaged sections were reported to be up to 200 feet (60 metres) in length. C.N.R. also reported flooding on the line between Clarenville and Bonavista. It was reported that about two days would be required to affect repairs to the lines and about 130 laid-off seasonal employees were called back for this operation. A derailment of a work train also occurred near Fishell's Cove when the embankment collapsed after being weakened by the high water.

Flooding was also reported in the Noel's Pond area as a result of debris blocking the bridges or culverts.

In Corner Brook, several roads were damaged and some houses were inundated with water and mud. Extensive damages were reported in the Massey Drive, Benoit's Cove and several other areas of the City where the drainage was not adequate. Some of the streets which sustained damage were the Old Humber Road, Riverside Drive, Barrett's Road and Maple View. Curbing and road shoulders were damaged on East Valley Road West Valley Road, Windsor Street, Meadows and Gillams Roads. In the Riverside Drive area, tons of mud and water inundated homes on the north side of Ballam Bridge. Much of the mud flowed onto the property of Owen Legge and filled the basement apartment. Several streets in the Curling NIP (Neighbourhood Improvement Program) area were damaged. Water also overtopped the dam at the Margaret Bowater Park.

A landslide occurred at Mount Moriah which blocked the highway.

Guard rails and hydro poles along the Humber River were reported as having little support as the flooding had washed away tons of earth.

The road to Cox's Cove was closed when portions of the roadbed along a quarter mile section was reported as having slid downhill when a culvert was washed out.

East of Corner Brook, the road to Trout River was reported to be washed out, as well as, the Trans Canada Highway at Little Rapids. The TCH was reported to be inundated near Birchy Lake but was said to be "passable".

At King's Point, Green Bay, floodwaters tore up a 100-foot section of the main street rupturing a water line and isolating about 40 families. The pumphouse was also inundated leaving 700 people without water. Debris was reported to have punctured the dam as well.

At Buchans, several miles of old workings and tunnels at the ASARCO* mines were flooded to a depth of about eight feet as a result of the mild spell and rain. The flooding was reported as not having affected production since the flooded tunnels are used mainly for access and ventilation.

In Grand Falls, Main Street was closed for some time because it was submerged. Work crews were kept busy clearing debris from storm sewers that caused water to back up and flood low lying sections of streets and several basements. Some tenants were forced to move out of their flooded basement apartments on Goodyear Avenue.

In Gander, many basement apartments were flooded and some residents had to leave their living quarters.

Six miles (9.7 km) east of Gander, Soulis Brook overflowed a 200-foot (60 metre) section of the TCH washing out the shoulders and undermining some pavement closing it to traffic for a period until waters receded.

The TCH was also reported to have been flooded in the Port Blandford area by Southwest Brook causing problems for vehicular traffic. The water was reported to be 10 inches (0.25 metres) deep.

On the Bonavista Peninsula, numerous roads were said to have been inundated and/or washed out and were impassable to all but large trucks and four wheel drive vehicles. Two of these areas were at Lethbridge and Lockers Brook (weakened bridge).

Other gravel roads in the Province were washed out and in some cases communities were isolated. Some of these roads were in the Buchans area and the Harbour Breton Road between Pool's Cove and St. Jacques.

In St. John's, basements were flooded and roads were inundated and some washed out.

The Waterford River overflowed its banks inundating sections of Bowring Park and the Kinsman Park on Squires Avenue. On the southside of Squires Avenue the flood waters were reported to have eroded several feet of land to the rear of the homes. Flooding was also reported around Quidi Vidi Lake and in the Higgins Line area. Houses in the newly developed Spratt Place, east of Canada Drive, experienced severe basement flooding.

Problems were also experienced in the Highland Park Subdivision and on Southside Road as a result of clogged storm drains. The basement of the C.N.R. customer station and express building as well as the diesel building, tracks and parking lot on Water Street West were inundated. The problem was suspected to be related to the trunk sewer.

Magnitude:

At Corner Brook, 1.25 inches (31.8 mm) of rain and temperatures in the order of 11°C to 13°C (52°F to 55°F) were recorded. At Buchans, 1.27 inches (32.3 mm) of rain was recorded on the 27th. A rainfall of 1.80 inches (45.7 mm) was recorded at St. John's with temperatures in the order of 10°C (50°F).

Brooks and ponds were reported to be far above normal across the Province.

Residents of Riverside Drive, Corner Brook, said "problems had never occurred in that area until about three years ago when changes were made to the gravel pit on the hill behind their homes. Prior to this, the water from the pit normally flowed through a brook into the Humber River.

Damages:

In Corner Brook, damages to city streets were estimated at $100,000.

Newspaper accounts stated that highway damages could reach several hundred thousand dollars. No other estimates of damages were presented.

FLOOD OF SEPTEMBER 30 - OCTOBER 2, 1978

Cause: Heavy rainfall

Description:

A photo** was published showing a 60-foot (about 18 metres) section of C.N.R. rail bed which had been eroded by the heavy rainfall. The damaged section of road bed was on the 5.5 mile (8.8 km) spur linking the Labrador Linerboard Mill to the line at Noel's Pond near Stephenville.

Magnitude:

The mean daily discharge for Blanche Brook near Stephenville, (Station No. 02YJ002), for October 1st was 57.0 cfs (1.6 $m^3/s$).

Damages:

No information on repair plans or cost of repairs was available. The linerboard mill had been closed for a year but it was reported that a substantial sum was paid to the C.N.R. for maintenance of the spur.

* The Western Star, May 19, 1978, pg. 1

** The Western Star, October 3, 1978, pg. 3

FLOOD OF MARCH 6-9, 1979

Cause: Rain, melting snow, blockage of drainage systems by ice and debris

Description:

A four-day period of spring-like weather caused flooding and some washouts in central and western Newfoundland.

Both the Robinson's and Barachois Rivers overflowed their banks when ice collected at the bridges resulting in flooded roadways.

In Stephenville, the high flows and ice in Blanche Brook undercut footings of four 20-foot (6 metres) sections of a retaining wall causing them to collapse. Town work crews were kept busy in an attempt to fill the breach with boulders and concrete to reduce the possibility of the Labatt Brewery complex being flooded.

In the Corner Brook area, some flooding of basements and minor road damages were reported throughout the city. Some flooding was reported at the intersection of Dave's Road and Primrose Avenue and on the highway between Brake's Cove and Riverside Drive. Corner Brook Stream rose to less than two feet from the bottom of the bridge on Main Street and backed up into the sewer system through an outfall.

In the Roddickton area, on the Northern Peninsula, blockage of culverts and ditches in several places caused overflow onto the roads.

The Baie Verte Peninsula also experienced some flooding of a minor nature and a few washouts.

In Central Newfoundland, flooded streets and basements were reported in most of the towns. A section of the Bay d'Espoir highway was inundated about 25 miles (40 km) south of Bishop's Falls at a point where Rattling Brook is closest to the highway. This resulted in a short closure of the highway.

Magnitude:

Total precipitation in the order of 64 millimetres (about 2.5 inches) was reported at Corner Brook during the period. Total precipitation for March at Corner Brook was 186.3 millimetres (7.33 inches) of which 142.1 millimetres (5.59 inches) was in the form of rain. The month was described as the wettest since 1938.

The flooded section of the Bay d'Espoir highway was reported as being flooded for the first time in the road's 12 year history.

Damages:

At Stephenville, the Town Manager was reported to have said that "the complete and permanent repair to the damaged cribbing is expected to be between $100,000 and $120,000". It was reported that Alexander Engineering was contracted by the Town to carry out permanent repairs at the site. The permanent structure was reported to be an interlocking steel sheet pile facewall.

No other estimates of the damages were presented. Some photographs* were published for Stephenville and the Corner Brook area.

FLOOD OF JULY 1 -20, 1979

Cause: Heavy rainfall

Description:

In the Stephenville area, Blanche Brook, swollen by heavy rains, carried trees and other debris downstream which collected and jammed at the Hansen Highway Bridge. The section of Hansen Highway from Brook Street to the intersection of Queen Street was closed on the morning of the 18th. Transportation Department employees were kept busy attempting to keep debris from floating downstream causing further damage. The center pier of the bridge had been moved by the force of the debris and water. The structure collapsed around mid-day on the 19th.

Debris also was reported to have piled up where the brook runs between Carolina Avenue and Main Street but no damage resulted. Many Stephenville residents had flooded basements and the Harmon Convenience Store on Carolina Avenue foundered when its basement gave way.

In Corner Brook, a few minor washouts had occurred but no serious flooding or damages were reported within the city limits.

Magnitude:

The technical services department of Bowater Newfoundland Ltd. recorded 1.66 inches (42.2 mm) of rainfall from 1:30 p.m. on the 17th until 8:00 a.m. on the 18th.

Damages:

No estimate of the damage was reported for the Harmon Convenience Store; however, it was reported that they were open for business on the 19th. Hines Esso on the Hansen Highway near the collapsed bridge reported extensive losses in business and as a result was required to lay-off employees. No estimates for a replacement bridge were presented. The original bridge across Blanche Brook was built by the United States military in the mid-1950's. The newspaper accounts reported that it took about three weeks to get a Bailey Bridge in place. Some photos* of the structure were published (before and after the collapse).

FLOOD OF JULY 28-29, 1979

Cause: Heavy rain and debris jams

Description:

Blanche Brook, in Stephenville, was reported to be in flood once again. The brook was said to be three or four times its normal size and debris was beginning to pile up at the site of the bridge which had collapsed during the flooding of July 19th.

Magnitude:

No information was presented. A photograph** was published showing the brook at the site of the collapsed bridge.

Damages: No description or estimates of the resultant damages were presented.

FLOOD OF AUGUST 17-18, 1979

Cause: Heavy rainfall

* The Western Star, July 18, 1979, pg. 1; The Western Star, July 19, 1979, pg. 3; and, The Western Star, July 20, 1979, pg. 1.

** The Western Star, July 30, 1979, pg. 2.

FLOOD OF APRIL 22-30, 1982

Cause: Heavy rain and melting snow

Description:

The April 22, 1982 Western Star reported Blanche Brook had begun to swell. By April 30th heavy rain and melting snow had caused basement flooding in the Blanche Brook area. Blanche Brook was reported to be swollen but not flooding.

However, the May 1, 1982 Western Star reported "extensive" basement flooding in Noels Pond, forcing some residents to leave their homes. Residents stated that the new Stephenville industrial access highway completed to gravel standard in 1981 has caused flooding in an area of the community not previously flooded. Noels Pond experiences annual flooding when Noels Pond and Warm Creek overflow with spring runoff.

Particular problems were experienced by residents of the Sunset Crescent area, Queen Street, near Cornwall Heights and along Blanche Brook as well as the Park Avenue - White's Trailer Court area.

Magnitude:

No information was presented.

Damages:

No estimate of the resultant damages were presented.

FLOOD OF JANUARY 11-13, 1983

Cause: Heavy rain and melting snow

Description:

Blanche Brook and Warm Creek swelled to rise by as much as 1.3 m in some areas. Localized areas of flooding were reported in the Town of Stephenville near Park avenue and White's Trailer Park, with mainly access routes being affected. Extensive basement flooding was reported in Noels Pond with the industrial access route to the Town of Stephenville being blamed as the major causative factor.

Magnitude:

A total precipitation of 59.4 m was recorded at the Stephenville meteorologic station, combined with a melt of 11 cm of snow cover over the 3-day period.

Damages:

No estimate of compensative damages or claims were reported.

FLOOD OF JUNE 1-3, 1983

Cause: Heavy rainfall

Description:

Flooding was reported in some areas of Noels Pond along Warm Creek while no significant flooding was reported downstream in the Town of Stephenville.

Magnitude:

The Stephenville meteorologic station recorded a total rainfall amount of 86.2 mm.

Damages:

No information was given on resultant flood damages.

# Appendix B

# Background for Statistical Hydrological Analyses

# Appendix B Background for Statistical Hydrological Analyses

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# Appendix B Background for Statistical Hydrological Analyses

The following sections provide additional information describing the methodology applied in undertaking peak flow estimates by various statistical techniques:

## B.1 Statistical Analysis of Harrys River

Initially, it was assumed that a single station flood frequency analysis using the Harrys River hydrometric data base could be transferred to Blanche Brook. The following sections describe the analyses which were undertaken to test this assumption.

### B.1.1 Testing the Data Base

The sample data set of maximum instantaneous flows was tested for stationarity and randomness, using graphical techniques. A summary of the sample extreme-value data set available for the Harrys River is given in Table B.1. The sample data set was tested for stationarity and randomness using the following technique:

a) A progressive mean of annual flood peak ($m^3/s$), (Figure B.1), i.e.

$$QP_n = \frac{1}{n} \sum_{i=1}^{n} Q_{pi} \qquad (B.1)$$

where : $QP_n$ = cumulative moving mean max. instantaneous peak flow at year n

$i$ = year, 1 to n

$Q_p$ = annual max. instantaneous flood peak ($m^3/s$)

b) Temporal distribution of QP (Figure B.2)

where : $Q_p$ = annual max. instantaneous flood peak ($m^3/s$)

The results of this analysis reveal that the data can be regarded as stationary (i.e. no trend or persistence is evident from the graphical analyses). There is also no evidence of non-stationarity in the data set. as evident in the data plots presented in Figures B.1 and B.2.

### B.1.2 Flood Frequency Analysis

Several distributions were fitted to the available Harrys River data set for comparison purposes, including Gumbel, Log Normal (LN), Three Parameter Log Normal (3PLN) and Log Pearson Type III (LP3). The computer program, FDRPFFA (21), provided by the Water Planning and Management Branch, Environment Canada, was utilized for this purpose and for computing the 1:20 and 1:100 year flood peaks and the corresponding 95% confidence limits. Table B.2 presents a summary of the different distributions and the corresponding standard errors.

In flood frequency analysis, it is not possible to pre-determine which frequency distribution should be used. The selection of the appropriate distribution for use in this investigation was based on comparisons of the computed "T" year flood using the methods as outlined in the FDRPFFA program (21), and summarized as follows:

- comparison of various fitting techniques to the regional peak flow estimates using available regional techniques
- comparison of the theoretical statistics of skew and kurtosis to those estimated from the station data
- visual comparison of the degree of fit of each distribution to the available data set
- estimation of upper limit of flow estimates.

For Newfoundland, Poulin (38) recommends the Gumbel distribution be used. This distribution has fixed coefficients of skew and kurtosis of 1.14 and 5.4 respectively. The Harrys River data set was found to have coefficients of skew and kurtosis of 1.28 and 6.4 respectively which tend to agree with the use of the Gumbel distribution. However, it can be seen from Table B.2 that the flood estimates using the Gumbel distribution appear low when compared to the other flood frequency distributions. To be conservative, and to allow the transfer of a coefficient of skew which has been fit to the data series, the Log Pearson Type 3 was selected for use in this study.

The selection of the LP3 distribution enabled the skew of the logarithms to be transferred and the application of frequency factors to calculate flood estimates on Blanche Brook, thus allowing use of a flow transfer procedure recommended by the U.S. Corps of Engineers (47). Based on the LP3 distribution, the estimated 1:20 and 1:100 year instantaneous peak flows on Harrys River at the gauge location were found to be 608 $m^3/s$ and 806 $m^3/s$ respectively.

It can be seen from the frequency plot (Figure B.3) of the data points that the annual peak of 1973 is substantially higher than all the other data points and may possibly be a high outlier. With this data point removed from the flow series, the estimated 1:20 and 1:100 year instantaneous peak flows on Harrys River at the gauge location are reduced to 481 $m^3/s$ and 554 $m^3/s$ respectively.

## B.2 Statistical Transfer ~ Harrys River to Blanche Brook

When streamflow records at a gauge site of interest are limited to a few years of data, a reliable flood frequency analysis is not possible. It is sometimes possible to utilize other stations in the area which have a larger record of data.

The Harrys River gauge was selected for use in this analysis as it is in close proximity to the Blanche Brook watershed, and because it is the only nearby watercourse with sufficient hydrometric data available. The meteorological conditions of the two watersheds are also considered to be similar. However, the physiographic and hydrologic characteristics of the Blanche Brook and Harrys River watersheds appear to be somewhat different. For example, the mean annual runoff for Harrys River is approximately 20 percent greater than that for Blanche Brook. This should result in greater unit peak discharges for Harrys River and hence an over-estimation of the transferred design flow rates might be expected. Table B.3 presents a summary comparison of several hydrologic and meteorologic characteristics determined for each watershed.

Once the sample statistics are derived for the station of shorter record, the following equation can be utilized to compute desired flood estimates;

$$Y_T = \mu_y + k\,\sigma y \qquad \text{(B-2)}$$

where $Y_T$ = logarithmic value of the flood estimate

$\mu_y$ = adjusted value of the mean of the logarithmic array for the station with the short record length

$\sigma_y$ = adjusted value of the standard deviation of the logarithmic array for the station with the short record length

$k$ = frequency factor corresponding to a desired return period flood estimate

## Transfer of Mean

A report by the U.S. Army Corps of Engineers (47) presents the following equations for adjusting the value of the mean of the logarithms of the array of annual peak discharges for a short-term station:

$$M'_1 = M_1 + (M'_2 - M_2)\, r\, \frac{SD_1}{SD_2} \qquad \text{(B-3)}$$

where $M'_1$ is the adjusted value of the mean of the logarithmic array for the short-term station (4.328)* (4.251)**

$M_1$ is the mean of the logarithmic array for the short-term station for the years of concurrent record at the short- and long-term stations (4.1669)*/**

$M'_2$ is the mean of the logarithmic array for the long-term station for the entire period of record at the long-term station (5.7876)* (5.7254)**

$M_2$ is the mean of the logarithmic array for the long-term station for the years of concurrent record at the short- and long-term stations (5.6574)*/**

$r$ is the coefficient of correlation for logarithmic peak discharges for the short- and long-term stations (.995)*/**

$SD_1$ is the standard deviation of the logarithmic array for the short-term station for the years of concurrent record at the short- and long-term stations (.4058)*/**

$SD_2$ is the standard deviation for the logarithmic array for the long-term station for the years of concurrent record at the short- and long-term stations (.3261)*/**

* used in statistical transfer including 1973 Harrys River event

** used in statistical transfer with 1973 Harrys River event removed

Transfer of Standard Deviation

The report by the U.S. Army Corps of Engineers (1962) presents the following equation which is an approximate but simple equation for adjusting the value of the standard deviation of the logarithms of the array of annual peak discharges for a short-term station:

$$SD'_1 - SD_1 = (SD'_2 - SD_2)\ r\ \frac{SD_1}{SD_2} \tag{B.4}$$

where $SD'_1$ is the adjusted value of the standard deviation of the logarithmic array for the short-term station (.4558)* (.3773)**

$SD_1$ is the standard deviation of the logarithmic array for the short-term station for the years of concurrent record at the short- and long-term stations (.4058)*/**

$SD'_2$ is the standard deviation of the logarithmic array for the long-term station for the entire period of record at the long-term station (.3667)* (.3030)**

$SD_2$ is the standard deviation of the logarithmic array for the long-term station for the years of concurrent record at the short- and long-term stations (.3261)*

r is the coefficient of correlation for concurrent logarithmic peak discharges for the short- and long-term stations (.995)*/**

* used in the statistical transfer including 1973 Harrys River event

** used in the statistical transfer with 1973 Harrys River event removed.

The coefficient of skew of the logarithmic array for the long-term station (Harrys River) was used directly to calculate floods on Blanche brook for various return periods using frequency factors as previously discussed in this section. The resulting flood peaks are presented in Table B.4.

As previously discussed, there is some question as to whether the Harrys River data set should be modified by removing the very high 1973 flood (688 $m^3/s$). For comparison purposes, the statistical transfer procedure was repeated assuming the 1973 flood event was a high outlier. The resulting flood peaks are also presented in Table B.4 and indicate that peak flows at the gauge are lower than those predicted using the Regional or deterministic techniques (refer to Table 3.7 in the main text). This would tend to indicate that the Harrys River 1973 flood event is not a high outlier and as such should be included in any transfer analysis. However, due to the high degree of variability in peak flow estimates as a result of removing the Harrys

River 1973 flood event, it is recommended that this procedure not be used for estimating design flows. Future application of this technique may be more practical if the longer (future) record length reduces the level of sensitivity of single station peak flow estimates for the 1973 event.

## B.3 Regional Flood Flow Estimates

### B.3.1 Canada-Newfoundland Peak Flow Estimates

The sub-committee on "Flood Frequency Analysis" consisting of Federal and Provincial members has derived Regional Flood Flow regression equations for the Island of Newfoundland. The regression equations for Southern Newfoundland are based on using available records at hydrometric stations with at least 10 years of record and take the following form:

$$\log_{10} QP_{20} = -3.7581141 + 0.84990968 \log_{10} DA + 2.360446 \log_{10} MAR - 1.5247540 \log_{10} ACLS - 1.6244318 \log_{10} SHAPE \qquad \text{(B-5)}$$

$$\log_{10} QP_{100} = -3.5915295 + 0.81413934 \log_{10} DA + 2.4090173 \log_{10} MAR - 1.6223283 \log_{10} ACLS - 1.4277436 \log_{10} SHAPE \qquad \text{(B-6)}$$

where $QP_T$ = annual instantaneous peak flow with a recurrence interval of T Years

ACLS = area controlled by lake and swamp (% of drainage area) (60.1%)* as determined from 1:50,000 NTS maps using criteria that lake or swamp with surface areas at least 1% of the drainage area to the lake or swamp outlet controls the area to the outlet

SHAPE = (0.28 x basin perimeter) / DA (1/km) from Chow's Handbook on Hydrology (1.68 $km^{-1}$)*

. MAR = Mean Annual Runoff (mm) over the area (1100 mm)*

DA = Drainage area (117.1 $km^2$)*

* Blanche Brook parameters for gauge location.

Table B.5 presents a summary of flood estimates calculated using this method.

It has been found that the flood flow estimates are fairly sensitive to parameter errors. For example, if a 10% error is assumed for each of the parameters individually, the following changes occur in $QP_{100}$ for Blanche Brook at the hydrometric station:

| ±Change in Parameter | Produces Following Change |
|---|---|
| DA | +8%, -8% |
| MAR | +26%, -22% |
| ACLS | -14%, +19% |
| SHAPE | -13%, +16% |

It is evident from the sensitivity analysis that the peak flow estimates by this technique are most sensitive to variations in the MAR and ACLS parameters. It can generally be assumed that ACLS can be estimated with fairly good accuracy (i.e. better than 10%) from topographic information. However, estimates of MAR (index of mean annual runoff) were provided by Environment Canada according to mapped values in order to improve estimates due to the short record length at the gauge location.

## Secondary Comparison

A secondary check evaluated the accuracy of the regression equation in estimating the peak flows on Harry's River. The regression equation provides an estimate of the 100 year instantaneous flood flow of 633 $m^3/s$ which is located between the value of 825 $m^3/s$ (obtained from the single station analysis with the very high 1973 flood left in), and 552 $m^3/s$ obtained by removing the 1973 flood flow of 688 $m^3/s$. (Single station analyses based on 13 years of record for the Harrys River.)

The 1:100 year flood flow estimates are based on the 3PLN distribution. If the LP3 distribution is selected, 1:100 flood estimates are

554 $m^3/s$ and 806 $m^3/s$ respectively. This secondary comparison tends to confirm the accuracy of the regional flood flow estimating technique for watersheds in the Stephenville area.

### B.3.2 Regional Peak Flow Estimates by Poulin's Method

A secondary check on the new regional flood flow estimates was obtained by utilizing equations derived by Poulin (38). These equations were derived for Newfoundland using data up to 1969 and are based on a Gumbel frequency distribution. This technique is similar to the recent regional analysis but is somewhat out of date due to the shorter period of record used in the analysis.

The equation used to compute the return frequency flood estimate first requires the estimation of the mean annual flood (MAF) which constitutes a 1:2.33 year flood based on a Gumbel distribution. The mean annual flood is then multiplied by index factors to obtain the desired flood frequency estimate. The MAF is computed as follows:

$$MAF = C\,(DA/2.59)^a\,(MAR/25.4)^b\,(ACLS)^c\,(SLOPE\times 10E4)^d \qquad (B.7)$$

where DA = drainage area (117.1 $km^2$)*

MAR = mean annual runoff (1100 mm.)*

ACLS = percentage of the drainage area controlled by lakes or swamps (60.1%)*

SLOPE = channel slope in % (Note: 10E4 = $10^4$) (1.27%)*

a = 0.99252

b = 1.07620

c = -1.08832

d = 0.24023

C = 5.563

* for Blanche Brook

Based on the physiographic parameters derived for Blanche Brook, the mean annual daily flood was calculated to be 49.4 $m^3/s$ at the gauge. Index factors of 2.72 and 3.49 were then determined to calculate the 1:20 and 1:100 year floods which are presented in Table B.5. Since this regional technique was derived for calculating mean daily flow rates, it was necessary to make an adjustment to instantaneous flows for comparison purposes. Based on a review of historic flow records, the ratio of maximum instantaneous flood flows to maximum mean daily flows was found to be approximately 1.60 and was used in adjusting the mean daily values.

It is evident by comparisons with other flood estimates that the Regional flood estimates by Poulin's method are somewhat different, although tending to confirm the CNFDRP estimates. The differences are attributed to the fact that the equations were derived based on a very limited data base and have been applied outside the range of their applicability, and due to the different frequency distributions underlying the two analyses.

It is believed that the recently developed regression equations result in more appropriate peak flow estimates due to the additional data base underlying this analysis.

TABLE B.1
Summary of Harrys River and Blanche Brook
Annual Peak Flow Data Set

| Year | Harrys River ($m^3/s$) | Blanche Brook ($m^3/s$) |
|---|---|---|
| 1969 | 436 | |
| 1970 | 281 | |
| 1971 | 183 | |
| 1972 | - | |
| 1973 | 688 | |
| 1974 | 326 | |
| 1975 | 430 | |
| 1976 | 379 | |
| 1977 | 300 | |
| 1978 | 340 | 36.2 |
| 1979 | 244 | 112.0 |
| 1980 | 442 | 71.7 |
| 1981 | 187 | 58.6 |
| 1982 | 281 | 65.6 |

# TABLE B.2

## Estimates and Standard Error for Various Distributions Tested

### Discharge Data

STATION NO. 02YJ001 - HARRYS RIVER

| YEAR | DATA | ORDERED | RANK | PROB. | RET. PERIOD |
|---|---|---|---|---|---|
| 1969 | 436. | 688. | 1 | 0.071 | 14.000 |
| 1970 | 281. | 442. | 2 | 0.143 | 7.000 |
| 1971 | 183. | 436. | 3 | 0.214 | 4.667 |
| 1973 | 688. | 430. | 4 | 0.286 | 3.500 |
| 1974 | 326. | 379. | 5 | 0.357 | 2.800 |
| 1975 | 430. | 340. | 6 | 0.429 | 2.333 |
| 1976 | 379. | 326. | 7 | 0.500 | 2.000 |
| 1977 | 300. | 300. | 8 | 0.571 | 1.750 |
| 1978 | 340. | 281. | 9 | 0.643 | 1.556 |
| 1979 | 244. | 281. | 10 | 0.714 | 1.400 |
| 1980 | 442. | 244. | 11 | 0.786 | 1.273 |
| 1981 | 187. | 187. | 12 | 0.857 | 1.167 |
| 1982 | 281. | 183. | 13 | 0.929 | 1.077 |

### Statistics

STATION NO. 02YJ001

SAMPLE STATISTICS

MEAN = 347. S.D. = 134.1 C.S. = 1.2768 C.K. = 6.4851

SAMPLE STATISTICS (LOGS)

MEAN = 5.7876 S.D. = 0.3667 C.S. = 0.1905 C.K. = 4.1774

SAMPLE MIN = 183. SAMPLE MAX = 688. N = 13

PARAMETERS FOR GUMBEL I A = 0.010321 U = 290.

PARAMETERS FOR LOGNORMAL M = 5.7876 S = 0.3667

PARAMETERS FOR THREE PARAMETER LOGNORMAL A = 69. M = 5.5262 S = 0.4737

STATISTICS OF LOG(X-A)

MEAN = 5.5262 S.D. = 0.4737 C.S. = -0.0814 C.K. = 4.0202

PARAMETERS FOR LOG PEARSON III BY MOMENTS A = 0.0349 B =0.1103E+03 LOG(M) = 1.9373 M = 0.6940E+01

PARAMETERS FOR LOG PEARSON III BY MAXIMUM LIKELIHOOD A = 0.0411 B =0.7357E+02 LOG(M) = 2.7616 M = 0.1583E+01

DISTRIBUTION STATISTICS MEAN = 5.7876 S.D. = 0.3528 C.S. = 0.2332

### Flood Frequency Distributions

| | GUMBEL I | | LOGNORMAL | | THREE PARAMETER LOGNORMAL | | LOG PEARSON III MAX. LIKELIHOOD | | LOG PEARSON III MOMENTS | |
|---|---|---|---|---|---|---|---|---|---|---|
| RETURN PERIOD | FLOOD ESTIMATE | ST. ERROR PERCENT | FLOOD ESTIMATE | ST.ERROR PERCENT | FLOOD ESTIMATE | ST. ERROR PERCENT | FLOOD ESTIMATE | ST. ERROR PERCENT | FLOOD ESTIMATE | ST. ERROR PERCENT |
| 1.005 | 128. | | 127. | | 143. | | 142. | | 135. | |
| 1.050 | 182. | | 178. | | 185. | | 187. | | 182. | |
| 1.250 | 244. | | 240. | | 238. | | 242. | | 239. | |
| 2.000 | 326. | | 326. | | 321. | | 322. | | 322. | |
| 5.000 | 435. | 11.10 | 444. | 11.80 | 444. | 12.90 | 437. | 12.10 | 442. | 12.50 |
| 10.000 | 508. | 12.20 | 522. | 13.70 | 530. | 15.90 | 517. | 14.80 | 526. | 15.20 |
| 20.000 | 578. | 13.20 | 596. | 15.60 | 617. | 19.50 | 596. | 18.60 | 608. | 19.00 |
| 50.000 | 668. | 14.10 | 693. | 17.90 | 734. | 24.60 | 703. | 24.50 | 719. | 25.30 |
| 100.000 | 736. | 14.80 | 765. | 19.60 | 825. | 28.50 | 787. | 29.50 | 806. | 30.50 |
| 200.000 | 803. | 15.30 | 839. | 21.10 | 920. | 32.40 | 875. | 34.80 | 896. | 36.00 |
| 500.000 | 892. | 15.90 | 938. | 23.10 | 1050. | 37.50 | 997. | 42.00 | 1020. | 43.70 |

TABLE B.3

Comparison of Meteorologic and Physiographic Characteristics
for Blanche Brook and Harrys River Watersheds

| | Units | Harrys River | Blanche Brook | Source |
|---|---|---|---|---|
| Drainage Area - to gauge | $km^2$ | 640 | 117 | Harrys R. - 1:50,000 |
| - to outlet | $km^2$ | 834 | 118.6 | Blanche B.- 1:12,500 |
| % Forest | % | 70 | 84 | Ref. (40) |
| Average Yearly Precipitation | mm | 1679 | 1166 | Ref. (18) |
| Average Yearly Temperature | °C | 3.5 | 4.8 | Ref. (18) |
| Area controlled by swamps | % | 75 | 60 | Ref. (13) |
| Mean Annual Runoff | mm | 1320 | 1100 | Ref. (13) |

TABLE B.4
Design Flow Estimates Using
Statistical Transfer Technique*

Peak Flow Estimates Blanche Brook at Gauge Location

| Return Period | $Q_p$** ($m^3/s$) | $Q_p$*** ($m^3/s$) |
|---|---|---|
| 1:20 | 165.3 | 123.6 |
| 1:100 | 235.7 | 147.5 |

* at gauge location

** including 1973 Harrys River Flood

*** with 1973 event removed from Harrys River Data set

TABLE B.5

Design Flow Estimates

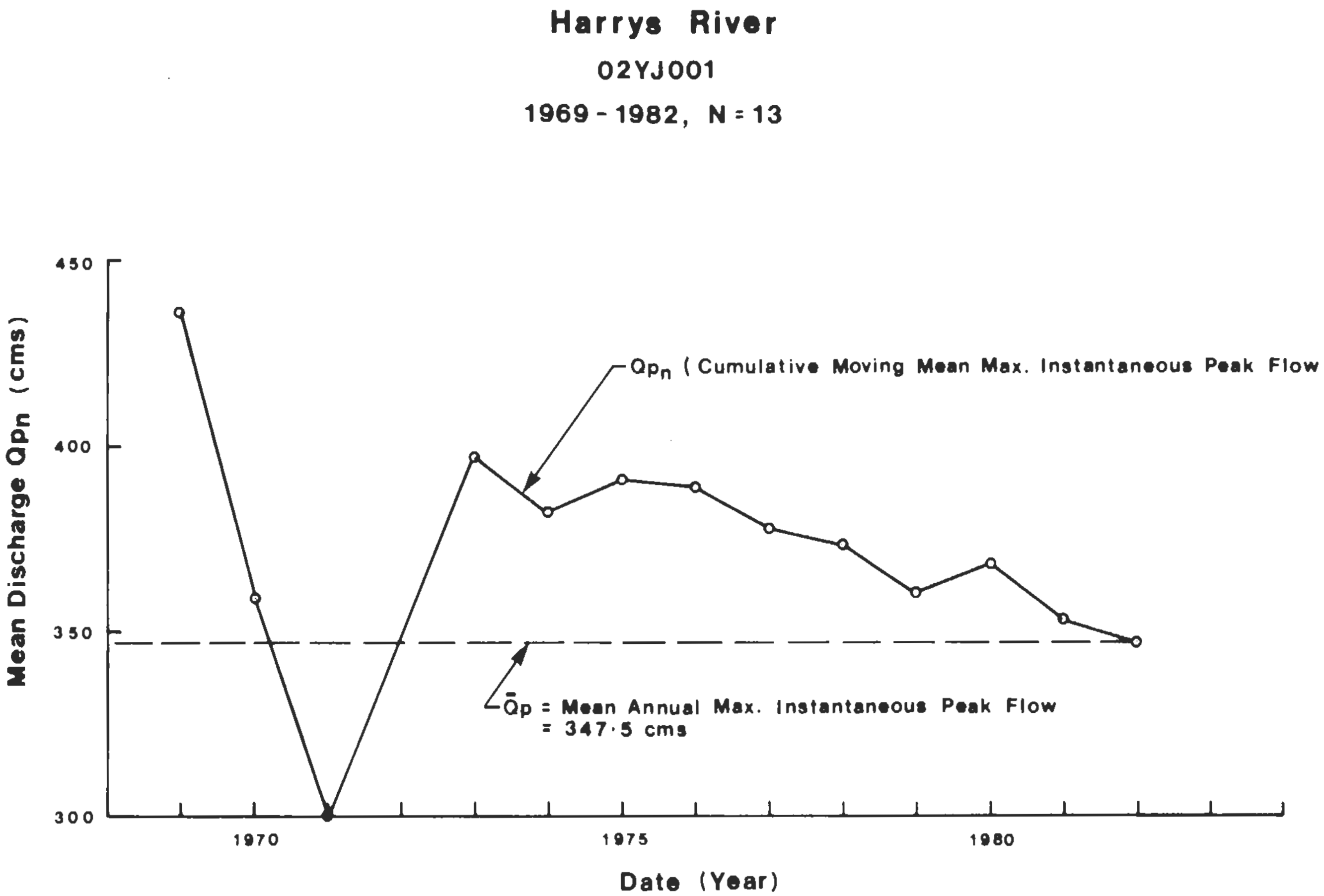
Using Regional Equations*

| Return Period (years) | $Q_p$** ($m^3/s$) | $Q_p$*** ($m^3/s$) |
|---|---|---|
| 1:20 | 134.3 | 126.1 |
| 1:100 | 172.2 | 162.8 |

* At gauge location

** Poulin's Method (Includes peaking factor of 1.60 to obtain Maximum Instantaneous Peak Discharge) (38)

*** Canada-Newfoundland FDRP Regional Flood Frequency Analysis (13)



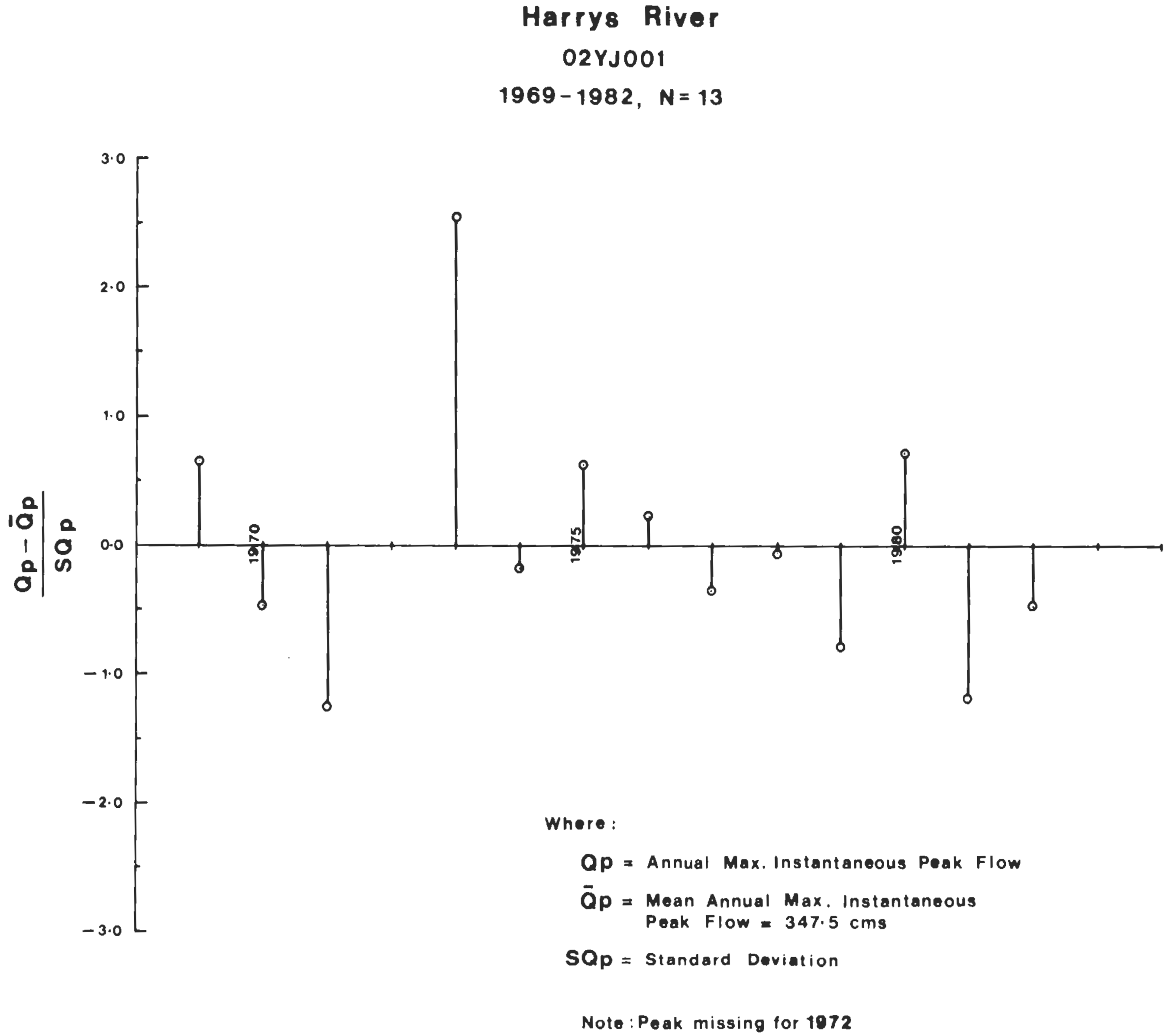

Canada - Newfoundland
Flood Damage Reduction Program

**Harrys River**

**Cumulative Moving Mean for Trend**

Nolan Davis & Associates Ltd  Cumming-Cockburn & Associates Ltd

**Figure B.1**

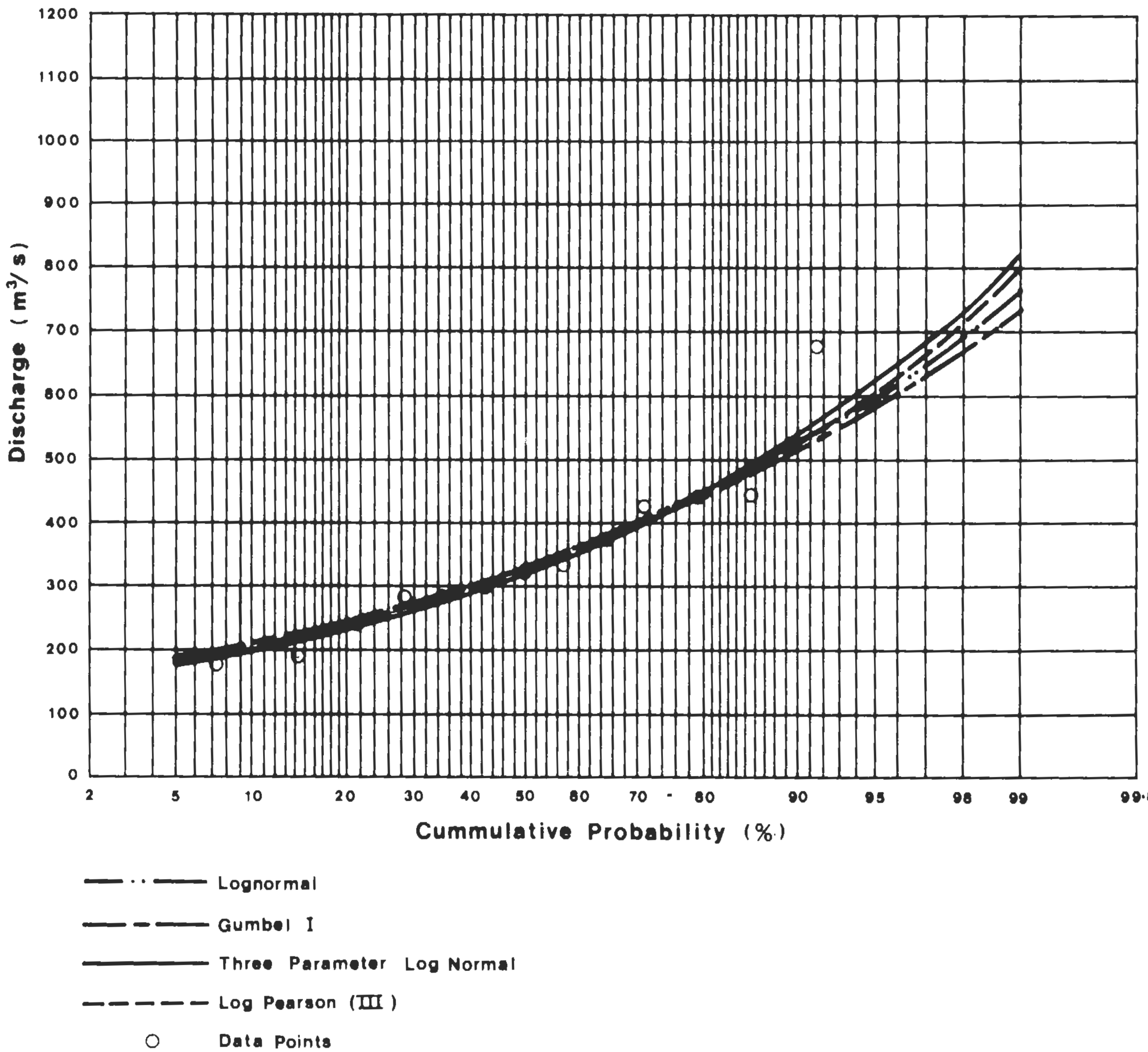



Canada-Newfoundland
Flood Damage Reduction Program

**Harrys River**
**Temporal Distribution for Trend And Persistance**

Nolan Davis & Associates Ltd. Cumming-Cockburn & Associates Ltd.

**Figure B.2**




| | Sample Statistics | | |
|---|---|---|---|
| | X | Log X | Log (X-A) |
| Mean | 347 | 5·7876 | 5·5282 |
| S.D. | 134·1 | 0·3667 | 0·4737 |
| C.S. | 1·2766 | 0·1905 | -0·0814 |
| C.K. | 6·4651 | 4·1774 | 4·0202 |

Canada-Newfoundland
Flood Damage Reduction Program

**Harrys River**

**Flood Frequency Analysis**

Nolan Davis & Associates Ltd Cumming-Cockburn & Associates Ltd

**Figure B.3**

# Appendix C

# Background for Deterministic Analyses

# Appendix C Background for Deterministic Analyses

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# Appendix C Background for Deterministic Analyses

The following sections provide additional information concerning the methodology applied in determining the parameters for use in the HYMO model for peak flow estimates.

## C.1 Hydrological Input Parameters

In order to assess the response of the watershed to a rainfall event, the time to peak and recession constant for the unit hydrograph must be determined. The following describes the various methods used to develop these parameters.

### C.1.1 Time to Peak and Recession Constant

a) The assessment of the time to peak parameter, $T_p$, for each subwatershed involved the application of the following equation developed for HYMO (44):

$$T_p = 4.63\ DA^{.422}\ SLP^{-.46}\ (L^2/DA)^{.133} \qquad (C.1)$$

where : DA = area of watershed ($mi^2$)
SLP = slope of watershed (ft/mi)
L = length of watershed (mi)

b) The initial determination of the hydrograph recession constant, K, involved the application of the following equation as given in the publication on the program HYMO (44):

$$K = 27.0\ DA^{.231}\ SLP^{-.777}\ (L^2/DA)^{.124} \qquad (C.2)$$

where $DA$ = watershed area ($mi^2$)

$L$ = watercourse length (mi)

$SLP$ = slope of watercourse (ft/mi)

NOTE: The above equations were originally (empirically) developed using the English system of units.

The initial parameter estimates are summarized in Table C.1.

### C.1.2 Model Calibration

The hydrologic model was calibrated by computing unit hydrograph parameters ($K$ and $T_p$) at the gauge location and for each subcatchment. In order to determine the unit hydrograph parameters of the watershed at the gauge location, the Collins hydrograph deconvolution method (31) was applied to a number of historical runoff events. This method was selected as it is one of few ways to separate a runoff event into a series of incremental hydrographs which are directly proportional (peak flow rate and runoff volume) to consecutive periods of varying runoff intensity. The Collins method involves finding a set of coefficients which describe the unit hydrograph shape by a series of trial and error approximations. The coefficients of the unit hydrograph were obtained for a number of events which were used to obtain an average set of values. From the deconvoluted unit hydrograph, parameters $K$ and $T_p$ were calculated directly and compared with the values estimated from equations C.1 and C.2.

It was found that the parameter values calculated using the empirical equations had to be increased slightly to match the deconvoluted unit hydrograph characteristics. The subwatershed unit hydrograph parameters were adjusted accordingly and are presented in Table C.2.

### C.1.3 Soil Cover Complex Number

The soil cover complex number, for each of the 8 subcatchments which comprise the Blanche Brook watershed, was calculated based on the method outlined by the Soil Conservation Service (41)

Using the values summarized in Table C.3, weighted curve numbers as presented in Table C.4 were calculated for each subwatershed.

CN numbers for average soil moisture conditions can be converted to the AMC III condition (saturated soil moisture conditions) by utilizing the relationships given in Table C.5.

### C.1.4 Reservoir Characteristics

As described in the report, Noels Pond was modelled using the storage indication method available as a subroutine in the HYMO computer model. Field measurements were obtained for the outlet structures and topographic maps were used to obtain the available storage.

The outflow/storage characteristics of Noels Pond were obtained from existing 1:12,500 scale mapping and hydraulic computations for the outlet structure. The preliminary HEC-2 backwater model was utilized to obtain outflow rates from the pond during high flow periods when the structure operates as a submerged weir. Manual calculations were utilized to compute outflows during low flow periods.

Based on a review of the reservoir operations procedures, the reservoir was assumed to have the stop logs in place for summer design events (rainfall) and the stop logs removed for spring design events (rainfall plus snowmelt). Table C.6 presents the outflow storage characteristics for each condition at the outlet structure.

## C.2 Rainfall Analysis

An analysis of historical rainfall events was carried out to determine a temporal distribution for the design rainfall events. The events selected were recent storms which occurred in the vicinity of the study area and are representative of rainfall events for Blanche Brook. Dimensionless mass curves were derived for each storm and an average temporal distribution was determined.

The resulting distribution was also compared to recent analyses by the Atmospheric Environment Services (AES) of Environment Canada for the Atlantic Provinces and it was found that both temporal distributions were very similar. The AES distribution was used to simulate design events since it would result in slightly more conservative peak flow estimates. Table C.7 presents a comparison of the two rainfall temporal distributions computed.

Figure C.1 presents the intensity duration characteristics of rainfall (summer and spring) and rainfall plus melt events. Total precipitation amounts for various storms are also presented in Table C.8.

## C.3 Snowmelt Analysis

### C.3.1 Atmospheric Environment Service Snowmelt Analysis

As discussed in the main text, the snowmelt analysis was undertaken using an algorithm derived from an energy budget method developed by the U.S. Army Corps of Engineers for forested areas. This algorithm uses recorded meteorologic data to compute various return frequency snowmelt amounts. It is preferred to use recorded meteorologic data because major spring runoff events are usually a combination of rainfall plus snowmelt conditions. Because the meteorologic inputs to such events (rainfall, temperature, snowpack, etc.) are not mutually exclusive (i.e., are somewhat dependent upon each other), their combined probability is not easily calculated.

Therefore, the rainfall plus snowmelt was established by analysis of available data for the Stephenville meteorologic station. This analysis was undertaken by staff of AES and uses historical meteorologic data to obtain the 1 to 10 day melt plus rainfall totals available for runoff.

## C.3.2 Secondary Snowmelt Analysis

A secondary snowmelt analysis was undertaken comprised of using a detailed energy budget equation represented by the following equation:

$$M = M_{rs} + M_{rl} + M_{cc} + M_p + M_g \qquad (C\text{-}3)$$

where $M$ = snowmelt
$M_{rs}$ = melt due to short wave radiation
$M_{rl}$ = melt due to long wave radiation
$M_{cc}$ = melt due to condensation and convection
$M_p$ = melt due to rainfall
$M_g$ = melt due to heat transfer from the ground

Utilizing the rainfall intensities at Stephenville Airport for the available years of record (1968 - 1981), a frequency analysis of the rainfall amounts for various durations was undertaken and Rainfall-Intensity-Duration curves were established for estimating design rainfall events which could occur during snowmelt conditions. The curves on Figure C.1 summarize the results of the rainfall analysis.

The above mentioned equation was then reduced to a function of two meteorologic variables (rainfall and mean daily temperature) following procedures given by Gray (31). Generally speaking, these variables are not independent, and hence it is difficult to determine probabilities associated with a design melt plus rain event. This analysis was only used as a secondary check of the 24-hour rain plus melt amounts calculated using actual events.

Substituting the total 24-hour spring rainfall amounts and the mean daily temperature for May into the detailed energy budget equation, a total 24-hr. 1:100 yr. rainfall plus snowmelt amount of 102.7 mm was obtained. It can be seen from Table C.8 that this value compares favourably with the AES analysis of meteorological records. The selection of the May mean daily air temperature was somewhat arbitrary. However, as a secondary check, it demonstrates that realistic meteorologic data can be combined to produce the design melt plus rain amounts.

It was, therefore, concluded that the Atmospheric Environment Service analysis was substantiated by the secondary snowmelt analysis and hence was used in the flow simulations.

### C.3.3 Snowmelt Flow Simulations

The 1:100 and 1:20 year spring snowmelt plus rainfall design events were also simulated with the HYMO model. Based on the spring snowmelt flow simulations, the 1:100 year and 1:20 year peak flows at the gauge location were computed to be 130.3 $m^3/s$ and 83.8 $m^3/s$ respectively. As discussed in Section 3.0, these spring peak flow estimates were found to be considerably smaller in magnitude than comparable summer rainfall events (see Table 3.6). Therefore, it was concluded that summer meteorologic design events should be utilized for estimating peak design flow rates within the study area.

TABLE C.1

Summary of Computed Unit Hydrograph Parameters

| Subwatershed | K* (hrs) | $T_p$* (hrs) |
|---|---|---|
| 101 | 2.47 | 2.86 |
| 102 | 1.33 | 1.45 |
| 103 | 0.40 | 0.31 |
| 201 | 1.54 | 1.96 |
| 202 | 1.61 | 1.00 |
| 203 | 0.69 | 0.74 |
| 204 | 0.60 | 0.44 |
| 300 | 0.82 | 0.52 |
| Gauge | 2.80 | 3.84 |

* Parameter values are initial estimates from the empirical equations C.1 and C.2

TABLE C.2

Summary of Calibrated Unit Hydrograph Parameters

| Subwatershed | K* (hrs) | $T_p$** (hrs) |
|---|---|---|
| 101 | 2.94 | 3.27 |
| 102 | 1.58 | 1.65 |
| 103 | 0.48 | 0.35 |
| 201 | 1.84 | 2.24 |
| 202 | 1.91 | 1.15 |
| 203 | 0.82 | 0.84 |
| 204 | 0.71 | 0.51 |
| 300 | 0.97 | 0.59 |
| at Gauge | 3.34 | 4.40 |

* Calibrated K Parameter = 1.19 x Empirically computed K Parameter

** Calibrated $T_p$ Parameter = 1.14 x Empirically computed $T_p$ Parameter

TABLE C.3

Summary of Typical Runoff Curve Numbers

| | Hydrologic Soil Group* | | | | |
|---|---|---|---|---|---|
| Land Use | B | BC | C | CD | D |
| Residential | 85 | 88 | 90 | 91 | 92 |
| Row Crop | 71 | 74 | 78 | 79 | 91 |
| Forest | 66 | 71 | 77 | 80 | 83 |
| Pasture | 69 | 74 | 79 | 82 | 84 |

* Based on Antecedent Moisture Condition II

Source: Reference (41)

TABLE C.4

Computed Subwatershed Curve Numbers

| Sub-Watershed | CN (AMC II) | CN (AMC III) |
|---|---|---|
| 101 | 73 | 87 |
| 102 | 73 | 87 |
| 103 | 76 | 89 |
| 201 | 75 | 88 |
| 202 | 82 | 92 |
| 203 | 75 | 88 |
| 204 | 78 | 90 |
| 300 | 80 | 91 |
| Weighted Average | 75 | 88 |

TABLE C.5

Relationship Between AMC II and AMC III

| CN for AMC II | Corresponding CN for AMC III |
|---|---|
| 100 | 100 |
| 95 | 98 |
| 90 | 96 |
| 85 | 94 |
| 80 | 91 |
| 75 | 88 |
| 70 | 85 |
| 65 | 82 |
| 60 | 78 |
| 55 | 74 |
| 50 | 70 |
| 45 | 65 |
| 40 | 60 |
| 35 | 55 |
| 30 | 50 |
| 25 | 43 |
| 20 | 37 |
| 15 | 30 |
| 10 | 22 |
| 5 | 13 |

AMC II The average condition
AMC III Highest runoff potential. Soils in the watershed are saturated from antecedent rains

Source: Reference (41)

TABLE C.6

Reservoir Characteristics

| Stop Logs Out Outflow ($m^3/s$) | Stop Logs Out Storage (hec-m) | Stop Logs In Outflow ($m^3/s$) | Stop Logs In Storage (hec-m)* |
|---|---|---|---|
| 0 | 0 | 0 | 0 |
| 1.3 | 8.7 | 1.0 | 1.7 |
| 3.6 | 23.3 | 4.0 | 3.6 |
| 3.9 | 33.1 | 7.0 | 8.2 |
| 4.2 | 35.1 | 9.5 | 13.3 |
| 4.6 | 36.8 | 12.0 | 18.1 |
| 4.9 | 38.7 | 15.0 | 24.4 |
| 8.0 | 43.3 | 17.0 | 30.7 |
| 11.5 | 48.4 | 20.5 | 37.7 |
| 14.9 | 53.2 | 24.5 | 44.6 |
| 16.0 | 59.5 | 28.0 | 54.3 |
| 18.0 | 65.8 | 31.5 | 61.2 |
| 22.0 | 72.8 | 35.0 | 70.9 |
| 25.5 | 79.7 | 39.0 | 76.1 |
| 29.0 | 89.4 | 55.0 | 81.7 |
| 33.0 | 96.3 | 64.0 | 85.9 |
| 37.0 | 106.0 | | |
| 44.5 | 111.2 | | |
| 62.0 | 116.8 | | |
| 73.0 | 121.0 | | |

* Storage in hectare-metres; note that one hectare-metre equals 10,000 $m^3$

TABLE C.7

Rainfall Distribution Comparison

for the 12-hour Design Storms (a)

| Time (hours) | (b) Design Storm Distribution Estimated by Analysis of Historical Events | (c) AES 12-hour Design Storm Distribution (%) (4) |
|---|---|---|
| 0 - 1 | 3 | 5 |
| 1 - 2 | 11 | 8 |
| 2 - 3 | 8 | 8 |
| 3 - 4 | 6 | 10 |
| 4 - 5 | 11 | 10 |
| 5 - 6 | 13 | 14 |
| 6 - 7 | 9 | 13 |
| 7 - 8 | 13 | 8 |
| 8 - 9 | 10 | 10 |
| 9 - 10 | 9 | 8 |
| 10 - 11 | 6 | 4 |
| 11 - 12 | 1 | 2 |
| | 100 | 100 |

Notes:

(a) Percent of total rainfall over 12-hour period. Distribution given reflects the percentage of the total rainfall that falls over the period

(b) Source - Analysis of historical rainfall events by CCL, 1983

(c) Source - Analysis by Environment Canada (4)

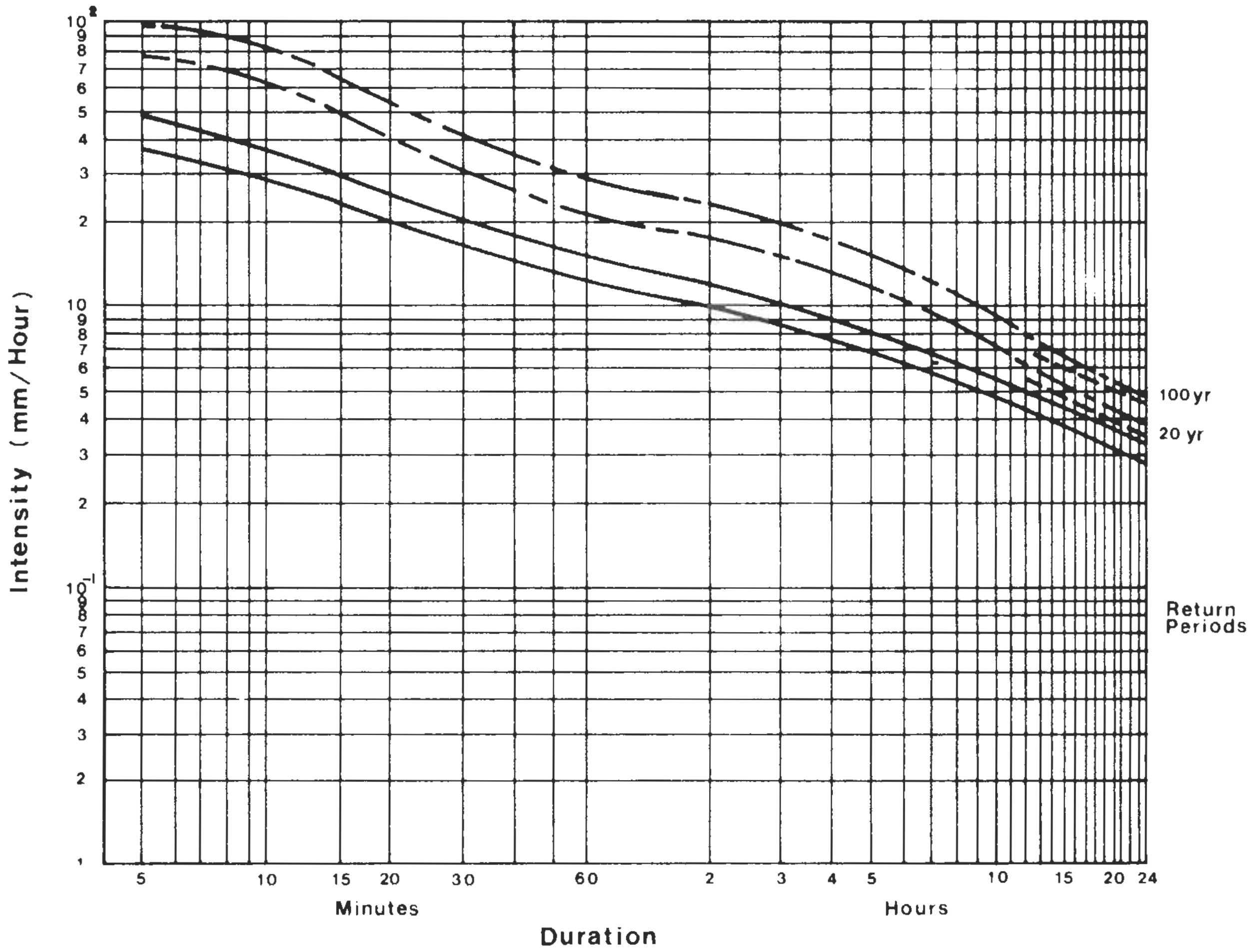
TABLE C.8

Precipitation Summary

| Return Period (years) | Rainfall* (mm) 6 hr. | 12 hr. | 24 hr. | Rain + Melt** (mm) 24 hr. |
|---|---|---|---|---|
| 2 | 35.88 | 45.48 | 56.4 | 38.28 |
| 5 | 47.94 | 58.08 | 72.29 | 56.62 |
| 10 | 55.98 | 66.48 | 82.8 | 68.77 |
| 25 | 66.06 | 77.04 | 96.24 | 84.12 |
| 50 | 73.50 | 84.84 | 106.08 | 95.50 |
| 100 | 80.94 | 92.64 | 115.92 | 106.30 |

* based on AES analysis at Stephenville meteorological station (13 yrs. data)

** based on AES analysis of melt plus rain events




Rainfall (spring)

Rainfall & Snowmelt (AES)

Rainfall (summer)

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**Rainfall and Snowmelt**
**Intensity Duration Curves**

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**Figure C.1**

# Appendix D

## Summary of Computed Water Surface Elevations

# APPENDIX D

## SUMMARY OF COMPUTED WATER SURFACE ELEVATIONS

Table D.1 presents a tabular summary of the 1:20 and 1:100 year flood elevations. Refer to Figure 4.3 for approximate cross-section locations and to Figures 4.6 and 4.7 for the corresponding hydraulic profiles.

The areal extent of the flood prone area is delineated on topographic maps at a scale of 1:2500 for the study area.

## APPENDIX D

### TABLE D.1

Summary of Computed Water Surface Elevations
- Blanche Brook -

| Section No. | Location/ Description | 1:20 Year Water Level (m) | 1:100 Year Water Level (m) | | |
|---|---|---|---|---|---|
| | | Free Flow Condition | Free Flow Condition | Free Flow Condition (Using Upper 95% C.L. Discharge) | Assuming 50% Blockage of Hydraulic Structure |
| Blanche Brook: | | | | | |
| 881.0 | | 1.5 | 1.5 | 1.7 | 1.7 |
| 882.1 | Kin Place Bridge | 1.9 | 2.2 | 3.1 | 4.5 |
| 882.0 | | 1.9 | 2.8 | 5.7 | 5.7 |
| 882.2 | | 2.1 | 2.9 | 5.7 | 5.7 |
| 882.3 | | 2.1 | 3.0 | 5.7 | 5.7 |
| 883.0 | | 2.9 | 4.0 | 5.7 | 5.7 |
| 884.0 | | 3.1 | 4.1 | 5.8 | 5.7 |
| 885.0 | | 3.7 | 4.4 | 5.7 | 5.7 |
| 886.0 | Water Survey Canada | 4.9 | 5.4 | 6.4 | 6.4 |
| 888.0 | Gauge No. 02YJ002 | 6.8 | 7.4 | 8.0 | 8.0 |
| 887.1 | Minnesota Dr. Bridge | 7.3 | 7.8 | 8.5 | 8.5 |
| 887.0 | | 7.3 | 7.8 | 8.5 | 8.5 |
| 889.0 | | 7.3 | 7.8 | 8.5 | 8.5 |
| 8810.0 | | 7.7 | 8.0 | 8.5 | 8.6 |
| 8811.0 | | 8.7 | 9.0 | 9.5 | 9.9 |
| 8812.0 | | 9.8 | 10.1 | 9.9 | 9.9 |
| 8813.1 | Main Street Bridge | 9.9 | 10.2 | 10.3 | 10.7 |

TABLE D.1 (cont'd)

| Section No. | Location/ Description | 1:20 Year Water Level (m) Free Flow Condition | 1:100 Year Water Level (m) Free Flow Condition | 1:100 Year Water Level (m) Free Flow Condition (Using Upper 95% C.L. Discharge) | 1:100 Year Water Level (m) Assuming 50% Blockage of Hydraulic Structure |
|---|---|---|---|---|---|
| Blanche Brook: | | | | | |
| 8813.0 | | 10.4 | 10.8 | 11.5 | 11.5 |
| 8813.2 | | 10.4 | 10.8 | 11.5 | 11.5 |
| 8813.3 | | 10.4 | 10.8 | 11.5 | 11.6 |
| 8814.0 | | 10.6 | 11.0 | 11.7 | 11.7 |
| 8815.0 | | 11.8 | 12.0 | 12.4 | 12.3 |
| 8816.0 | | 14.3 | 14.5 | 15.2 | 15.2 |
| 8817.0 | | 18.4 | 18.6 | 19.3 | 19.3 |
| 8818.0 | | 19.9 | 20.1 | 20.5 | 20.5 |
| 8819.0 | | 23.0 | 23.2 | 23.4 | 23.4 |
| 8820.0 | | 25.2 | 25.5 | 25.9 | 25.9 |
| 8821.0 | | 27.1 | 27.4 | 27.8 | 27.8 |
| 8822.1 | Hansen Hwy. Bridge | 28.1 | 28.4 | 28.8 | 28.8 |
| 8822.0 | (Route 460) | 28.2 | 28.5 | 28.9 | 30.0 |
| 8823.0 | | 28.4 | 28.7 | 29.1 | 30.3 |
| 8824.0 | | 30.8 | 31.0 | 31.3 | 31.3 |
| 8825.0 | | 34.6 | 34.9 | 35.2 | 35.2 |

APPENDIX D

TABLE D.2

Summary of Computed Water Surface Elevations
- Warm Creek -

| Section No. | Location/ Description | 1:20 Year Water Level (m) Free Flow Condition | 1:100 Year Water Level (m) Free Flow Condition | 1:100 Year Water Level (m) Free Flow Condition (Using Upper 95% C.L. Discharge) | 1:100 Year Water Level (m) Assuming 50% Blockage of Hydraulic Structure |
|---|---|---|---|---|---|
| Warm Creek: | | | | | |
| 221.0 | | 7.0 | 7.5 | 8.1 | 8.1 |
| 222.0 | | 7.7 | 8.1 | 8.4 | 8.4 |
| 223.1 | Carolina Ave. Bridge | 8.3 | 8.6 | 9.0 | 9.1 |
| 223.0 | | 8.3 | 9.2 | 9.3 | 9.3 |
| 223.2 | | 8.4 | 9.2 | 9.3 | 9.3 |
| 223.3 | | 8.4 | 9.2 | 9.3 | 9.3 |
| 224.0 | | 8.8 | 9.2 | 9.4 | 9.4 |
| 225.0 | | 9.0 | 9.3 | 9.5 | 9.5 |
| 226.1 | Mississippi Dr. Bridge | 9.9 | 10.1 | 11.1 | 11.1 |
| 226.0 | | 10.2 | 10.5 | 11.1 | 11.2 |
| 227.0 | | 10.5 | 11.0 | 11.3 | 11.3 |
| 228.0 | | 10.7 | 11.1 | 11.3 | 11.3 |
| 229.0 | | 12.0 | 12.3 | 12.8 | 12.8 |
| 2210.0 | | 14.3 | 14.6 | 14.8 | 14.8 |
| 2211.1 | Weir near Transformer | 15.6 | 15.8 | 16.2 | 16.2 |
| 2211.0 | Sub-station (upstream | 16.6 | 16.9 | 17.3 | 17.3 |
| 2211.2 | of Mississippi Bridge) | 16.6 | 17.0 | 17.3 | 17.3 |

TABLE D.2 (cont'd)

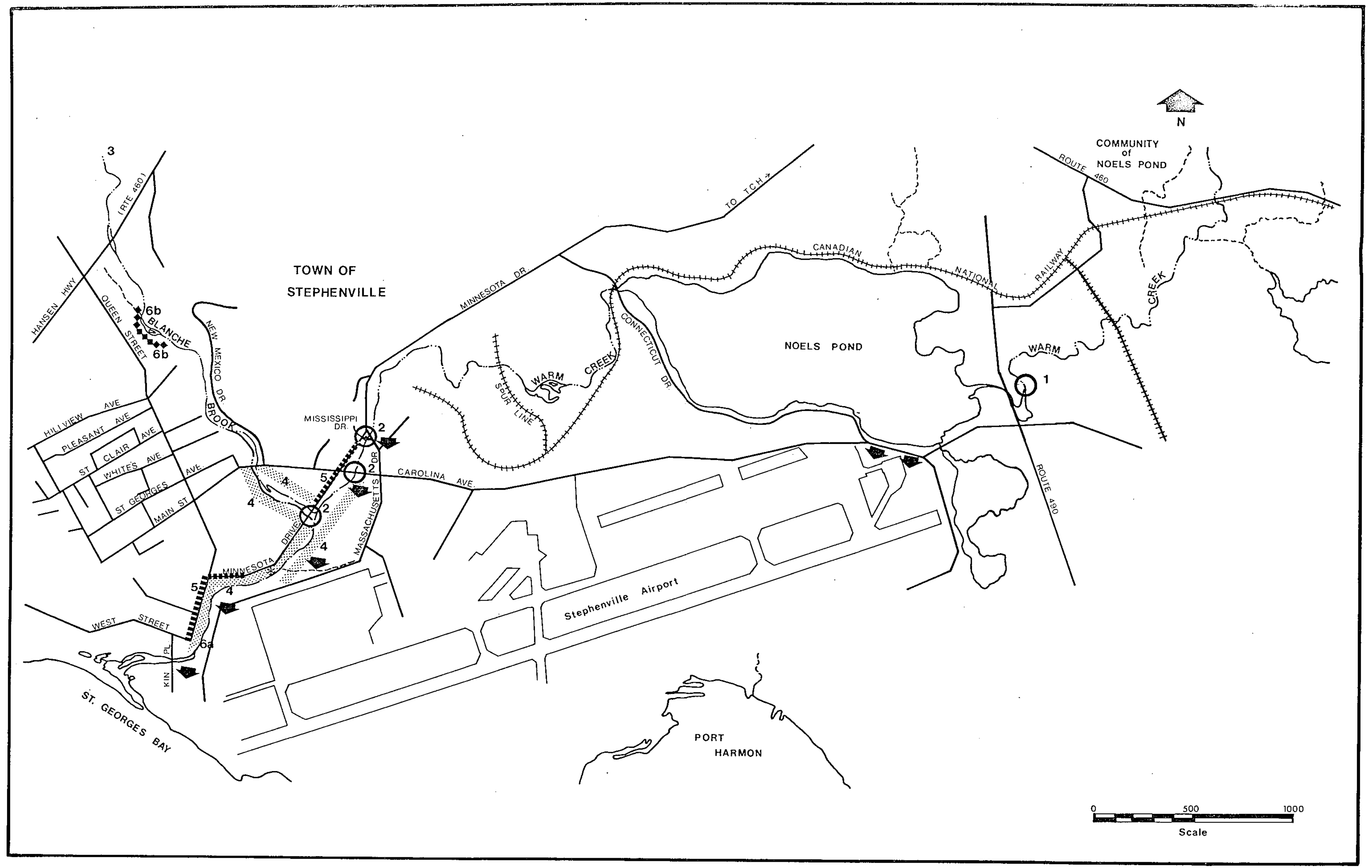
| Section No. | Location/ Description | 1:20 Year Water Level (m) Free Flow Condition | 1:100 Year Water Level (m) Free Flow Condition | 1:100 Year Water Level (m) Free Flow Condition (Using Upper 95% C.L. Discharge) | 1:100 Year Water Level (m) Assuming 50% Blockage of Hydraulic Structure |
|---|---|---|---|---|---|
| Warm Creek: | | | | | |
| 2212.0 | | 17.4 | 17.7 | 18.1 | 18.1 |
| 2213.0 | | 18.2 | 18.5 | 18.9 | 18.9 |
| 2214.0 | | 18.4 | 18.7 | 19.2 | 19.2 |
| 2215.0 | | 20.2 | 20.5 | 21.4 | 21.4 |
| 2216.0 | | 20.6 | 21.0 | 21.9 | 21.9 |
| 2217.1 | Railway Spur Line | 22.1 | 22.5 | 22.7 | 22.7 |
| 2217.0 | near Baseball Field | 22.8 | 22.9 | 23.0 | 23.0 |
| 2218.0 | | 22.8 | 22.9 | 23.0 | 23.0 |
| 2219.0 | | 22.8 | 22.9 | 23.0 | 23.0 |
| 2220.0 | | 22.8 | 23.0 | 23.1 | 23.1 |
| 2221.0 | | 22.9 | 23.1 | 23.3 | 23.3 |
| 2222.1 | Railway Trestle d/s | 23.5 | 23.7 | 23.9 | 23.9 |
| 2222.0 | of Connecticut Dr. | 23.5 | 23.7 | 24.0 | 24.0 |
| 2222.2 | | 23.9 | 24.0 | 24.1 | 24.1 |
| 2222.3 | | 23.9 | 24.0 | 24.1 | 24.1 |
| 2223.0 | | 24.1 | 24.1 | 24.1 | 24.1 |
| 2224.1 | Connecticut Dr. Bridge | 24.1 | 24.1 | 24.2 | 24.2 |
| 2224.0 | | 24.1 | 24.1 | 24.3 | 24.5 |
| 2224.2 | | 24.1 | 24.1 | 24.3 | 24.5 |
| 2224.3 | Noels Pond Outlet Weir | 24.2 | 24.2 | 24.4 | 24.6 |
| 2224.4 | | 24.2 | 24.2 | 24.4 | 24.6 |

TABLE D.2 (cont'd)

| Section No. | Location/ Description | 1:20 Year Water Level (m) Free Flow Condition | 1:100 Year Water Level (m) Free Flow Condition | 1:100 Year Water Level (m) Free Flow Condition (Using Upper 95% C.L. Discharge) | 1:100 Year Water Level (m) Assuming 50% Blockage of Hydraulic Structure |
|---|---|---|---|---|---|
| Warm Creek: | | | | | |
| 2225.1 | Rte 490 Culverts | 24.4 | 24.5 | 24.5 | 24.6 |
| 2225.2 | (Noels Pond Inlet) | 24.3 | 24.4 | 24.3 | 24.6 |
| 2225.3 | | 24.3 | 24.5 | 24.4 | 24.6 |
| 2225.4 | | 24.5 | 24.8 | 25.3 | 26.4 |
| 2225.0 | | 24.6 | 24.9 | 25.5 | 26.4 |
| 2226.0 | | 24.6 | 24.9 | 25.5 | 26.4 |
| 2227.0 | | 24.6 | 24.9 | 25.5 | 26.4 |
| 2227.1 | Railway Trestle u/s of | 24.6 | 25.0 | 25.5 | 26.4 |
| 2228.1 | Route 490 | 24.8 | 25.2 | 25.6 | 26.4 |
| 2228.2 | | 25.2 | 25.3 | 25.6 | 26.4 |
| 2228.0 | | 25.3 | 25.4 | 25.6 | 26.4 |
| 2229.0 | | 25.4 | 25.4 | 25.6 | 26.4 |
| 2230.0 | | 25.4 | 25.5 | 25.7 | 26.4 |
| 2231.0 | | 25.6 | 25.7 | 25.9 | 26.4 |
| 2231.8 | Railway trestle d/s | 25.6 | 25.7 | 25.6 | 26.4 |
| 2231.9 | of Route 460 | 25.6 | 25.7 | 25.9 | 26.4 |
| 2231.1 | Route 460 Bridge near | 25.8 | 25.9 | 26.4 | 26.4 |
| 2231.2 | Community of Noels Pond | 25.8 | 26.7 | 26.7 | 26.7 |
| 2231.3 | (Hansen Hwy) | 26.7 | 26.7 | 26.7 | 26.7 |
| 2231.4 | | 26.7 | 26.7 | 26.7 | 26.7 |
| 2231.5 | | 26.7 | 26.7 | 26.7 | 26.7 |

TABLE D.2 (cont'd)

| Section No. | Location/ Description | 1:20 Year Water Level (m) Free Flow Condition | 1:100 Year Water Level (m) Free Flow Condition | | |
|---|---|---|---|---|---|
| | | | | Free Flow Condition (Using Upper 95% C.L. Discharge) | Assuming 50% Blockage of Hydraulic Structure |
| Warm Creek: | | | | | |
| 2232.0 | | 25.7 | 26.7 | 26.7 | 27.0 |
| 2233.0 | | 26.7 | 26.8 | 26.8 | 27.0 |
| 2234.0 | | 28.1 | 28.2 | 28.4 | 28.4 |
| 2235.0 | | 29.6 | 29.8 | 29.9 | 29.9 |
| 2236.0 | | 33.5 | 33.6 | 34.1 | 34.1 |

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## Legend

Earth berm

Raise road profile where shown

Gabion protection

Replace/enhance existing structure

2 Remedial measure alternative

Potential spill location under existing condition

Refer to Section 5.2 for specific description of potential improvement measure.

## Index to Improvement Schemes

1 Replace Route 490 Culverts

2 Replace structures at: Mississippi Dr., Carolina Ave. and Minnesota Dr.

3 Upstream Detention Facility

4 Dykes/berms south of Main St.

5 Raising of Minnesota Dr.

6 -Debris clearing
- Dredging, Channel widening
- Gabion works

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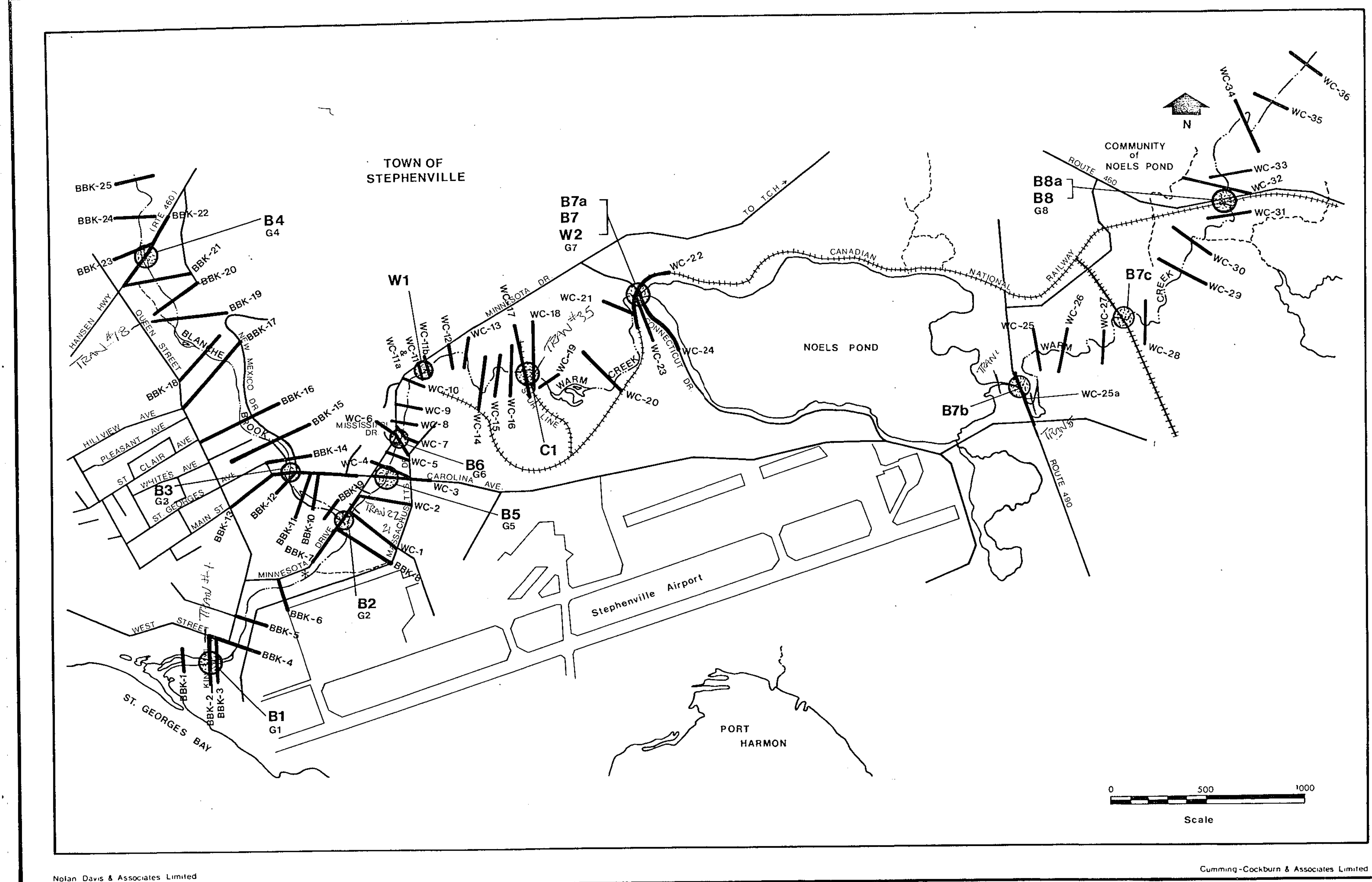
**Location of Potential Remedial Work Schemes**

Figure 5.1




## Legend

Cross-Section Location/no.

Structure Location/no.

B3
G3

Guage Location

## Index

| | |
|---|---|
| BBK | Blanche Brook Cross-Section |
| WC | Warm Creek Cross-Section |
| B | Bridge Structure |
| C | C.M.P. Structure |
| W | Weir Structure |

Refer to Supplemental Report "Physical Surveys and Field Program" for details on cross-section and hydraulic structure characteristics.

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**Location of Hydraulic Structures and Field Surveyed Cross-Sections**

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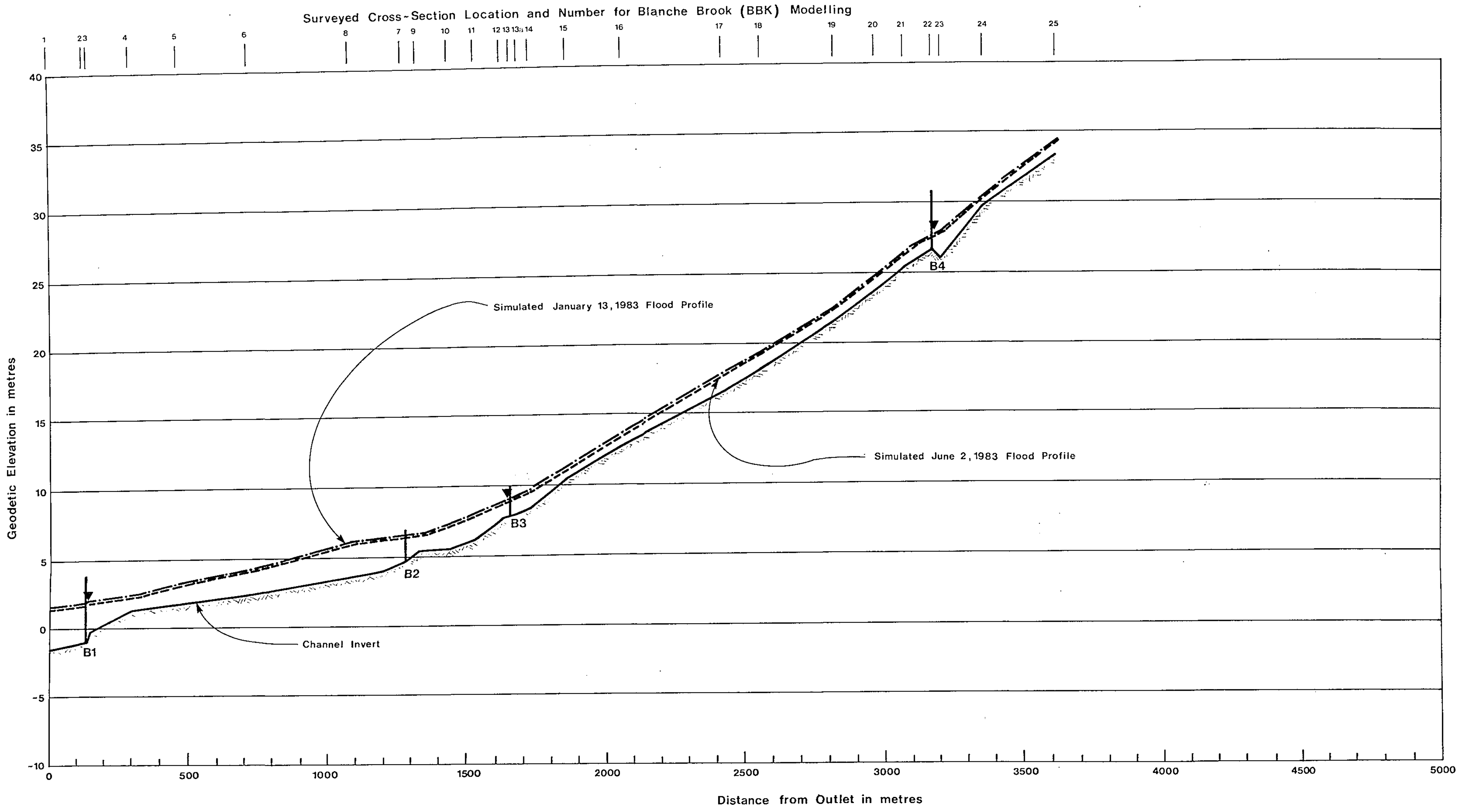
**Figure 4.3**




### Hydraulic Structure Characteristic Index

| Number | Location | Type | Elevations (m) Soffit | Deck | Approach* |
|---|---|---|---|---|---|
| B1 | Kin Place | Conc. Box Culvert | 3.86 | 4.70 | 3.74 |
| B2 | Minnesota Drive | Wood Support | 7.29 | 8.18 | 6.82 |
| B3 | Main Street | Wood / Conc. Support | 10.68 | 11.57 | 9.94 |
| B4 | Hansen Highway | Conc. Support | 29.74 | 31.34 | 30.78 |

*Minimum elevation of road profile. Control elevation for potential surcharge of roadway.

**Legend**

▼ Recorded January 13,1983 Water Level

▽ Recorded June 2,1983 Water Level (Data Missing Due to Guage Vandalism)

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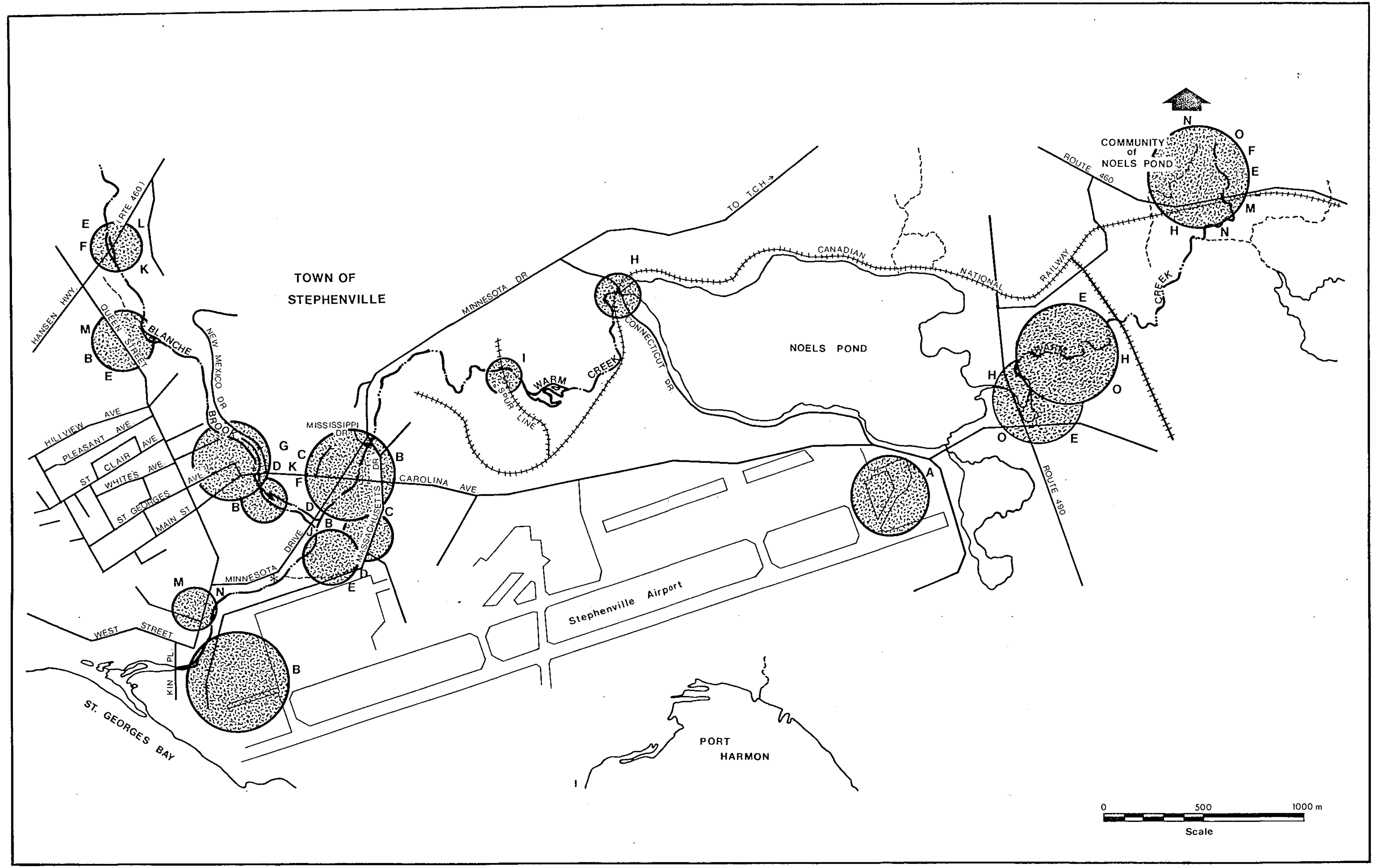
**Comparison of Measured and Calibrated Flood Profiles: - Blanche Brook -**

**Figure 4.4**

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## Index to Documented Floods

| Key | Flood Event |
|---|---|
| A | September 1-2, 1948 |
| B | May 19-21, 1969 |
| C | May 15-18, 1972 |
| D | February 2-3, 1973 |
| E | August 1-4, 1973 |
| F | November 1-2, 1974 |
| G | December 22-25, 1975 |
| H | December 27-29, 1977 |
| I | Sept. 30 - Oct. 2, 1978 |
| J | March 6-9, 1979 |
| K | July 17-20, 1979 |
| L | July 28-29, 1979 |
| M | April 22-30, 1982 |
| N | January 11-13, 1983 |
| O | June 1-3, 1983 |

Refer to Table 2.1 and Appendix A for detailed descriptions of historical floods.

Source: Reference (25)

### Legend

Flood Prone Areas

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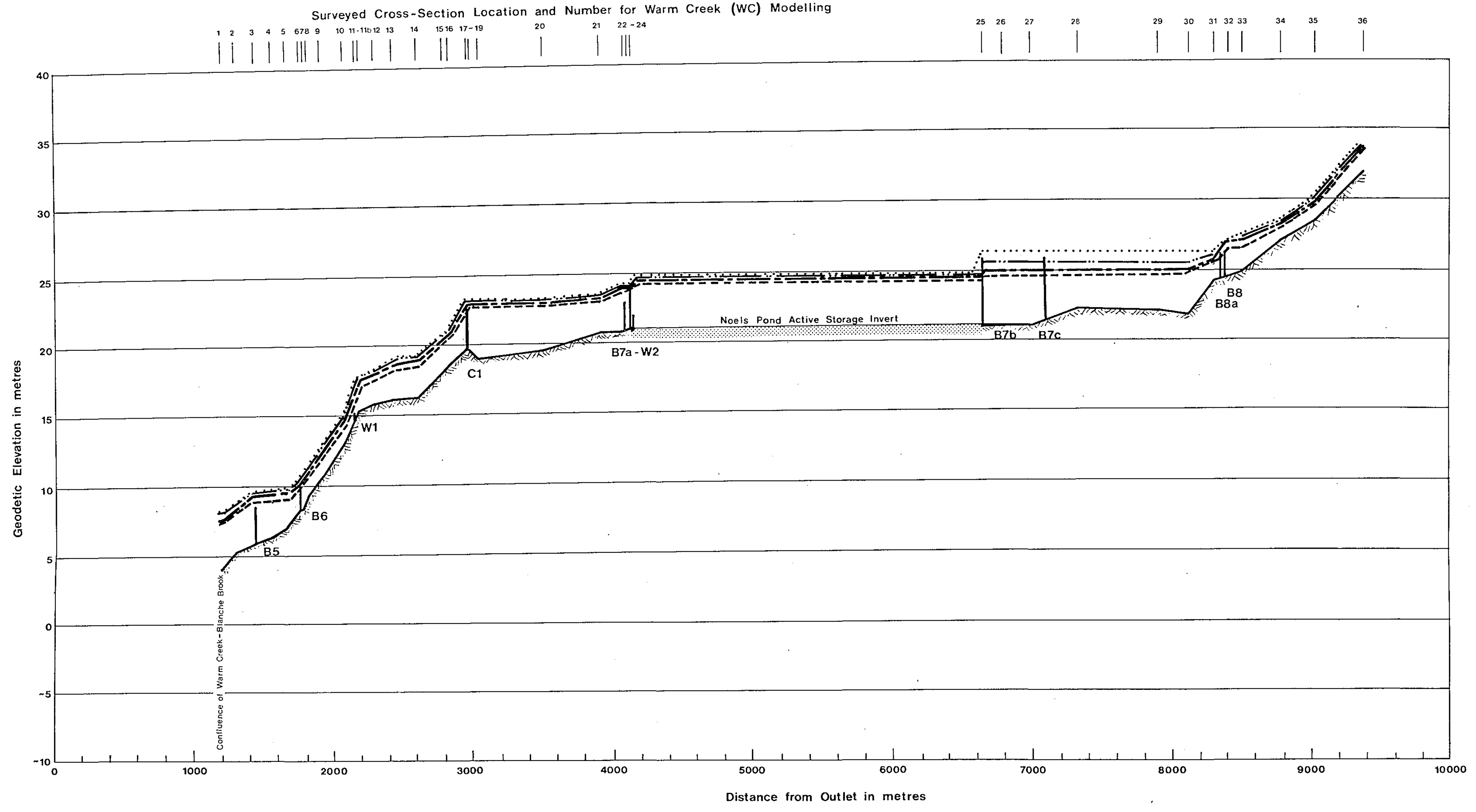
**Historical Flood Prone Areas**

**Figure 2.1**




## Hydraulic Structure Characteristic Index

| Number | Location | Type | Elevation (m) Soffit | Deck | Approach* |
|---|---|---|---|---|---|
| B5 | Carolina Avenue | Conc. Box Culvert | 8.58 | 9.50 | 8.35 |
| B6 | Mississippi Drive | Conc. Box Culvert | 10.09 | 10.79 | 9.98 |
| W1 | Transformer Sub-Station | Weir | N/A | 14.47 | N/A |
| C1 | Railway Spur Line | 4 C.M.P. Culverts | 21.05 | 22.47 | 22.47 |
| B7a | Connecticut Drive R.R. Trestle | Conc. Bridge | 23.15 | 23.80 | 22.93 |
| B7 | Connecticut Drive | Conc. Bridge | 24.07 | 24.69 | 24.04 |
| W2 | Noels Pond Outlet | Weir | N/A | 21.97 | N/A |
| B7b | Route 490 | 3 C.M.P. Culverts | 23.48 | 25.81 | N/A |
| B7c | CNR Trestle | Bridge | 24.40 | 24.90 | N/A |
| B8 | Route 460 | Conc. Box Culvert | 25.87 | 26.80 | 26.20 |
| B8a | CNR Trestle | Wood Bridge | 26.10 | 26.20 | N/A |

* Minimum Elevation of Road Profile Control Elevation for Potential Surcharge of Roadway

### Legend

- Channel Invert
- 1:20 Year Flood Profile
- 1:100 Year Flood Profile
- 1:100 Year Upper 95% Confidence Discharge
- 1:100 Year Upper 95% Confidence with Blockage

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**Hydraulic Profiles**
**- Warm Creek -**

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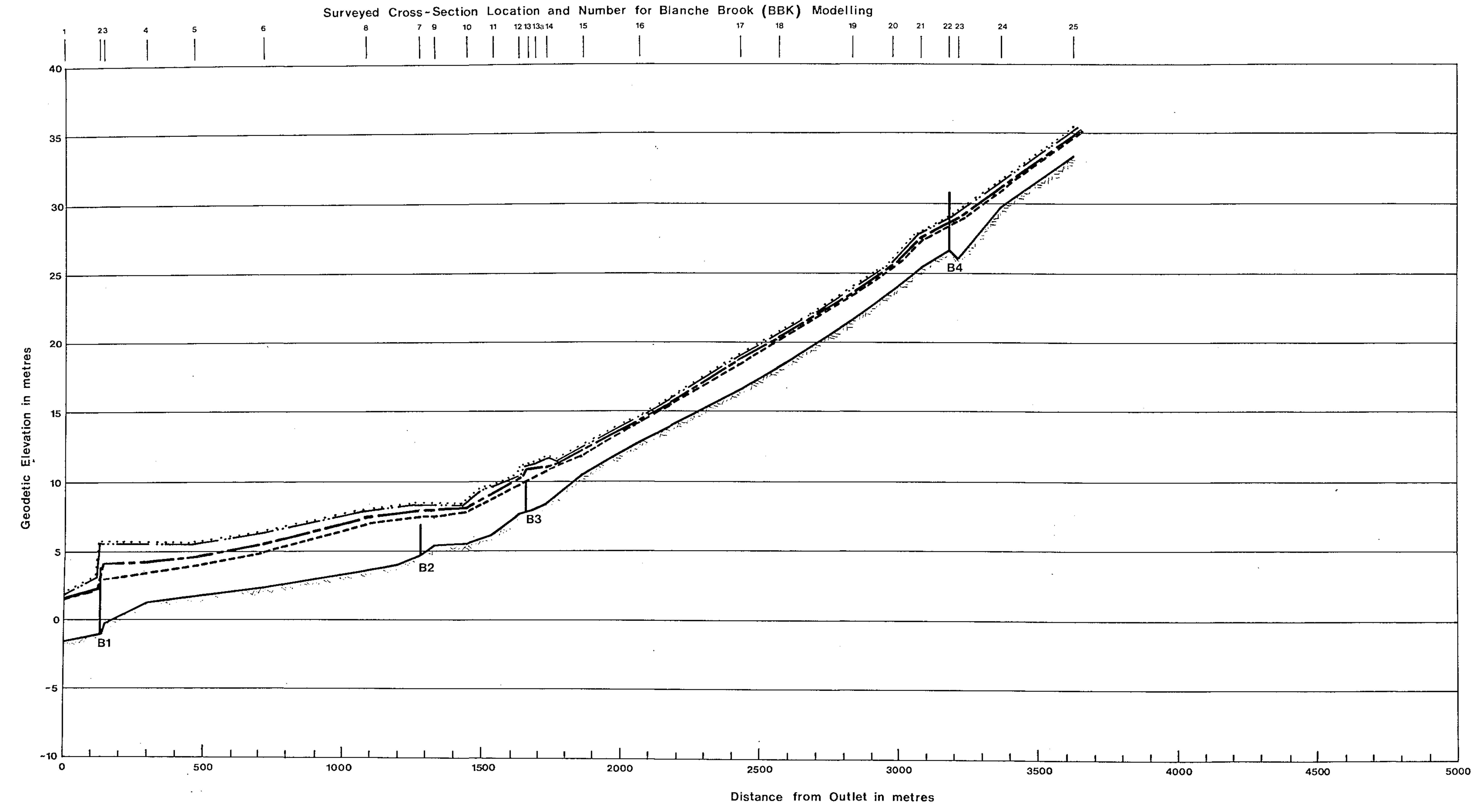
**Figure 4.7**




### Hydraulic Structure Characteristic Index

| Number | Location | Type | Elevations ( m ) Soffit | Deck | Approach* |
|---|---|---|---|---|---|
| B1 | Kin Place | Conc. Box Culvert | 3.86 | 4.70 | 3.74 |
| B2 | Minnesota Drive | Wood Support | 7.29 | 8.18 | 6.82 |
| B3 | Main Street | Wood / Conc. Support | 10.68 | 11.57 | 9.94 |
| B4 | Hansen Highway | Conc. Support | 29.74 | 31.34 | 30.78 |

*Minimum elevation of road profile. Control elevation for potential surcharge of roadway.

### Legend

Channel Invert
1:20 Year Flood Profile
1:100 Year Flood Profile
1:100 Year Upper 95% Confidence Discharge
1:100 Year Upper 95% Confidence with Blockage

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Flood Damage Reduction Program

**Hydraulic Profiles**
**- Blanche Brook -**

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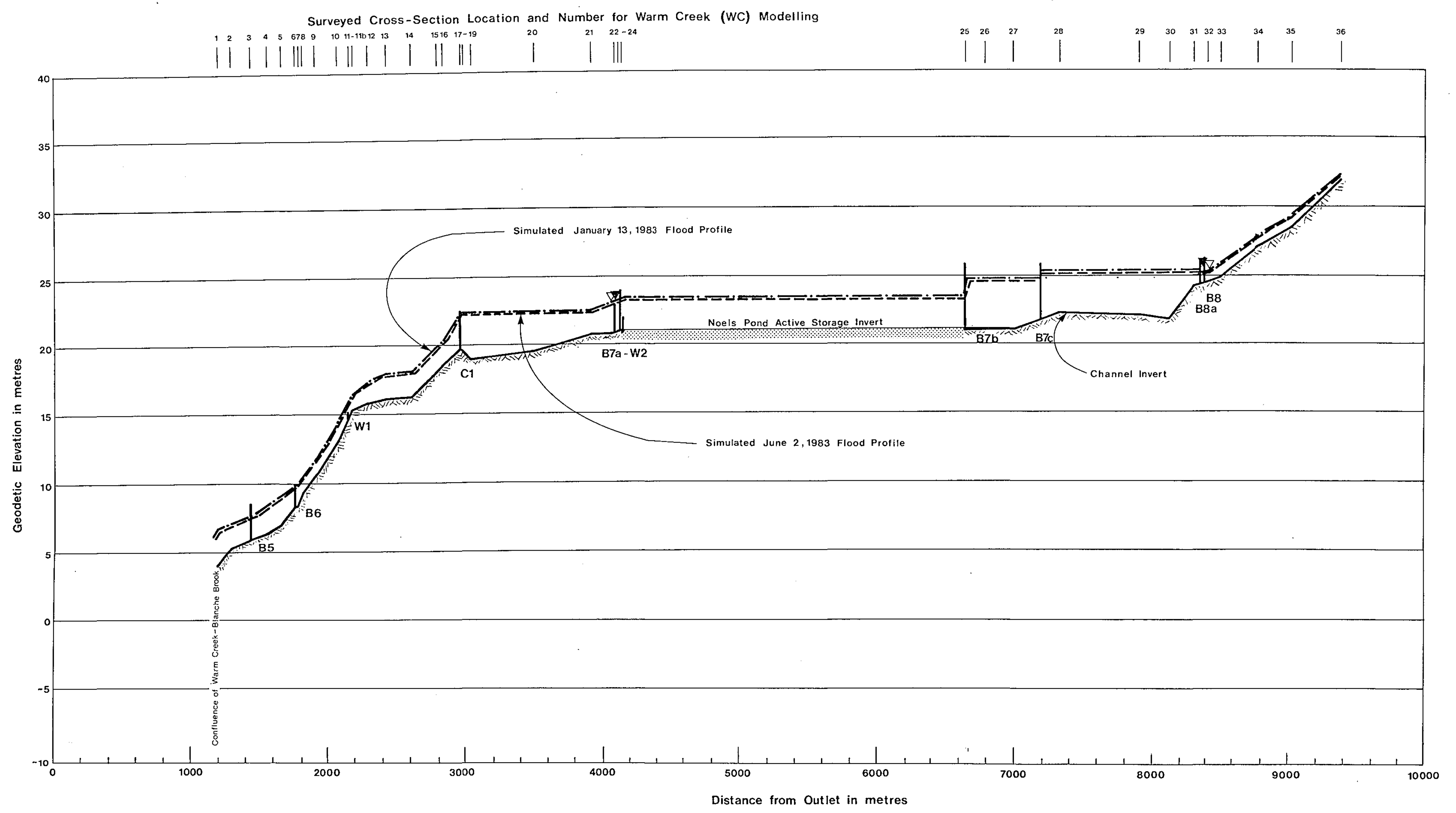
**Figure 4.6**




### Hydraulic Structure Characteristic Index

| Number | Location | Type | Elevation (m) Soffit | Deck | Approach* |
|---|---|---|---|---|---|
| B5 | Carolina Avenue | Conc. Box Culvert | 8.58 | 9.50 | 8.35 |
| B6 | Mississippi Drive | Conc. Box Culvert | 10.09 | 10.79 | 9.98 |
| W1 | Transformer Sub-Station | Weir | N/A | 14.47 | N/A |
| C1 | Railway Spur Line | 4 C.M.P. Culverts | 21.05 | 22.47 | 22.47 |
| B7a | Connecticut Drive R.R. Trestle | Conc. Bridge | 23.15 | 23.80 | 22.93 |
| B7 | Connecticut Drive | Conc. Bridge | 24.07 | 24.69 | 24.04 |
| W2 | Noels Pond Outlet | Weir | N/A | 21.97 | N/A |
| B7b | Route 490 | 3 C.M.P. Culverts | 23.48 | 25.81 | N/A |
| B7c | CNR Trestle | Bridge | 24.40 | 24.90 | N/A |
| B8 | Route 460 | Conc Box Culvert | 25.87 | 26.80 | 26.20 |
| B8a | CNR Trestle | Wood Bridge | 26.10 | 26.20 | N/A |

* Minimum Elevation of Road Profile Control Elevation for Potential Surcharge of Roadway

### Legend

▼ Recorded January 13, 1983 Water Level

▽ Recorded June 2, 1983 Water Level

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## Comparison of Measured and Calibrated Flood Profiles: -Warm Creek-

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