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and Labrador

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

Government
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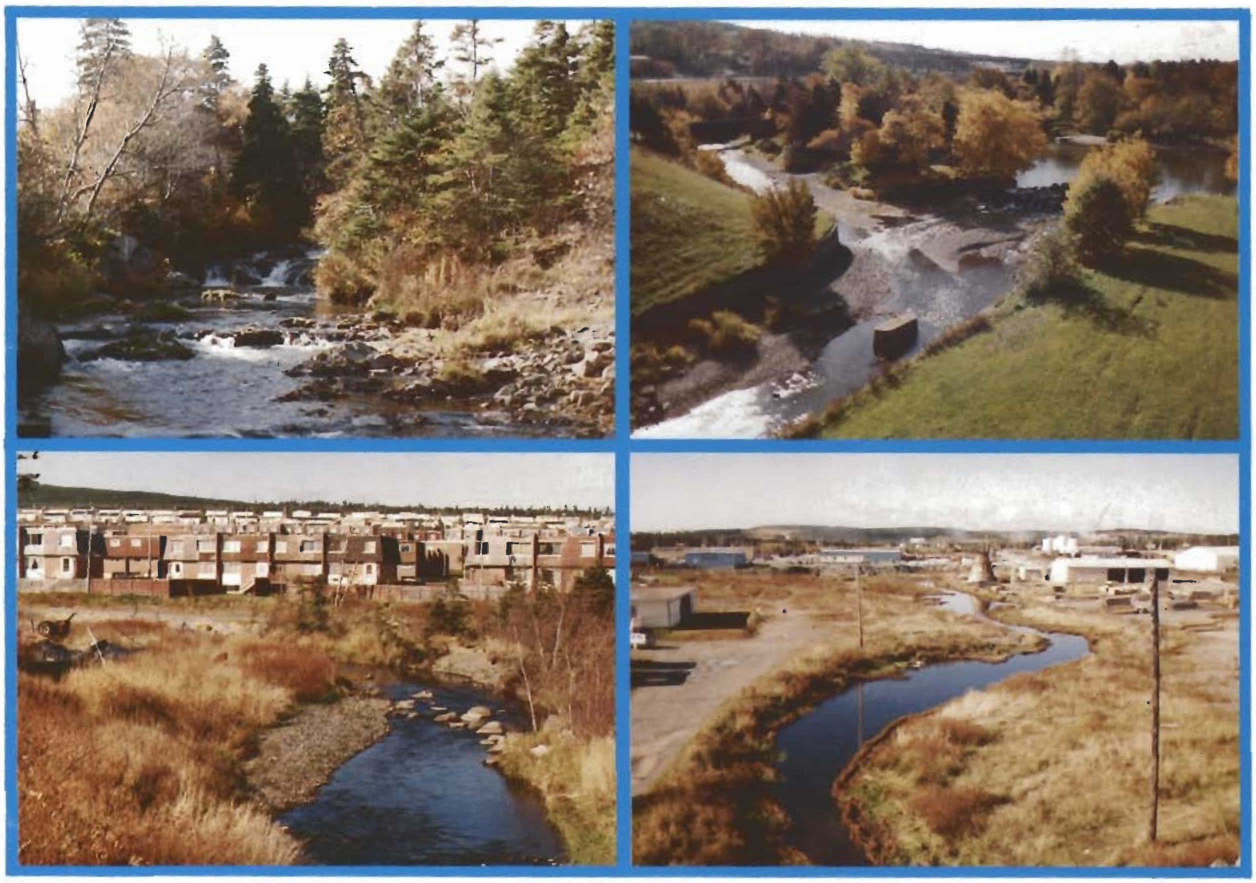
Environment Canada
Inland Waters Directorate
Dartmouth, Nova Scotia

National Water Research Institute
Burlington, Ontario

WATERSHED MODELLING REPORT
HYMO, 1988
MAIN REPORT



Urban Hydrology Study of the Waterford River Basin

TECHNICAL REPORT No.
UHS-WRB 1.10

EVALUATION OF URBANIZATION EFFECTS ON STORM RUNOFF CHARACTERISTICS IN THE WATERFORD RIVER BASIN (APPLICATION OF THE HYMO MODEL)

VOLUME 1

(MAIN REPORT)

WATER RESOURCES DIVISION
NEWFOUNDLAND DEPARTMENT OF ENVIRONMENT AND LANDS
ST. JOHN'S, NEWFOUNDLAND

WATER PLANNING & MANAGEMENT BRANCH
INLAND WATERS DIRECTORATE
CONSERVATION AND PROTECTION
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DARTMOUTH, NOVA SCOTIA

April 1988

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April 18, 1988

Your file Votre référence

Our file Notre référence

5100-15

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Dear Dr. Ullah:

On behalf of those who participated in the task, "Evaluation of Urbanization Effects on Storm Runoff Characteristics in the Waterford River Basin (Application of the HYMO model)", I am pleased to submit the final report.

Yours sincerely,

David A. Smith

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ABSTRACT

The Governments of Canada and the Province of Newfoundland agreed to undertake a five-year urban hydrology study of the Waterford River on a work shared basis starting in April, 1980. The study contained a number of components. For this component, a simple single event hydrologic model (HYMO) was applied to attempt to discern if urbanization between 1973 and 1981 was having a significant impact on peak flows resulting from rain storms. Estimates of 20 and 100 year peak flows were made for the flood study component, and an indication was provided of the potential effects on peak flows of further development in the study area.

RÉSUMÉ

Les gouvernements canadien et terre-neuvien ont entrepris, en avril 1980, une étude conjointe quinquennale concernant l'hydrologie urbaine du bassin versant de la rivière Waterford. Le rapport qui suit décrit un des volets de cette étude: l'utilisation du modèle hydrologique HYMO pour déterminer si l'urbanisation qui s'est produite entre 1973 et 1981 a eu une influence sur les débits causés par des orages de grande intensité. Le modèle HYMO a également été utilisé pour estimer les débits de pointe pour les périodes de récurrence de vingt et cent ans et pour fournir une indication de l'influence sur les débits de pointe que pourrait produire tout nouveau développement urbain.

PREFACE

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

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

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

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

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

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

The study was administered by a Steering Committee appointed by the governments of Newfoundland and Canada. To implement the study plan, a Technical Committee, consisting of two representatives of each government, was established.

Subsequently, the Technical Committee appointed subcommittees and working groups to prepare and carry out the work plans for the various components of the study. The report that follows deals with one such component related to the effects of urbanization on the surface runoff characteristics such as peak flows and peak volumes.

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1.0 INTRODUCTION

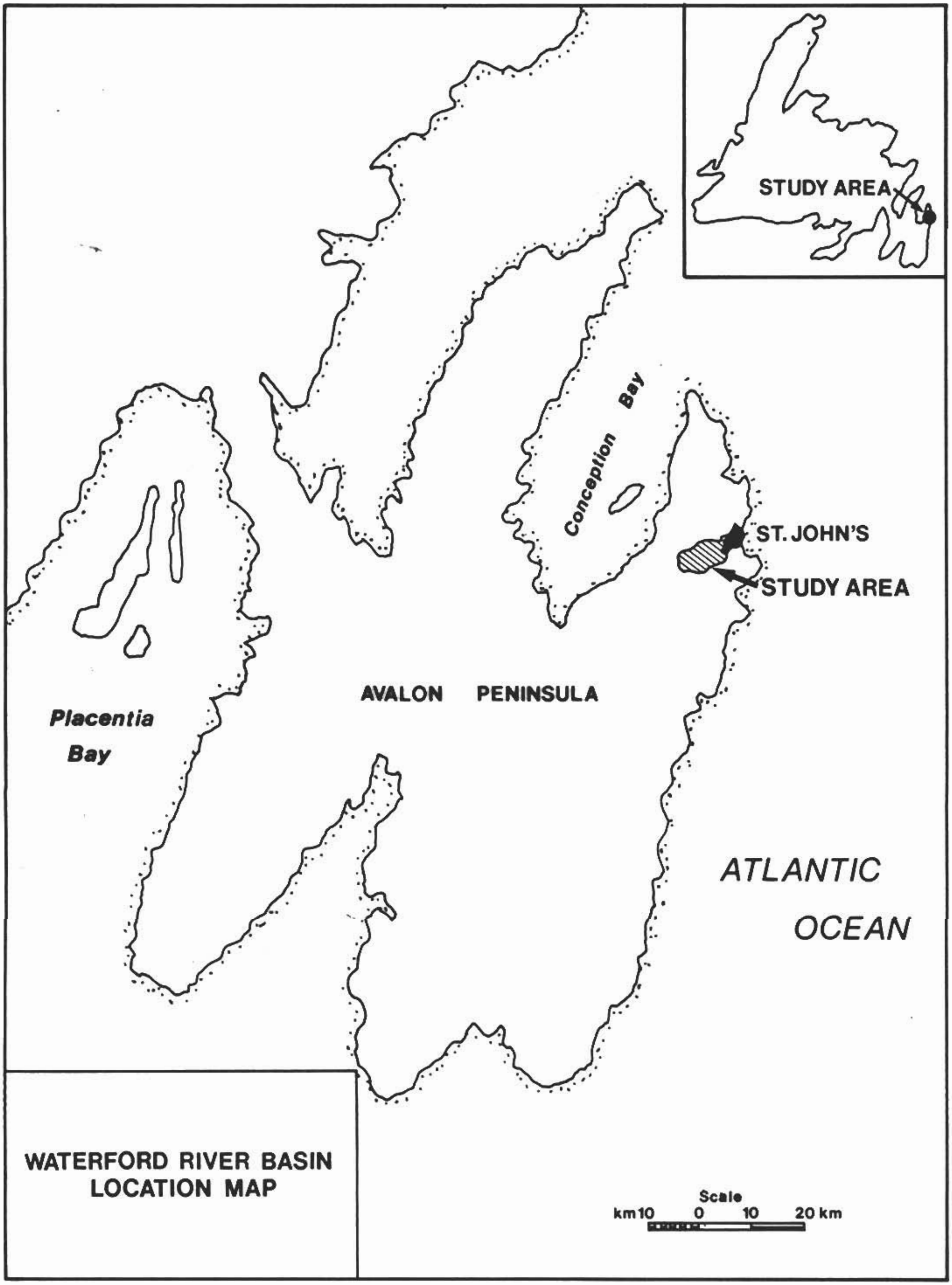
1.1 General

The process of urbanization brings about significant changes in the land use of river basins. The transformation of natural areas into residential, industrial and commercial uses results in a significant and often rapid increase in the amount of impervious area. Such changes modify, in a variety of ways, the hydrologic behaviour of a basin and invariably result in one or more water management problems such as: increased incidences of flooding, deterioration of both surface water and groundwater quality, a decline in groundwater levels and disruption of biotic river environments.

Since the mid 1970's, a wide variety of water management problems experienced in the Waterford River Basin near St. John's, Newfoundland, (Figure 1.1) have been attributed to an accelerated rate of urbanization of the basin.

In 1980, Environment Canada and the Department of Consumer Affairs and Environment of the Province of Newfoundland initiated the Waterford River Basin Urban Hydrology Study. In general, the objectives of the study were: (i) to examine the processes leading to changes in the hydrologic regime of the watershed (including the monitoring of changes); (ii) to utilize that information in a hierarchy of hydrologic and water quality models to evaluate the impacts of urbanization on the water resources of the basin; and (iii) to recommend solutions to specific water management problems for the Waterford River Basin as well as to similar problems facing other watersheds in the Province.

In order to achieve the study objectives, a variety of sub-studies were initiated. These included investigations into storm drainage, flooding, water quality, groundwater, land use and urbanizing



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

Figure 1.1

watersheds. A Working Group on Modelling was established to plan and undertake modelling activities associated with the storm drainage and urbanizing watersheds investigations, and to provide the hydrologic input required for the flood study.

The Working Group on Modelling, which consisted of staff at Environment Canada and the Newfoundland Department of the Environment, defined a number of modelling tasks which were undertaken by agencies of the Inland Waters Directorate of Environment Canada.

An investigation of potentially suitable models was undertaken early in the study by the Newfoundland Department of the Environment (ref. 26). A large number of hydrologic models are available to provide information on spatial and temporal characteristics of urban storm water (including streamflows) both in terms of quantity and quality. Some hydrologic models have been found to be more suitable under certain circumstances whereas other models have proven their superiority under different sets of circumstances. The choice of a suitable model becomes even more difficult in areas where the applicability of these models has yet to be tested. This was the case in the Province of Newfoundland in 1980. The models considered for application in this investigation range from simple to complex in their formulation, and also in their capability to simulate hydrologic response. Some require extensive data bases for their application.

Subsequently, the National Water Research Institute, located in Burlington, Ontario, agreed to apply the ILLUDAS and SWMM models to investigate a small urbanized catchment within the Waterford River Basin. The Water Planning and Management Branch, Dartmouth, Nova Scotia, agreed to apply the HYMO and HSP-F models to investigate the impacts on the overall urbanizing basin of some 53 km^2 as well as for several catchment areas within the basin.

The ILLUDAS[1] and HYMO[3] models are fundamentally single event, deterministic hydrologic models which are applied to simulate the short term response of a catchment to individual storm events. The SWMM[2] and HSP-F[4,5] models, on the other hand, are deterministic models which can be used both to simulate single events as well as to continuously simulate the hydrology of catchments including the extreme cases of floods and droughts.

Studies based on the ILLUDAS, SWMM and HSP-F models are presented in other technical reports of the Waterford River Basin Urban Hydrology Study. This report describes the application of the HYMO model.

1.2 Objectives

The following objectives were established for the investigations involving the use of the HYMO model:

(1) to examine and evaluate the changes in the general characteristics of storm runoff caused by urbanization in the Waterford Basin;

(2) to assess the effects of selected urbanization scenarios on peak streamflow characteristics in the Waterford River; and

(3) to estimate the 20 and 100 year recurrence interval peak flows for three flood prone reaches in the Waterford River.

---

1. ILLUDAS - Illinois Urban Drainage Area Simulator (ref. 23)
2. SWMM - Storm Water Management Model of the U.S. Environmental Protection Agency (ref. 22)
3. HYMO - Hydrologic Model of the U.S. Department of Agriculture (ref. 18)
4. HSP-F - Hydrologic Simulation Program - Fortran, of the U.S. Environmental Protection Agency (ref. 24)
5. The work using the HSP-F model was completed by NWRI

HYMO is a fairly simplistic single event deterministic hydrologic model which is based on the U.S. Soil Conservation Service's rainfall-runoff relationship and unit hydrograph theory. The HYMO model, originally intended for use in rural basins, can be employed to simulate the response of a watershed (even one undergoing urbanization) to a given rainfall and/or snowmelt event, provided the model has been properly calibrated.

In this study, the HYMO model was applied for a number of purposes. It was used to provide an indication of the peak flows which could occur, based on the land use existing in the Waterford River Basin in 1981. The model was also used to provide a general indication of whether or not there had been any significant change in the response of the watershed to snowmelt and/or storm rainfalls since streamflow records began to be maintained for this stream in late 1973. The model was used once again to provide an indication of the change in the characteristics of storm runoff anticipated as a result of future urbanization.

1.3 Scope

During the conduct of this investigation in 1982 and 1983, the HYMO model was calibrated for both dry and wet antecedent moisture conditions. Three events were used for each condition. The accuracy of the calibrated model was verified using one additional event for each antecedent moisture condition. The number of events which could be used for calibration and verification was limited since the events selected had to have occurred in 1981 to 1982 and have resulted in relatively high flow conditions. Most of the data collection network used in the study was not established until 1981. Furthermore, it was considered to be important to use events which occurred during a short period for which land use could be considered to be fairly stable.

The urbanization impact was evaluated using the model calibrated based on 1981 land use in conjunction with precipitation data for two high flow events which occurred during 1974. The simulated hydrographs were then compared with the measured hydrographs.

A design storm was developed and, using the calibrated model, the 20 and 100 year recurrence interval flood flows were simulated. The flows determined in this manner were then compared to the results of a flood frequency analysis of streamflow data recorded on the Waterford River since 1973.

One future development scenario was developed and the corresponding flood flows were simulated.

1.4. Description of the Study Area

The Waterford River Basin, which drains into St. John's Harbour, has a drainage area of approximately 76.7 km^2. The portion of the Basin included in this study is located upstream of the Water Survey of Canada hydrometric station 02ZM008 - Waterford River at Kilbride. The study area, delineated in Figure 1.2, drains an area of 52.7 km^2. Above the hydrometric station, the basin may be considered to be composed of two distinct sub-basins, the main stem of the Waterford River and a major tributary, South Brook. South Brook converges with the Waterford River about one third of a kilometre above the hydrometric station at Kilbride.

The main stem of the river originates at an elevation of about 180 metres in the western region of the basin above Bremigans Pond, a private water supply reservoir. From Bremigans Pond, the river flows in a northeasterly direction for 3.8 kilometres to Donovans where it turns easterly and travels through the Town of Mount Pearl and into the City of St. John's. Two waterfalls, one at Dunn's Bridge (Park St., Mount Pearl)

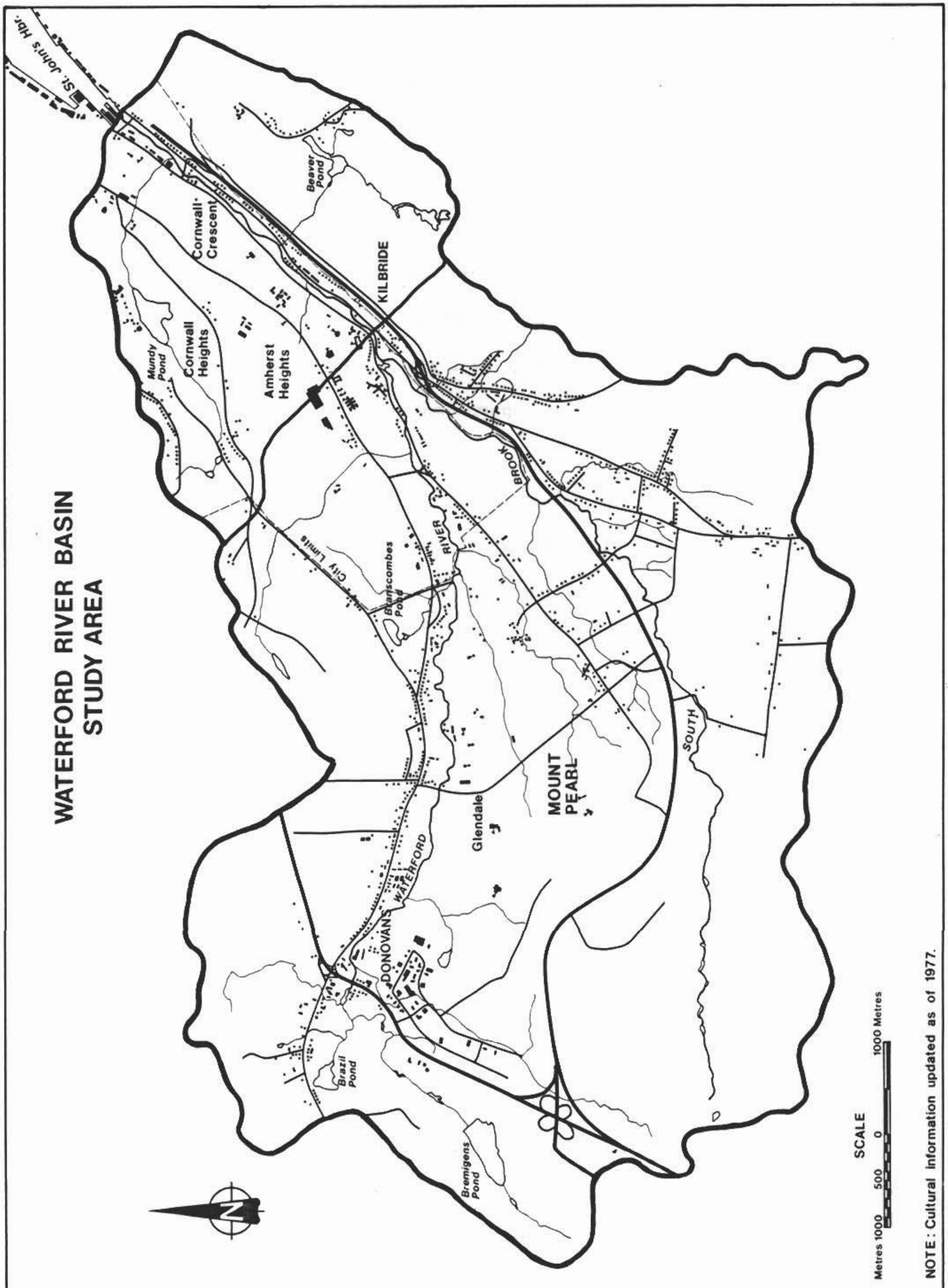

WATERFORD RIVER BASIN
STUDY AREA
St. John's Hbr.
Beaver Pond
KILBRIDE
Cornwall Crescent
Mundy Pond
Cornwall Heights
Amherst Heights
City Limits
Branscombes Pond
WATERFORD
RIVER
BROOK
SOUTH
MOUNT PEARL
Glendale
DONOVANS
Brazil Pond
Bremigens Pond
SCALE
Metres 1000 500 0 1000 Metres
NOTE: Cultural information updated as of 1977.

Figure 1.2

and the other in Bowring Park, are located within this 9.0 km reach of the river.

South Brook originates in the southwestern portion of the basin, in a swampy depression at an elevation of about 190 metres, bordered on each side by hills which rise to a maximum elevation of 240 metres. The Brook flows in an easterly direction through mainly undeveloped forested rural land and slowly curves to the northeast. After a distance of about 10.0 kilometres from the origin, it passes under the Harbour Arterial Highway. Here South Brook descends over the first of two waterfalls, the second waterfall being located approximately one kilometre downstream from the first.

The river channels are generally gravel-lined with some areas being cobble-paved, where bedrock controls the channel. Where the channel bed is movable, gravel bars are formed and are scoured out during high flow events.

2.0 ANALYSIS OF HISTORICAL DATA

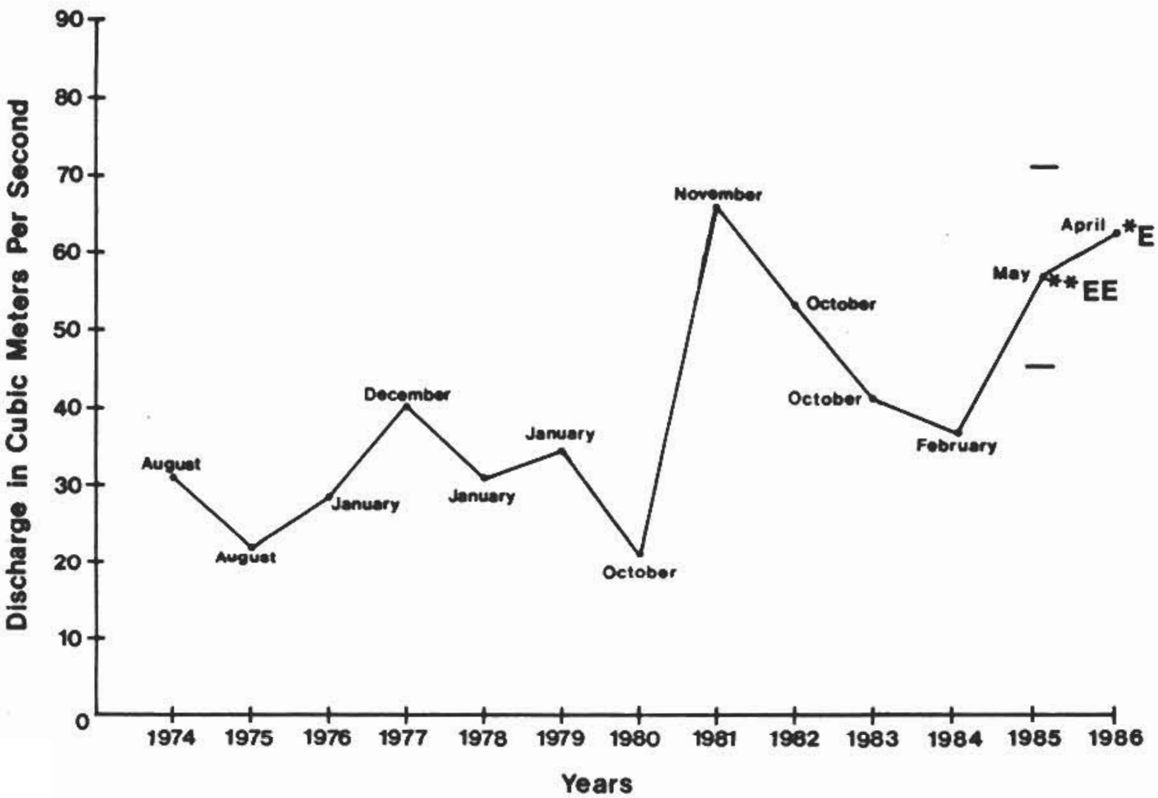
One of the main objectives of this investigation was to examine and evaluate the changes in the general characteristics of storm runoff caused by urbanization in the Waterford River Basin. Prior to beginning the analysis using hydrologic models, an investigation was conducted based on a graphical and tabular analysis of streamflow and climatologic data.

As a first step, annual maximum instantaneous and maximum daily streamflows, recorded at the hydrometric station located at Kilbride (station number 02ZM008), were obtained and plotted (see Figures 2.1 and 2.2). At that time, data were available only for 1974 to 1982. The peaks for 1983 to 1986 became available subsequently. With the exception of the November 1981 and October 1982 instantaneous peaks, no definite trend was evident between the time the hydrometric station began operating in late 1973 and 1982.

Tables 2.1 and 2.2 present detailed climatologic information for the instantaneous and daily peaks from 1974 to 1982. Based on the information contained in these figures and tables, it appears that the difference in annual streamflow peaks is essentially due to differences in the quantity of rainfall and snowmelt as well as antecedent conditions. This is strongly supported by Figure 2.2 and Table 2.2. The fact that the November 1981 and October 1982 instantaneous peak discharges are so high, both in comparison to the instantaneous peaks for previous years, as well as to the corresponding daily mean discharges, can be explained by the extremely short duration of the storms (just several hours).

It cannot, however, be conclusively stated, based on this analysis, that urbanization is not, or might not, be affecting peak flows

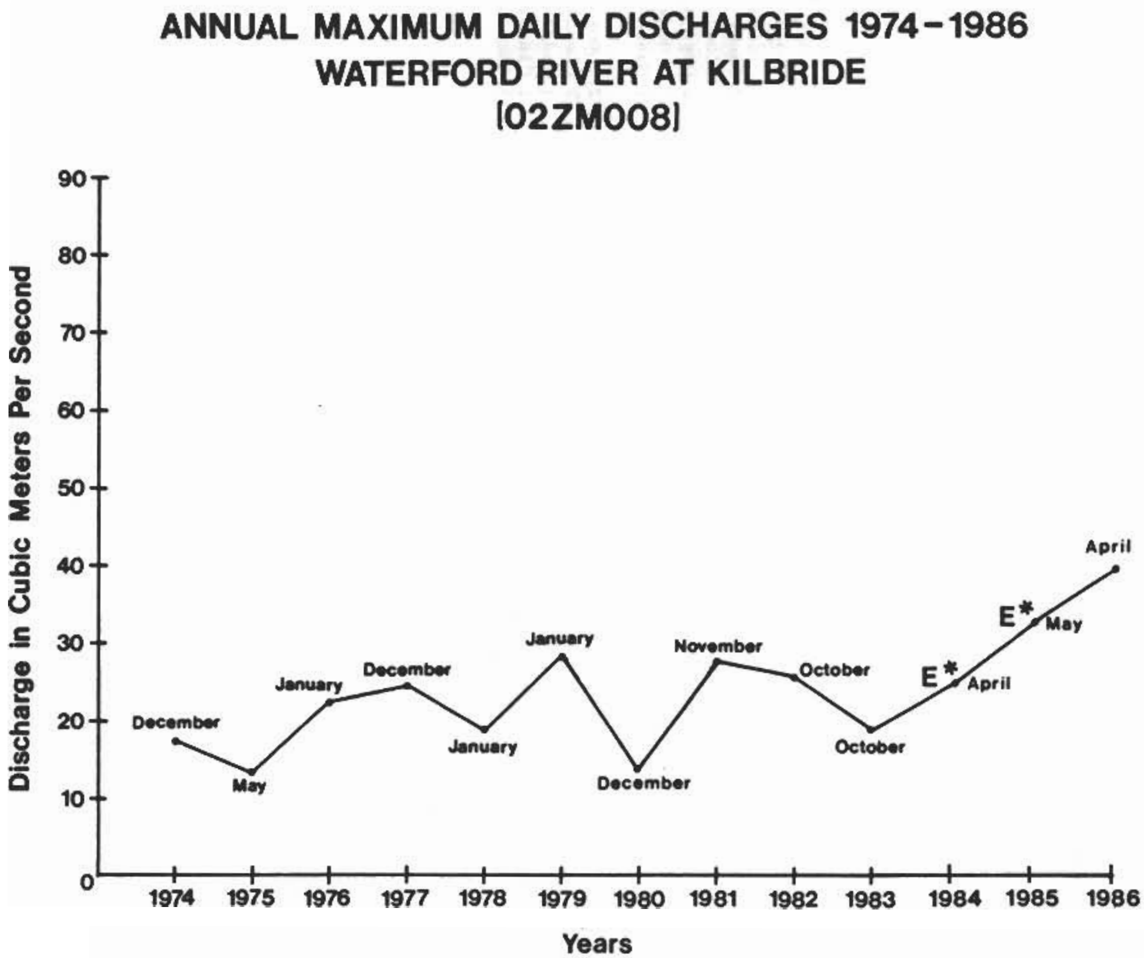
ANNUAL MAXIMUM INSTANTANEOUS DISCHARGES 1974-1986
WATERFORD RIVER AT KILBRIDE
(02ZM008)



* E = Estimated data (provided by Water Survey of Canada)

** EE = Estimated data (not provided by Water Survey of Canada)
Also shown are confidence limits (±1 standard deviation)
for the estimated event based solely on ratios of instantaneous
to daily flows for each year.

Figure 2.1



* E = Estimated data (provided by Water Survey of Canada)

Figure 2.2

TABLE 2.1

ANNUAL MAXIMUM INSTANTANEOUS DISCHARGES 1974–1982

Maximum Instantaneous Discharge (Time) Date	(m^3/s)	Antecedent Rain (mm) 5 day	30 day	Antecedent Snow (cm) 5 day	30 day	No. of Days With Maximum Temperatures Over 20° C 5 day	30 day	Prevailing Wind Direction and Speed (km/h) 5 day	30 day	Hours of Sunshine 5 day	30 day	Sky Condition 5 day	30 day	Event Duration and Rainfall (mm)	
(0316 NST) Aug. 31/74	30.9	4.8	79.8			1	13	W21	W22	20.9	165.7	3 Sunny 1 Cloudy 1 Overcast	16 Sunny 10 Cloudy 4 Overcast	Aug. 29,30 Aug. 31 Total	72.9 None 72.9
(2037 NST) Aug. 23/75	21.8	11.7	109.5			0	18	WSW23	WSW24	28.2	191.9	3 Sunny 1 Cloudy 1 Overcast	19 Sunny 7 Cloudy 4 Overcast	Aug. 21,22 Aug. 23 Total	13.7 85.1 98.8
(1949 NST Oct. 7/80	26.2	38.7	155.2			0	1	WSW22	WSW23	18.0	144.6	2 Sunny 3 Cloudy	17 Sunny 9 Cloudy 4 Overcast	Oct. 6 Oct. 7 Total	30.0 30.4 60.4
(0107 NST) Oct. 4/82	53.4	16.6	201.1			1	10	WSW21	WSW21	19.9	148.1	2 Sunny 1 Cloudy 2 Overcast	17 Sunny 9 Cloudy 4 Overcast	Oct. 2,3 Oct. 4 Total	98.7 0.2 98.9
(1637 NST) Nov. 26/81	66.1	26.4	174.3			0	0	WSW23	WSW23	4.3	65.7	2 Cloudy 3 Overcast	4 Sunny 16 Cloudy 10 Overcast	Nov. 25 Nov. 26 Total	11.8 76.2 88.0
(0521 NST) Jan. 9/76	28.3	7.4 rain and snow	151.1 rain and snow	3.3	43.5	Jan. 7 above freezing. Prior below freezing		SW16	SW16	14.2	61.1	2 Sunny 2 Cloudy 1 Overcast	4 Sunny 14 Cloudy 12 Overcast	Jan. 8 Jan. 9 Total Rain	42.7 20.8 63.5
(1234 NST) Dec. 27/77	40.2	16.7 Total 13.4 (Rain)	191.8 Total 79.3 (Rain)	3.3	105.5	Dec. 22–25 above freezing. Prior below freezing.		W28	W24	2.0	60.2	2 Cloudy 3 Overcast	4 Sunny 15 Cloudy 11 Overcast	Dec. 26 Dec. 27 Total Rain	12.8 29.2 42.0
(1539 NST) Jan. 27/78	30.9	Nil	210.7 Total 147.9 Rain	12.9	62.8	Generally below freezing		WSW27	W28	7.7	72.1	1 Sunny 1 Cloudy 3 Overcast	7 Sunny 9 Cloudy 14 Overcast	Jan. 26 Jan. 27 Total (Rain)	40.6 7.4 48.0
(2232 NST) Jan. 29/79	34.5	55.2 Total (All Rain)	84.4 Total 78.5 (Rain)	Nil	44.0	Jan. 24–26 below freezing Jan 16–23 below freezing		W27	W28	0.0	45.8	5 Overcast	4 Sunny 7 Cloudy 19 Overcast	Jan. 27,28 Jan 29 Total (Rain)	79.4 32.4 111.8

SNOW ON GROUND

1976 Month	Day	Amount	1978 Month	Day	Amount	1979 Month	Day	Amount	1977 Month	Day	Amount
January	3	25 cm	January	21	31 cm	January	22	30 cm	December	21	36 cm
	4	25 cm		22	30 cm		23	36 cm		22	36 cm
	5	28 cm		23	28 cm		24	20 cm		23	30 cm
	6	25 cm		24	20 cm		25	14 cm		24	12 cm
	7	25 cm		25	14 cm		26	8 cm		25	6 cm

TEMPERATURE

Date	Degrees
January 8, 1976	11°C
January 9, 1976	4°C
January 27, 1977	10°C
January 27, 1978	12°C

TABLE 2.2

ANNUAL MAXIMUM DAILY DISCHARGES 1974–1982

Date	Maximum Daily Discharge (m^3/s)	Antecedent Rain (mm) 5 day	Antecedent Rain (mm) 30 day	Antecedent Snow (cm) 5 day	Antecedent Snow (cm) 30 day	No. of Days With Maximum Temperatures Over 20° C 5 day	No. of Days With Maximum Temperatures Over 20° C 30 day	Prevailing Wind Direction and Speed (km/h) 5 day	Prevailing Wind Direction and Speed (km/h) 30 day	Hours of Sunshine 5 day	Hours of Sunshine 30 day	Sky Condition 5 day	Sky Condition 30 day	Event Duration and Rainfall (mm)	
Jan. 9/76	22.6	Total 7.4 (Both Rain and Snow)	Total 176.3	2.9	43.5	Jan. 7 above freezing. Prior below freezing		NW27	WNW25	14.2	16.1	2 Sunny 2 Cloudy 1 Overcast	5 Sunny 13 Cloudy 12 Overcast	Jan. 8 Jan. 9 Total (Rain)	42.7 20.8 63.5
Jan. 27/78	18.7	Nil	210.7 Total (Rain) 147.9	12.9	62.8	Generally below freezing		WSW27	W28	7.7	72.1	1 Sunny 1 Cloudy 3 Overcast	7 Sunny 9 Cloudy 14 Overcast	Jan. 26 Jan. 27 Total (Rain)	40.6 7.4 48.0
Jan. 28/79	28.5	All Rain 55.2	146.5 Total 78.1 (Rain)	Nil	44.0	Jan. 23–26 above freezing. Jan. 16–17 below freezing		W27	W30	0.0	42.7	5 Overcast	4 Sunny 7 Cloudy 19 Overcast	Jan. 27 Jan. 28 Total (Rain)	44.6 34.8 79.4
Dec. 11/74	17.2	15.7 Total 4.6 (Rain)	131.2 Total	13.7	27.4	Dec. 8,9 above freezing, prior 5 out of 6 above freezing		W29	W25	12.5	69.7	1 Sunny 2 Cloudy 2 Overcast	7 sunny 12 Cloudy 11 Overcast	Dec. 9, 10 Dec. 11 Total (Rain)	86.9 Trace 86.9
Dec. 27/77	24.7	16.7 Total 13.4 (Rain)	191.8 Total 79.3 (Rain)	3.3	105.5	Dec. 22–25 above freezing – prior below freezing		W28	W24	2.0	60.2	2 Cloudy 3 Overcast	4 Sunny 15 Cloudy 11 Overcast	Dec. 26 Dec. 27 Total (Rain)	12.8 29.2 42.0
Nov. 26/81	27.8	24.2	174.2					WSW23	WSW23	4.3	65.7	2 Cloudy 3 Overcast	4 Sunny 16 Cloudy 10 Overcast	Nov. 25 Nov. 26 Total	11.8 76.2 88.0
May 7/75	13.2	13.2	58.4			0	0	SW19	NNE25	33.3	131.8	4 Sunny 1 Cloudy	12 Sunny 13 Cloudy 6 Overcast	May 6 May 7, 8 Total	40.1 8.4 48.5
Aug. 17/80*	17.6	30.9	128.6			0	8	WSW19	WSW18	11.7	120.6	1 Sunny 2 Cloudy 2 Overcast	10 Sunny 13 Cloudy 7 Overcast	Aug. 15,16 Aug. 17 Total	63.7 4.4 68.1
Oct. 4/82	25.3	5.0	201.1			1	10	WSW20	WSW20	19.9	148.1	2 Sunny 1 Cloudy 2 Overcast	14 Sunny 10 Cloudy 6 Overcast	Oct. 2,3 Oct. 4 Total	98.7 0.2 98.9

SNOW ON GROUND

Month	1976 Day	Amount	Month	1978 Day	Amount	Month	1979 Day	Amount	Month	1977 Day	Amount	Month	1974 Day	Amount
January	3	25 cm	January	21	31 cm	January	22	30 cm	December	21	36 cm	December	4	0 cm
	4	25 cm		22	30 cm		23	28 cm		22	36 dm		5	0 cm
	5	28 cm		23	28 cm		24	20 cm		23	30 cm		6	5 cm
	6	25 cm		24	20 cm		25	14 cm		24	12 cm		7	7 cm
	7	25 cm		25	14 cm		26	8 cm		25	8 cm		8	4 cm

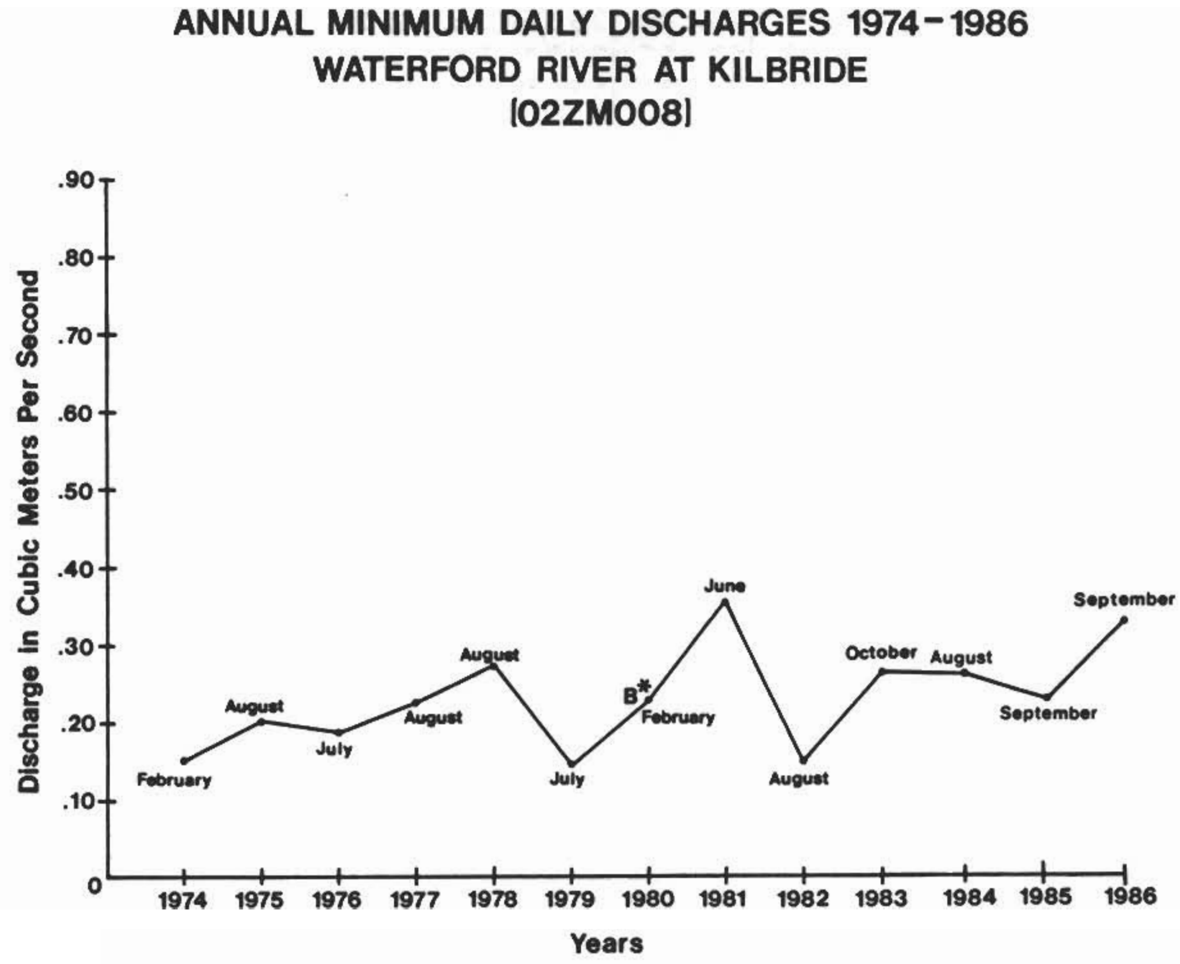
* The Water Survey of Canada subsequently revised data so that the maximum daily discharge for 1980 was just 14.8 m^3/s

on the Waterford River at Kilbride. If it is, however, the rate of change does not appear to be dramatic. With the addition of data for the years 1983 to 1986 to Figures 2.1 and 2.2, and ignoring climatological information contained in Tables 2.1 and 2.2, it could be concluded that there is a gradual trend toward higher peak flows. On the other hand, if peak flows were increasing in the Waterford river Basin, then low flows should be decreasing as a result of reduced infiltration. Reference to Figure 2.3 and Table 2.3, which present annual minimum daily discharges and relevant climatic information, seem to indicate that this is not the case.

The foregoing conclusions must be placed in proper perspective. They focus on the possible change in peak streamflow characteristics for the Waterford River at Kilbride during the period 1973 to 1986. It is important to examine the changes in land use for that period of time.

In 1973, approximately 10 percent of the study was in residential use, compared to 14 percent in 1984 (ref. 7). In 1973, approximately 3 percent of the study area was in commercial, industrial or institutional use compared to 5 percent in 1984. Changes occurred in other land use sectors also. In general, the forested land has decreased (41% down to 33%) as has agricultural land (14% down to 11%). On the other hand, the amount of unproductive land has increased slightly (26% to 29%) as did the residual or "other" category (6% to 8%).

While the changes in land use between 1973 and 1984 for various sectors are certainly measurable, it would, perhaps, be unreasonable to expect changes of such magnitude to have produced an immediately obvious change in the extreme flows, both high and low, of the Waterford River at Kilbride. This is certainly the case if one considers the accuracy with which streamflows and climate can be recorded.



* B = Backwater (probably due to ice) may affect accuracy.

Figure 2.3

TABLE 2.3

ANNUAL MINIMUM DAILY DISCHARGES 1974-1982

Date	Minimum Daily Discharge (m^3/s)	Antecedent Rain (mm)		Antecedent Snow (cm)		No. of Days With Maximum Temperatures Over 20° C		Prevailing Wind Direction and Speed (km/h)		Hours of Sunshine		Sky Condition		Precipitation on Date of Minimum Flow (mm)	
		7 day	30 day	7 day	30 day	7 day	30 day	7 day	30 day	7 day	30 day	7 day	30 day		
Aug. 12/75	0.201	11.9	91.9			5	7	SW20	WSW24	50.2	268.5	4 Sunny 3 Cloudy	25 Sunny 2 Cloudy 3 Overcast	Aug. 12	0.8
Aug. 17/77	0.224	113.3	56.5			4	25	SW22	SW20	44.7	239.3	6 Sunny 1 Cloudy	22 Sunny 7 Cloudy 1 Overcast	Aug. 17	2.4
Aug. 22/78	0.272	4.2	66.2			4	22	WSW18	WSW19	45.8	256.3	4 Sunny 1 Cloudy 2 Overcast	24 Sunny 3 Cloudy 3 Overcast	Aug. 22	9.2
Aug. 18/82	0.150	5.8	49.8			7	19	WSW19	WSW20	66.4	229.9	7 Sunny	23 Sunny 7 Sunny	Aug. 18	16.0
July 24/76	0.187	0.3	60.5			6	13	SW16	SW24	63.9	196.3	5 Sunny 2 Cloudy	17 Sunny 10 Cloudy 3 Overcast	July 24	None
July 28/79	0.145	0.4	110.0			3	14	SW18	SW18	45.8	183.6	4 Sunny 3 Cloudy	16 Sunny 12 Cloudy 2 Overcast	July 28	None
June 6/81	0.356	11.0	46.5			2	4	WSW22	WSW22	46.3	204.4	3 Sunny 2 Cloudy 2 Overcast	21 Sunny 2 Cloudy 7 Overcast	June 6	None
Feb. 17/74	0.150	Total 4.3 (Snow)	Total 127.8 Both Rain and Snow	3.0	90.3	Feb. 14 above freezing - 6 out of 7 temperatures below freezing		W32	NW32	36.2	100.5	4 Sunny 2 Cloudy 1 Overcast	11 Sunny 13 Cloudy 6 Overcast	Feb. 17	23.4
Feb. 15/80	0.2278*	Total 52.0 Snow	Total 142.0 (47.8 Rain)	41.0	78.8	From Feb 1 to Feb. 14 below freezing		W28	W28	16.0	78.9	2 Sunny 2 Cloudy 3 Overcast	5 Sunny 16 Cloudy 9 Overcast	Feb. 15	Trace

SNOW ON GROUND

February 1974 Day	February 1974 Amount	February 1980 Day	February 1980 Amount
10	7 cm	8	19 cm
11	7 cm	9	18 cm
12	7 cm	10	18 cm
13	6 cm	11	18 cm
14	5 cm	12	52 cm
15	5 cm	13	50 cm
16	5 cm	14	47 cm

B* Accuracy of data may be affected due to the existence of an ice cover on the river.

It is not possible, based on the above analysis, to discern how much the existing development in the study area has affected extreme flows. Furthermore, this information cannot be used to make a projection of the changes which can be anticipated. It is also important to point out that the discussion has focussed on the influence of urbanization on extreme flows at one location - the Waterford River at the outlet of the study area. For other locations in the study area, for which the relative land use changes of the associated drainage areas could have been considerably greater, some significant impacts may have occurred. For example, small tributary streams, such as those in the Mount Pearl area where significant change has occurred in residential land use, may have experienced sharp increases in peaks - particularly if storm sewer networks from subdivisions discharge directly into those streams.

3.0 DEVELOPMENT OF THE RUNOFF MODEL OF THE WATERFORD RIVER BASIN

Runoff for significant rainfall events was simulated using the hydrologic model HYMO. The following sections contain a general description of HYMO, a discussion of how HYMO was applied to simulate the runoff for the study areas, discussions of relevant physical information, such as land use and soils types, and the available precipitation and streamflow data.

3.1 THE HYMO MODEL

The HYMO (HYdrologic MOdel) model is a problem oriented computer language for use in modelling the runoff from watersheds. It is designed to transform rainfall into runoff hydrographs and to route these hydrographs through streams and valleys or reservoirs using normally available data.

The HYMO modelling program consists of three major components: the rainfall-runoff relationship, the unit hydrograph and the routing of hydrographs through reaches or reservoirs.

The rainfall-runoff relationship is based on the United States Soil Conservation Service (SCS) rainfall-runoff equation (ref. 10):

$$Q = \frac{(P - 0.2S)^2}{P + 0.8S} \qquad (1)$$

where Q = runoff

P = maximum potential runoff or precipitation

S = the watershed retention (rain that is not converted to runoff)

where $S = \frac{25400}{CN} - 254$ (mm)

CN = SCS runoff curve number

Equation 1 assumes that the initial abstraction (I_a) of the watershed is 20 percent of the watershed retention (S). The initial abstraction consists of infiltration, interception, and surface storage - the latter two of which are assumed to be at a maximum before runoff begins.

The unit hydrograph technique is comprised of three equations for the computation of the hydrograph. From the beginning of the rise to the inflection point of the recession limb, t_o, the hydrograph follows the two parameter gamma distribution relationship:

$$q = q_p \, [t/t_p]^{(n-1)} \, e^{(1-n)(t/t_p - 1)} \qquad (2)$$

where
- q = flow rate at time t
- q_p = peak flow rate
- t_p = time to peak
- n = dimensionless parameter dependent upon K/t_p
- K = recession constant

The dimensionless shape parameter, n, mentioned above is related to the ratio of K and t_p by the following equation (ref. 15):

$$\frac{K}{t_p} = \frac{\left[- \frac{t_o}{t_p}\right]^{n-1}}{(n-1)\left[\left(\frac{t_o}{t_p}\right)^{n-2} - \left(\frac{t_o}{t_p}\right)^{n-1}\right]} \qquad (3)$$

With t_o, the time to the inflection point, defined by:

$$t_o = 1 + \frac{1}{(n-1)} \qquad (4)$$

The peak flow rate is computed by the equation:

$$q_p = \frac{BAQ}{t_p} \tag{5}$$

in which B = a watershed parameter dependent upon n

A = watershed area

Q = volume of runoff

From the inflection point to the point where $t_1 = t_o + 2K$ the hydrograph follows the recession depletion equation:

$$q = q_o e^{\frac{(t_o - t)}{K}} \tag{6}$$

where q_o = flow rate at the inflection point

t_o = time at the inflection point

K = recession constant

The remaining portion of the hydrograph follows the recession depletion equation:

$$q = q_1 e^{\frac{(t_1 - t)}{K_1}} \tag{7}$$

where q_1 = flow rate at t_1

$K_1 = 3K$ = a second recession constant

The two parameters required for the solution of these equations are t_p and K. The HYMO manual (ref. 17) provides empirical equations for the estimation of these parameters. These equations, which are based on data from rural watersheds with drainage areas varying from 0.5 to 25 square miles in the southern United States, have been found by many

Canadian users to be non-representative of their region (ref. 12). Appropriate estimates of these parameters can only be obtained by calibration to several rainfall/runoff events, provided suitable rainfall and streamflow data are available.

The rainfall that is included in the retention storage (S), including the initial abstraction (assumed to be equal to 0.2S), disappears as far as the model is concerned. Furthermore, baseflow is assumed negligible compared to the peak discharges. Therefore, it is assumed in the model that baseflow does not affect the event hydrograph. This assumption is valid when simulating very large peaks. Baseflow can, however, be estimated and added onto the HYMO output.

Reservoir routing in HYMO is performed by the well known, storage indication method. An outflow discharge versus reservoir storage relationship is input to the model. This method works well for most overflow structure reservoir outlets but becomes increasingly complicated as variable opening, gated structures are encountered since separate curves are then required for each gate configuration.

River routing is performed by the Variable Storage Coefficient (VSC) method, which accounts for the change in the water surface slope in a reach as the discharge changes. This method requires a representative stage-discharge relationship for each routing reach. These may be input directly from flow measurements or they may be estimated by using surveyed stream cross-sections and the slopes of the channel and flood plain. More details of the method used are available elsewhere (ref. 17 and 13).

3.2 Discretization of the Study Area

The study objectives (refer to Section 1.2) dictated the level of discretization of the basin for use in model development. For example, to determine the overall impact that development has had on storm runoff peak flows and volumes on the Waterford River, the entire study area could have been considered, at least initially, as one hydrologic unit (or perhaps two, to more appropriately consider South Brook). The only location in the study area for which streamflow data are available, prior to the time of the study, is at the outlet.

To provide estimates of the 20 and 100 year flood flows for use in the flooding component, a minimum of 3 or 4 hydrologic units would have been required - one for each of the three flood reaches along the main stem, and perhaps another to represent the response of South Brook.

The objective which ultimately controlled the level of discretization selected was that of assessing the effects of selected urbanization scenarios on peak streamflow characteristics. In order to make use of the maximum resolution offered by HYMO, it was decided to set the size of the hydrologic units at approximately the minimum size (1 square mile or 2.6 km^2) recommended by Williams (ref. 18). As previously mentioned, the equations recommended by Williams for the determination of the unit hydrograph parameters K and t_p were derived using data for basins with drainage areas ranging from 0.5 to 25 square miles (1.3 to 65 km^2).

The discretization of the study area into sub-basins, or hydrologic units, was established through examination and interpretation of available 1:1,250 and 1:2,500 scale contour maps coupled with air photo interpretation and some field checking. The sub-basin boundaries were transferred to a 1:12,500 scale base map and the drainage area, channel length (where applicable) and elevation changes were measured for each hydrologic unit (HU).

A schematic showing the model configuration adopted for the study is shown in Figure 3.1. The detailed discretization of the study

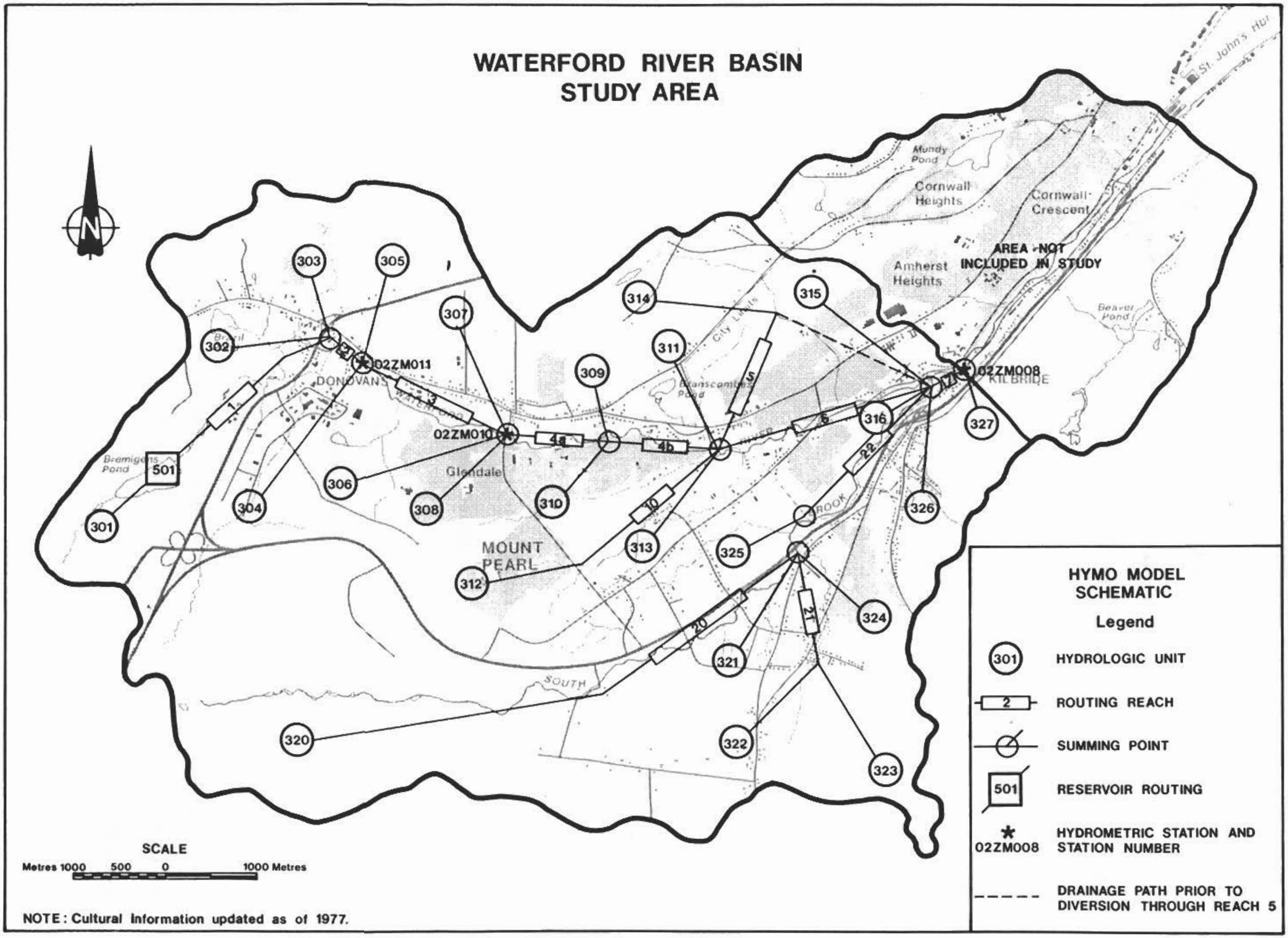

WATERFORD RIVER BASIN
STUDY AREA
Mundy Pond
Cornwall Heights
Cornwall Crescent
Amherst Heights
AREA NOT INCLUDED IN STUDY
Beaver Pond
St. John's Hbr
KILBRIDE
02ZM008
02ZM011
02ZM010
DONOVANS
Glendale
MOUNT PEARL
Bremigens Pond
City Limits
Branscombe Pond
SOUTH
BROOK
301
302
303
304
305
306
307
308
309
310
311
312
313
314
315
316
320
321
322
323
324
325
326
327
501
1
2
3
4a
4b
5
6
10
17
20
21
22
HYMO MODEL SCHEMATIC
Legend
301 HYDROLOGIC UNIT
2 ROUTING REACH
SUMMING POINT
501 RESERVOIR ROUTING
02ZM008 HYDROMETRIC STATION AND STATION NUMBER
DRAINAGE PATH PRIOR TO DIVERSION THROUGH REACH 5
SCALE
Metres 1000 500 0 1000 Metres
NOTE: Cultural Information updated as of 1977.
N

Figure 3.1

portion of the basin is presented in a large fold-out map in Appendix K (Map K.1) located in Volume 2.

The mainstem watershed was subdivided into 16 HUs. Furthermore, in order to simulate routing along the Waterford River between the points where runoff from each of the HUs was added, seven routing reaches were established.

The HU at the uppermost reach of the main stem discharges through a water supply reservoir called Bremigans Pond. Flows were routed through that reservoir using the theoretical equation for a broad-crested weir (ref. 3):

$$Q = \frac{2}{3} B (C_d) (2g)^{\frac{1}{2}} H_o^{\frac{3}{2}} \tag{8}$$

where Q = discharge over weir

B = width of the weir

H_o = head above the weir crest

C_d = a dimensionless discharge coefficient, selected as 0.62

g = gravitational acceleration constant

The South Brook watershed was subdivided into 8 HUs and 3 routing reaches were established.

Two additional routing reaches were established to represent a small tributary stream and a drainage diversion.

3.3 Land Use Characteristics

The amount of direct runoff resulting from a rainfall event is calculated in HYMO using the rainfall runoff relationship presented in Section 3.1 as Equation 1. Fundamental to that computation is the specification of CN, the SCS runoff curve number. The maximum CN of 100 implies that all precipitation falling on the basin contributes to direct runoff. The curve number is estimated based on consideration of land use, soil type and thickness, and antecedent moisture conditions.

Land use is a major factor affecting the runoff characteristics of watersheds. Much of the land use information for the study area was obtained from air photographs obtained in August 1981 by the Newfoundland and Labrador Department of Forest Resources and Lands and printed at a scale of 1:7000.

The Lands Directorate of Environment Canada used these photos to delineate and type the various land use/impermeability categories. These data were then transferred to a 1:12,500 scale base map using an Artograph 1000 opaque transfer projector.

Land use/impermeability classifications were established to correspond closely to the classifications contained in the United States Soil Conservation Service (SCS) National Engineering Handbook, Hydrology, Section 4 (ref. 10) (Tables 9.1 to 9.5 and Charts C2-2 to C2-4). Those classifications, presented in Appendix A, were then used to classify the various land uses in the study area existing in 1981. The process was repeated using photography for 1973, the year streamflow records began at Kilbride. The resulting land use map for 1981 is portrayed in Appendix K (Map K2). Detailed comparisons between land use in the study area in 1973, 1981 and 1984 are contained in the report of the land use component of this study (ref. 7).

3.4 <u>Soil Types</u>

Soil type and thickness have a major bearing on the hydrologic response of a watershed. Soils which are relatively impermeable or have a high water table, or consist of a thin layer over less permeable material (e.g. bedrock) do not permit a significant amount of infiltration. Consequently, the proportion of rain or snowmelt available to contribute to direct runoff is high. Conversely, thick soils for which significant infiltration can occur rapidly will result in a lesser amount of direct runoff. Baseflows for the latter types of soil are consequently higher than for the former.

The soils in the environ of the Waterford River Basin have developed from materials derived from the underlying slate-siltstone, sandstone, conglomerate, granitic and volcanic rocks. The entire region was glaciated and the materials were deposited in the form of ground moraine, outwash and other glaciofluvial deposits. Some streams in the basin have deposited alluvial sediments along their courses and, where the bed is movable, these migrate mainly as gravel bars during periods of high runoff (ref. 2).

The mineral soils are, for the most part, coarse to moderately coarse textured, strongly acidic to extremely acidic, and low in natural fertility. These soils belong mainly to the Humo-Ferric and Ferro-Humic Podzol great group. Gleysolic soils with dull colored, mottled profiles and organic soils are found in poorly drained areas.

The soils are composed of six main types; Bauline, Cochrane, Organic, Pouch Cove, Red Cove and Torbay. Appendix B1 contains a brief description of each of these. There are often mixtures or complexes of these six main types (ref. 1).

A 1:100,000 scale map accompanying the Newfoundland Soil Survey Report (ref. 1) was used to classify the soils of the study area into the United States Soil Conservation Service's (SCS) hydrologic soil groups. The descriptions of the SCS hydrologic soil groups are reproduced in Appendix B2. The SCS classification can be subdivided whenever such refinement is justified. For the purpose of this study it was decided that this refinement was not necessary for the main soil types. For soil complexes, however, a simple interpolation was employed.

Appendix B3 presents relevant soil symbols from the soil report, gives a brief description of hydrologic features of the soil and assigns a SCS hydrologic soil group classification. The symbol convention used in Appendix B3 is described in Appendix B4.

To facilitate transfer of the information from the published 1:100,000 scale soils map to the 1:12,500 scale base map, a portion of the soils map corresponding to the area of the Waterford River Basin was photographically enlarged to a scale of 1:12,500. The soil classifications were converted into the SCS classifications and the soils and land use maps were then overlaid to produce the large land use/soil SCS classification map presented in Appendix K (Map K2). This map was then used to determine the land use/soil complex required for estimating the runoff coefficient for each HU of the HYMO model.

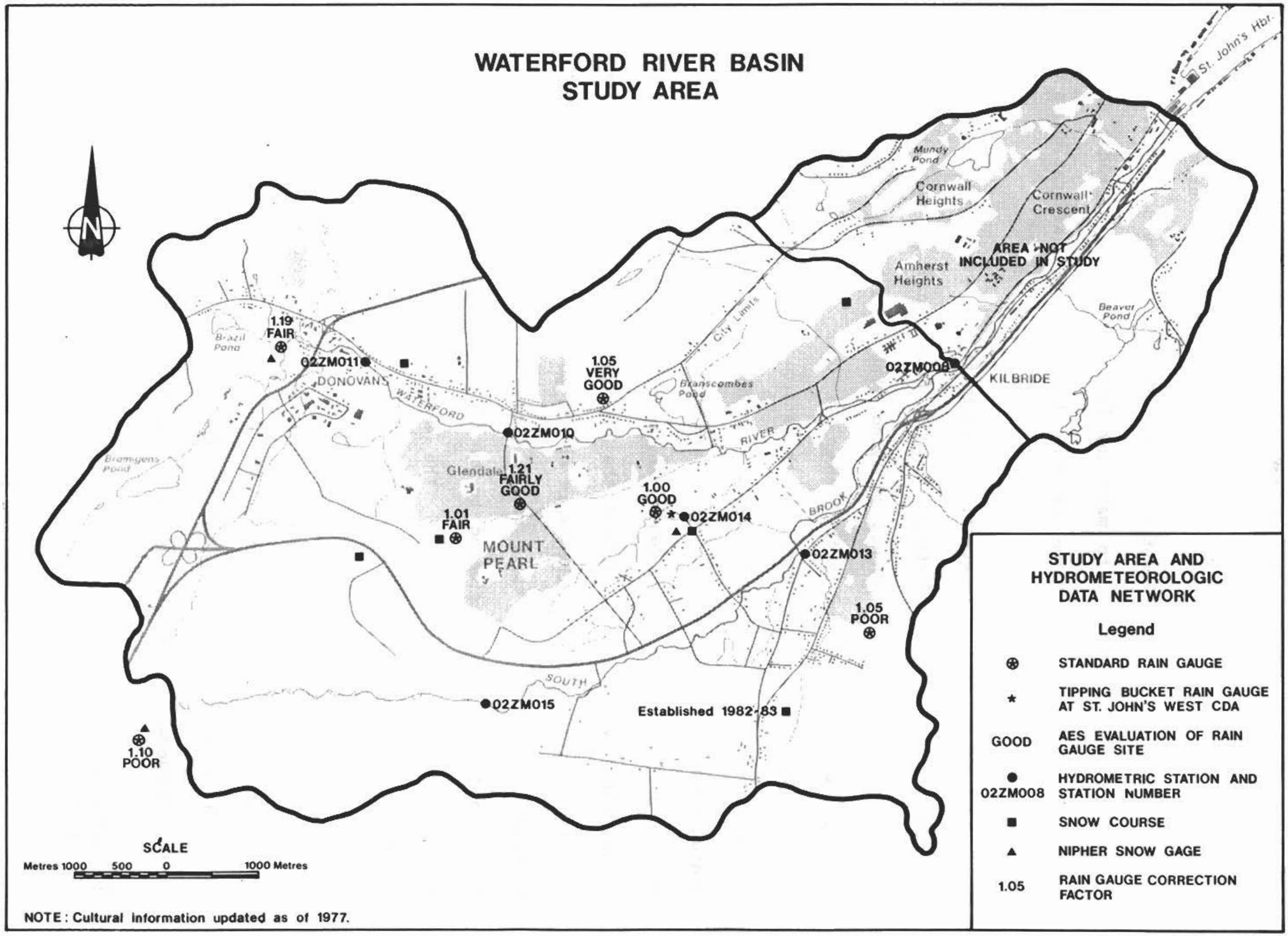
3.5 Precipitation Data

Precipitation and streamflow data were required for a number of recent rainfall events in order to calibrate and verify the watershed model to present land use conditions. It was also considered desirable (if not essential) that some precipitation and streamflow data exist for several rainfall events which occurred as far in the past as possible. Ideally, that would be for a time when the amount of development was significantly less than at present. It was felt that use of the model, calibrated to present conditions, in association with the rainfall data collected during a time when less development existed, would possibly provide an indication of the effects of urbanization on storm runoff peaks and volumes.

Prior to the study, precipitation was measured within the study area (on a daily basis) at the Agriculture Canada (CDA) Experimental Farm in St. John's West. Data have been collected there since 1954. During the course of this investigation, the Newfoundland Department of the Environment installed a number of standard rain gauges in or just outside the study area and the Atmospheric Environment Service, Environment Canada, added a continuously recording, tipping bucket gauge to the St. John's West CDA site. This network was designed to provide a continuous hyetograph coupled with the spatial variation in depth of rain over the basin. Unfortunately, it was not always possible to locate people willing to take rainfall readings over an extended period of time in the less populated sections of the basin. This left the higher topographic areas of the basin, particularly the southwest portion, essentially ungauged. Figure 3.2 shows the location of the rain gauges from which it was possible to use information for this study.

The precipitation measured by the tipping bucket recording gauge at the Saint John's West CDA station was used to determine the temporal distribution of rainfall over the basin. The standard rain gauge at the CDA station was used to adjust the hyetograph volume. This was required since tipping bucket gauges do not provide accurate total precipitation data due to undercatch problems - which are greatest during heavy rainfall events.

In order to investigate possible variation in temporal distribution of rainfall over the basin, rainfall intensities obtained from the Environment Canada radar located at Trepassey were employed. For this test, the watershed and nearby surrounding area were divided into four quadrants. For each quadrant, the rainfall for each radar



WATERFORD RIVER BASIN
STUDY AREA
N
St. John's Hbr.
Mundy Pond
Cornwall Heights
Cornwall Crescent
Amherst Heights
AREA NOT INCLUDED IN STUDY
Beaver Pond
KILBRIDE
02ZM008
City Limits
Branscombes Pond
Brazil Pond
1.19 FAIR
02ZM011
DONOVANS
WATERFORD
RIVER
1.05 VERY GOOD
02ZM010
Bremigens Pond
Glendale
1.21 FAIRLY GOOD
1.01 FAIR
MOUNT PEARL
1.00 GOOD
02ZM014
BROOK
02ZM013
1.05 POOR
SOUTH
02ZM015
Established 1982-83
1.10 POOR
SCALE
Metres 1000 500 0 1000 Metres
NOTE: Cultural Information updated as of 1977.
STUDY AREA AND HYDROMETEOROLOGIC DATA NETWORK
Legend
STANDARD RAIN GAUGE
TIPPING BUCKET RAIN GAUGE AT ST. JOHN'S WEST CDA
GOOD AES EVALUATION OF RAIN GAUGE SITE
02ZM008 HYDROMETRIC STATION AND STATION NUMBER
SNOW COURSE
NIPHER SNOW GAGE
1.05 RAIN GAUGE CORRECTION FACTOR

Figure 3.2

grid[1] was accumulated and averaged and the four resulting hyetographs were plotted using a ten minute time increment. The hyetographs were then compared to see if there was a variation in timing between each quadrant. Since no significant variation could be observed for several events, it was decided not to make any further use of the radar data unless timing problems became evident during model calibration and verification.

As a matter of interest, the estimates of precipitation rates obtained from the radar differed greatly from those provided by the ground network. For the events analyzed, the radar totals were just one-third of those provided by the gauges.

Most (80 to 90 percent) of the storm rainfall data recorded at six of the standard rain gauges were less than those recorded at the CDA standard gauge. This was not consistent with topographic considerations. For this reason, the Atmospheric Environment Service of Environment Canada (AES) was asked to inspect the precipitation gauge sites in the Waterford River Basin. On the basis of the potential for undercatch due to vegetation, buildings, etc., two of the gauge sites were evaluated as being poor, two were considered to be fair, one was considered to be fairly good and one very good. The evaluations of each station are presented in Figure 3.2. It is difficult to establish an ideal rain gauge network in an urban area.

As a result of the apparent undercatch problem and considering the topographic characteristics of the basin, it was decided that the storm rainfall amounts recorded at the standard gauges should be adjusted to be, on average, not less than those recorded at the CDA standard gauge.

---

1 Rainfall intensities are provided by radar with a resolution (grid) of 2km by 2km.

Accordingly, twelve significant rainfall events were selected and "correction" factors determined. Significant events only were selected so that the factors would be compatible with the magnitudes of rainfall for the events chosen for calibration, verification and application of the HYMO model. The resulting "correction" factors are displayed on Figure 3.2 for each temporary precipitation gauge.

An alternative approach to the technique discussed in the previous paragraph would have been to disregard the data obtained from all precipitation gauges. This was not done, however, as it was decided that the selected procedure allowed more appropriate representation of the spatial variation in total precipitation for each event.

Precipitation (in this case rainfall) data must be input to HYMO in a lumped manner for each HU. Since the temporal distribution was to be provided by the tipping gauge at CDA, it was decided to facilitate entry to the computer by developing a series of rainfall factors for each HU for each event used to calibrate and verify the model. In this way, only the hyetograph for the CDA, with a time increment of 10 minutes, had to be input in addition to a rainfall factor applicable to each HU.

The procedure used to develop the rainfall factors is discussed in Appendix H. It is relevant to state here that the Thiessen polygon approach (ref. 11) was employed to estimate the quantity of rainfall for each HU. Furthermore, the rainfall factors were determined so as to account for the apparent undercatch discussed previously.

Use of the Thiessen polygon technique facilitates data entry to hydrologic models when a number of rainfall events are to be considered. Since this technique assumes that the rainfall recorded at a gauge is representative of the amount which has fallen throughout the associated polygon, it is important that the network be designed so as to adequately represent various aspects (largely topographic) affecting rainfall. As stated previously, there are seven locations within the study area for which rainfall data were collected during the conduct of the study. This is a reasonably good number considering the size of the basin. As stated previously, however, it is possible that the network did not adequately represent the higher topographic areas, located at the drainage basin boundaries.

Snowfall and snow accumulation were also monitored. Two Nipher snow gauges were established and read daily. In addition, five, five-point snow courses were established and sampled on a weekly basis during the winter of 1981-1982. The network was increased to six snow courses during the winter of 1982-1983. The snow data collection network is also displayed on Figure 3.2. As it turned out, no major snowmelt runoff events occurred during the course of this study. These data were useful, however, for the continuous simulation modelling undertaken with the HSP-F model.

3.6 Streamflow Data

The Water Survey of Canada has operated a hydrometric station on the Waterford River at Kilbride (station number 02ZM008) since late 1973. This station is located at the downstream boundary of the study area (see Figure 3.2). The drainage area to the gauge at Kilbride is 52.7 km^2.

In 1981, two additional stations were established. One of these is located at Mount Pearl (station number 02ZM010), downstream of the Commonwealth Road Bridge (drainage area of 16.6 km^2). The other station (02ZM011), which has a drainage area of 11.4 km^2, is located downstream of the bridge entering the property of Newfoundland Fibrply near Donovans Industrial Park. These two gauges were established in order to assist in monitoring the effects of urbanization on the hydrologic regime for portions of the basin with differing degrees and types of land use.

In addition, several relationships between water level (stage) and streamflow (discharge) were established for three locations in the basin where discharges were required for water quality sampling and for which it was thought that flow data might be useful in the calibration of models. The locations of these sites are also shown on Figure 3.2.

Discharge records were provided by the Water Survey of Canada on a 15 minute time interval for those events that were identified for use in calibration and verification of the HYMO-based model.

3.7 Data Required for Streamflow Routing

The flood routing procedure used in HYMO is based on a storage flood routing method. The Variable Storage Coefficient (VSC) method (ref. 14) takes into consideration the variation in water surface slope and, as such, considers the change in slope, and consequently change in time of travel with stage, as the flood hydrograph flows through the reach. The basic input data required for this procedure consists of valley cross-sections, channel and floodplain slopes and estimates of the Manning's coefficient of roughness for the channel and banks.

A total of 28 valley cross-sections were determined from existing contour maps and field surveys and Manning's 'n' values were estimated in the range of 0.015 to 0.100 for the banks and 0.025 to 0.06 for the channel. These are presented in Table 3.1. Appendix I contains the cross-section information in the format provided to the HYMO program.

The HYMO program computes rating curves based on cross-sections and Manning's 'n' values unless a measured or computed rating curve (water level versus discharge relationship) is provided to it.

The VSC method computes the outflow for each reach as a function of the inflow, the outflow at the previous time step and some storage coefficients dependent on the travel time through the reach as well as the computational time interval selected. The travel time can be computed as a function of reach length, velocity, slope, and depth, as well as inflow and outflow from equations provided by Williams and Hann (ref. 17).

For this study twelve routing reaches were used. These are shown on the model schematic presented in Figure 3.1.

The Waterford River and South Brook contain four significant major waterfalls. It was believed that the inclusion of these in the reach slope would bias the slope of the routing model and, after concurrence from Mr. Williams, it was decided to subtract the height of the waterfalls when determining the slopes of the affected reaches.

TABLE 3.1
MANNING'S 'n' FOR CALIBRATED MODEL

Cross Section Number	Manning's 'n' Left Overbank	Channel	Right Overbank
1.1	0.060	0.045	0.060
1.2	0.035	0.035	0.035
2.1	0.050	0.035	0.045
2.2	0.050	0.035	0.045
2.3	0.025	0.030	0.025
3.2	0.060	0.045	0.100
3.3	0.065	0.060	0.060
4.1*	0.070	0.040	0.070
4.2*	0.100	0.040	0.070
4.3**	0.050	0.040	0.050
4.4**	0.050	0.040	0.050
5.1	0.025	0.025	0.025
5.2	0.050	0.030	0.050
6.2	0.070	0.050	0.070
6.3	0.070	0.050	0.070
7.1	0.015	0.040	0.070
7.2	0.070	0.040	0.020
10.1	0.070	0.050	0.070
10.2	0.070	0.050	0.070
10.3	0.070	0.050	0.070
20.2	0.050	0.035	0.050
20.3	0.050	0.040	0.050
20.4	0.050	0.035	0.050
21.1	0.040	0.030	0.040
21.2	0.015	0.030	0.015
22.1	0.050	0.030	0.040
22.2	0.050	0.045	0.040
22.3	0.050	0.040	0.050

* Reach 4A

** Reach 4B

4.0 CALIBRATION OF THE RUNOFF MODEL

Before the runoff model could be applied to address the objectives stated in Section 1.2, it had to be calibrated and verified. Since the objectives pertain to peak flows, the model's parameters were calibrated using data recorded during high streamflow events which occurred since the full streamflow and precipitation network was established (1981). The events selected all occurred between June 1981 to November 1981. For such a short period of time, it is reasonable to expect that changes in the amount of urbanization would be minimal. Hence the impact on the model parameters would be minimal.

The following sections describe the initial selection of the model parameters, the calibration process, the verification procedure (based on additional high flow events not used in the calibration process) and an analysis to test the sensitivity of the simulated streamflows to reasonable changes in the parameter values.

4.1 Initial Estimates of Major Hydrologic Parameters

In order to simulate the response of a specific watershed to a rainfall event using the HYMO procedures, three hydrologic input parameters are required. One parameter, a runoff index factor combining the hydrologic soil group and land use characteristics, is known as the hydrologic soil cover complex number (CN). Two unit hydrograph parameters are also required. These are the time to peak (t_p) and the recession constant (K). A description of the procedure used to estimate those parameters is included in the following two sections.

4.1.1 Determination of CN

The amount of direct runoff resulting from a storm rainfall event is a function of many factors including land use, soil characteristics and antecedent moisture conditions. In the HYMO model, the combined influence of these factors is considered to be spatially uniform for a given hydrologic unit. HYMO uses the United States Soil Conservation Service's (SCS) curve number (CN) as an indicator of runoff potential.

The SCS National Engineering Handbook(ref. 10) contains tables of CN as a function of land use and SCS hydrologic soils group (discussed in Sections 3.3 and 3.4) for average antecedent moisture conditions (AMC II). Those tables were used as guides to establish initial estimates of CN for the various land use/impermeability categories adopted for this study. A table of CN values (as a function of land use and soils type) applied initially in this study are presented in Table C.1 of Appendix C.

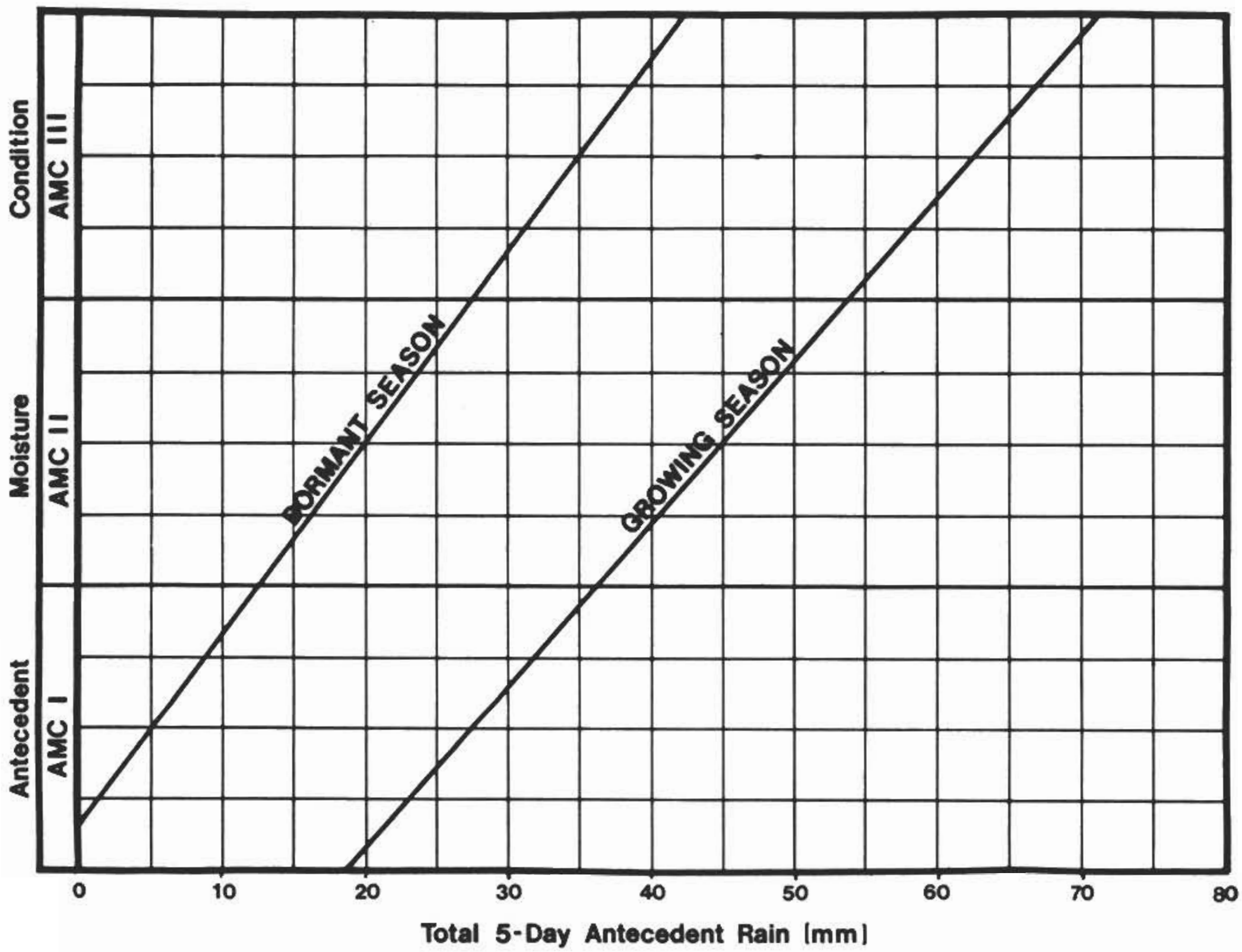
The SCS defines three antecedent moisture conditions. These are:

AMC I: Lowest runoff potential. A condition of watershed soils where the soils are dry but not to the wilting point, and when satisfactory plowing or cultivation takes place.

AMC II: The average case for annual floods. That is an average of the conditions that have preceded the occurence of the maximum annual flood on numerous watersheds.

AMC III: Highest runoff potential. If heavy rainfall or light rainfall and low temperatures have occurred during the five days previous to the given storm and the soil is nearly saturated.

To determine the antecedent moisture condition applicable to a given event, the SCS has suggested the chart of seasonal rainfall limits as shown in Figure 4.1. This chart has been used for the Waterford River Basin, keeping in mind that it was developed for different climatic conditions than for the Avalon Peninsula. For Newfoundland, evapotranspiration is controlled, not as much by air temperature, but by the moisture content of the incoming air. Figure 4.1, however, has been



DEFINITION OF

ANTECEDENT MOISTURE CONDITION

(ref. 10)

Figure 4.1

found to be adequate in other regions in Canada for obtaining a preliminary estimate of CN. These estimates of CN are then adjusted as required during the calibration process. This chart also considers two season categories: the growing season and the dormant season. The growing season, according to the SCS, applies when the soils are not frozen and there is no snow on the ground. This period for the Avalon Peninsula was considered to be from the beginning of May to the end of October. The remainder of the year was considered to be the dormant season.

Using the map (K2) consisting of the combined land use, the hydrologic soil grouping and hydrologic unit maps, an initial average hydrologic soil cover complex number was determined for AMC II for each hydrologic unit. Those numbers (often referred to as "curve numbers") were then adjusted to account for varying antecedent moisture conditions, using Table C.2 (Appendix C). The resulting initial estimates of CN for AMC I, AMC II and AMC III for each hydrologic unit are presented in Table 4.1.

4.1.2 Determination of K and t_p

The HYMO program provides empirical equations for the estimation of the parameters K (recession constant) and t_p (time to peak), which were discussed in Section 3.1.

The original version of HYMO as published by Williams and Hann (ref. 17) uses the equations:

$$K = 27.0\ A^{0.231}\ SLP^{-0.777}\ (L/W)^{0.124} \qquad (7)$$

and

$$t_p = 4.63\ A^{0.422}\ SLP^{-0.460}\ (L/W)^{0.133} \qquad (8)$$

TABLE 4.1

PRE-CALIBRATION HYDROLOGIC PARAMETERS

Hydrologic Unit	CN AMC I	CN AMC II	CN AMC III	t_p (hours)	K (hours)
301	50	66	79	0.852	1.661
302	54	70	84	0.570	0.712
303	58	73	84	0.479	0.599
304	57	70	84	0.700	0.858
305	61	77	89	0.349	0.445
306	57	73	86	0.354	0.409
307	60	77	89	0.381	0.413
308	61	78	89	0.363	0.499
309	58	74	84	0.219	0.188
310	63	79	90	0.514	0.649
311	56	74	87	0.240	0.204
312	60	76	87	0.648	0.892
313	58	75	86	0.413	0.589
314	55	73	86	0.364	0.409
315	62	78	89	0.361	0.331
316	60	76	86	0.468	0.799
320	51	67	81	0.959	1.226
321	57	74	87	0.952	1.187
322	62	79	90	0.543	0.970
323	58	75	88	0.480	0.554
324	60	77	89	0.491	0.777
325	54	72	86	0.515	0.723
326	55	73	86	0.413	0.377
327	49	69	84	0.120	0.121

where A = area of hydrologic unit in square miles

SLP = the difference in elevation in feet, divided by the flood plain distance in miles, between the watershed outlet and the most distant point on the watershed,

L/W = the watershed length (L) to width (W) ratio (L^2/A as used in HYMO)

K = recession constant in hours

t_p = time to peak in hours

Alternate equations have been established for basins with slopes greater than two percent (ref. 17). These are:

$$K = 16.1\ A^{0.24}\ SLP^{-0.84} \tag{9}$$

$$t_p = 6.54\ A^{0.39}\ SLP^{-0.50} \tag{10}$$

The following metric version of those equations were used for this study:

$$K = 5.95\ A^{0.231}\ SLP^{-0.777}\ (L/W)^{0.124} \tag{11}$$

$$t_p = 1.44\ A^{0.422}\ SLP^{-0.46}\ (L/W)^{0.133} \tag{12}$$

and for basin slopes greater than two percent:

$$K = 3.17\ A^{0.24}\ SLP^{-0.84} \tag{13}$$

$$t_p = 1.12\ A^{0.39}\ SLP^{-0.50} \tag{14}$$

where the difference in elevation is in metres, the flood plain distance is in kilometres and the area is in square kilometres.

The initial values for K and t_p for each hydrologic unit are shown in Table 4.1.

4.2 Calibration and Verification of the Watershed Model

The watershed model was calibrated and verified during 1982, based on rainfall and discharge data collected for the most severe storm events (i.e. the ones with the highest streamflows) which occurred during mid to late 1981. Data for two antecedent moisture conditions (SCS AMC I and AMC III) were available; however, no significant events corresponding to the AMC II[1] definition could be identified. The events are listed in Table 4.2. For each antecedent moisture condition, three events were used for calibration, while one event was reserved for verification. The following two sections describe the calibration and verification of the model for the AMC III and AMC I events, respectively.

4.2.1 AMC III Calibration and Verification

The calibration for the AMC III events (very wet initial conditions) consisted of a trial and error procedure, adjusting the hydrograph recession coefficients (K), the time to peak (t_p) and the CN. Each of these parameters were established for each hydrologic unit by multiplying the initial values(contained in Table 4.1) by a common factor. The hydrographs generated by the model were then compared with the actual streamflow hydrographs measured at Donovans, Mount Pearl and Kilbride. Adjustments were made primarily to match the rising limb and peak of each of the observed hydrographs. Emphasis was also placed on matching the volumes of hydrographs as well as the shape of the recession limbs. The event with the highest flows was reserved for verification.

Since baseflows are not simulated by HYMO, they were estimated as being the streamflows which occurred just prior to the start of each event. This simplified estimation of baseflow produces an error which accumulates with time. The error would probably not be significant at the time of peak discharge. The actual baseflow would, however, be somewhat higher at the end of direct runoff.

1 The SCS procedure for determining the antecedent moisture condition (depicted in Figure 4.1) was used in this study.

TABLE 4.2

RAINFALL EVENTS USED TO CALIBRATE AND VERIFY HYMO

Event No.	Date	AMC	Type of Event*	5 – Day Antecedent Rainfall	Rainfall Amount	Maximum Daily Mean Discharge at Donovans	Maximum Daily Mean Discharge at Mount Pearl	Maximum Daily Mean Discharge at Kilbride
				(mm)	(mm)	(m^3/s)	(m^3/s)	(m^3/s)
1	10–06–81	I	V	34.6	30.8	0.824	2.0	5.51
2	08–07–81	I	C	35.6	47.0	1.90	2.83	7.93
3	11–09–81	I	C	34.9	33.3	2.20	2.52	5.41
4	23–09–81	I	C	21.0	53.7	2.16	2.77	8.57
5	10–10–81	III	C	48.8	63.2	4.26	6.40	22.8
5A	11–10–81	III	C	81.3	42.1	4.00	5.93	22.6
6	16–10–81	III	C	47.5	70.3	4.00	6.78	19.6
7	26–11–81	III	V	26.4	88.0	7.78	10.6	37.8

* C = Calibration   V = Verification

The parameters, K and t_p for each hydrologic unit (HU), were ultimately set at 8 and 7, respectively, times that computed by Williams' equations. In addition, the CN for each HU was increased by 10 percent over those presented in Table 4.1 (the average CN was 95). The final parameters for the calibrated model for AMC III are shown in Table 4.3.

Figures 4.2, 4.3 and 4.4 display the observed and simulated hydrographs for each of the three locations at which streamflows were recorded. It is clear from those figures that the calibration is fairly good for Kilbride, fair for Mount Pearl and poor for Donovans.

For Kilbride, the volumes of water represented by the simulated hydrographs are just slightly larger than those for the observed hydrographs. Furthermore, the peak streamflows are somewhat high, about 10 percent on average. The simulated peaks generally occur within one hour of the recorded peaks. Furthermore, the shapes of the rising limb and the recession for the simulated hydrographs are very similar to those for the recorded hydrographs.

The parameters K and t_p could have been adjusted somewhat to provide simulated peaks at Kilbride which were, on average, 10 percent lower and therefore closer to the recorded peaks. The volumes of the simulated hydrographs could also have been reduced slightly by decreasing CN by about two percent. While these actions would have improved the match at Kilbride, the simulations for Mount Pearl and Donovans would have been worse.

For Mount Pearl, the volumes of the simulated hydrographs are close to those of the recorded hydrographs for one calibration event and the verification event. For the other calibration events, the volumes for the simulated hydrographs are about 20 percent less than for the recorded hydrographs. The simulated peaks range from 20 percent low to

TABLE 4.3

PARAMETERS FOR CALIBRATED EVENTS (1981)

AMC III

Hydrologic Unit	CN	K (hours)	t_p (hours)
301	87	13.288	5.964
302	92	5.696	4.046
303	92	4.792	3.353
304	92	6.864	4.900
305	98	3.560	2.443
306	95	3.272	2.478
307	98	3.304	2.667
308	98	3.992	2.541
309	92	1.560	1.695
310	99	5.192	3.598
311	96	1.632	1.680
312	96	7.136	4.536
313	95	4.712	2.891
314	95	3.272	2.548
315	98	2.648	2.527
316	95	6.392	3.276
320	89	9.808	6.713
321	96	9.496	6.664
322	99	7.760	3.801
323	97	4.432	3.360
324	98	6.392	3.703
325	95	5.784	3.605
326	95	3.016	2.891
327	92	0.968	0.840
Average:	95		

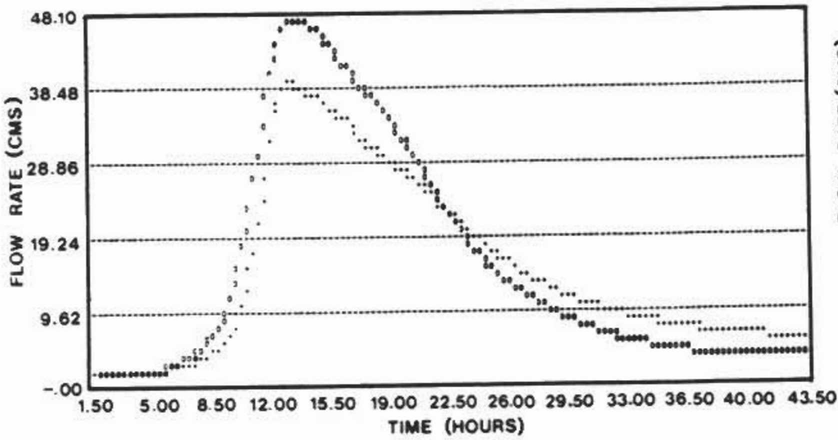
CALIBRATION AND VERIFICATION EVENTS

WATERFORD RIVER AT KILBRIDE

AMC III

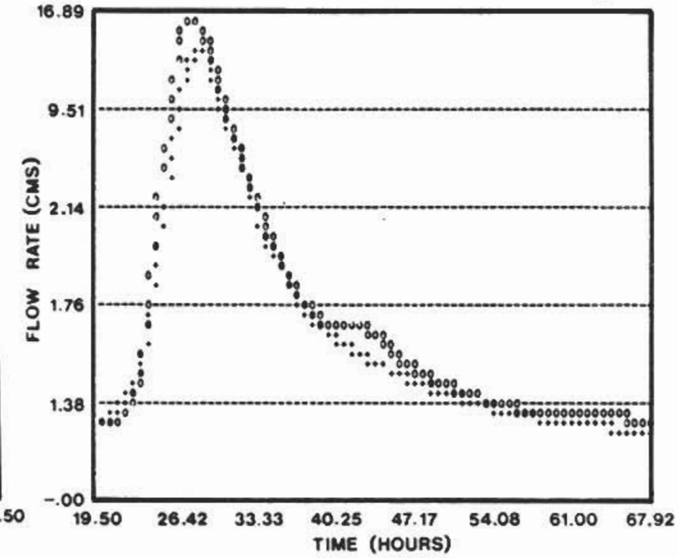
Event 5: Oct. 10, 1981
Calibration Event

Computed ∘
Observed ·
Simulated Peak 0.5 hr late
17% high



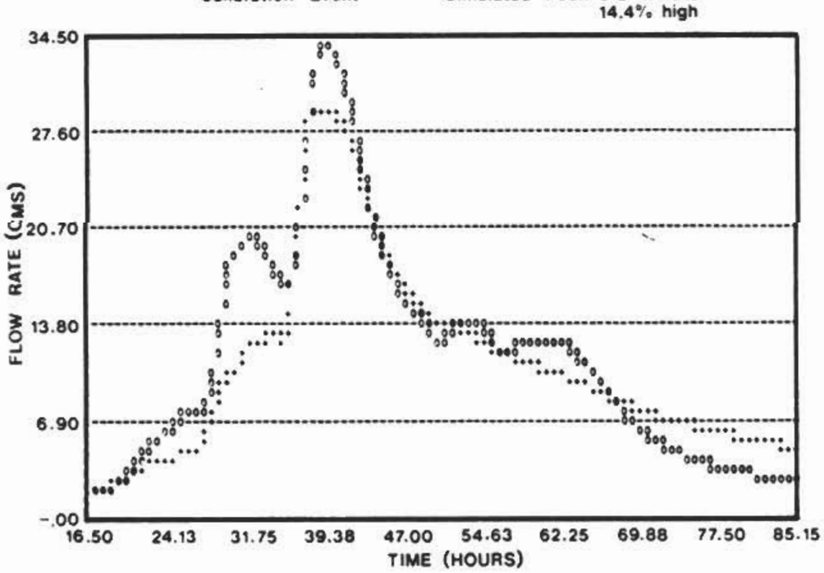
Event 5a: Oct. 11, 1981
Calibration Event

Computed ∘
Observed ·
Simulated Peak 0.75 hr early
7.3% high



Event 6: Oct. 16, 1981
Calibration Event

Computed ∘
Observed ·
Simulated Peak 0.5 hr late
14.4% high



Event 7: Nov. 26, 1981
Verification Event

Computed ∘
Observed ·
Simulated Peak 1.25 hr late
6.6% high

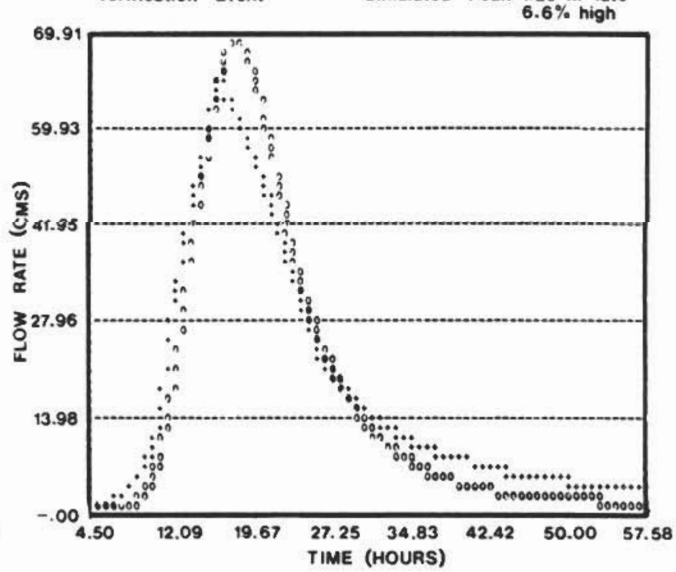


Figure 4.2

CALIBRATION AND VERIFICATION EVENTS

WATERFORD RIVER AT MOUNT PEARL

AMC III

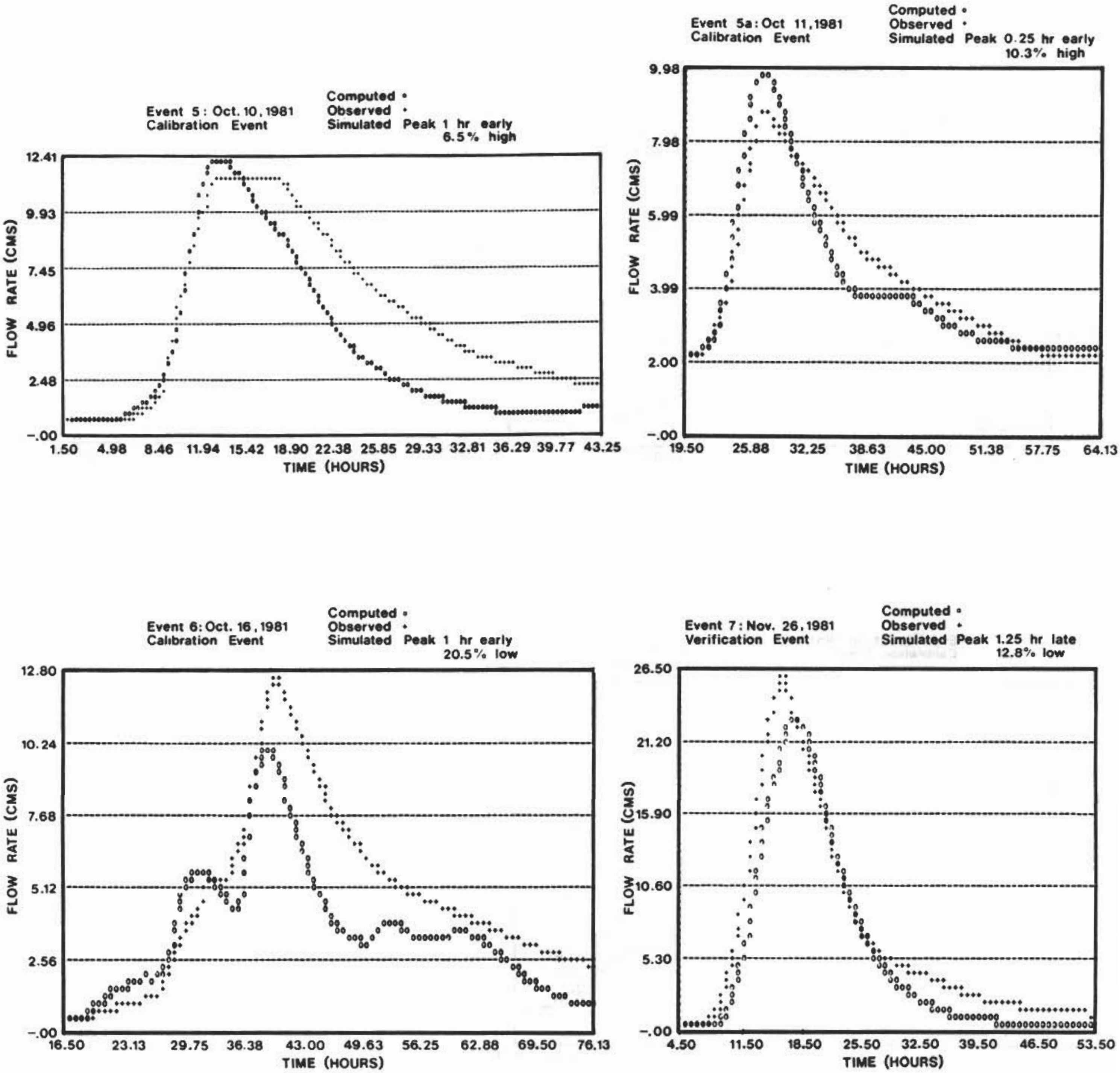
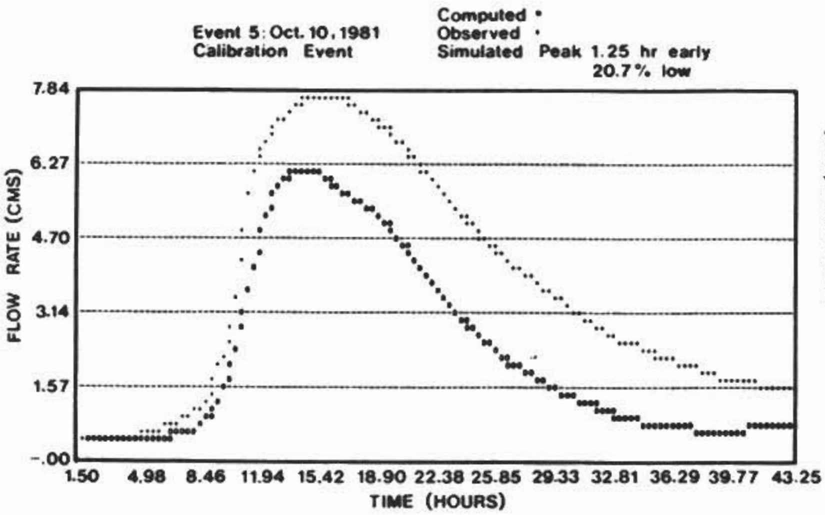


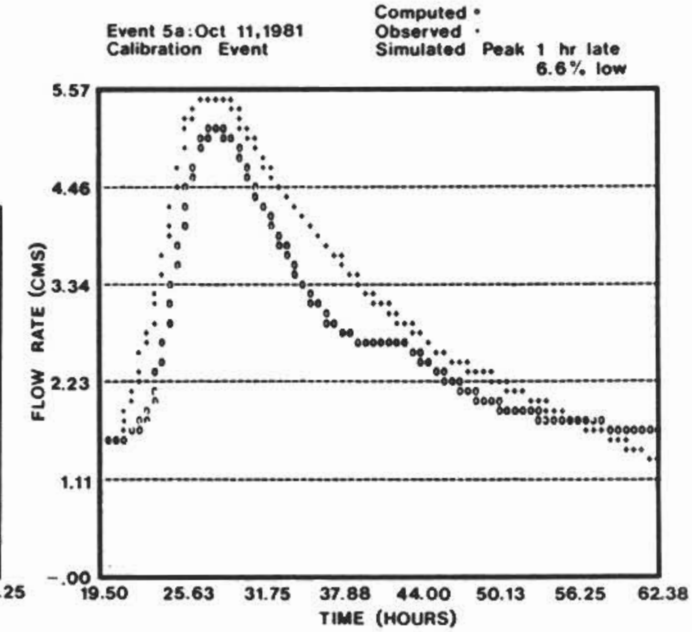
Figure 4.3

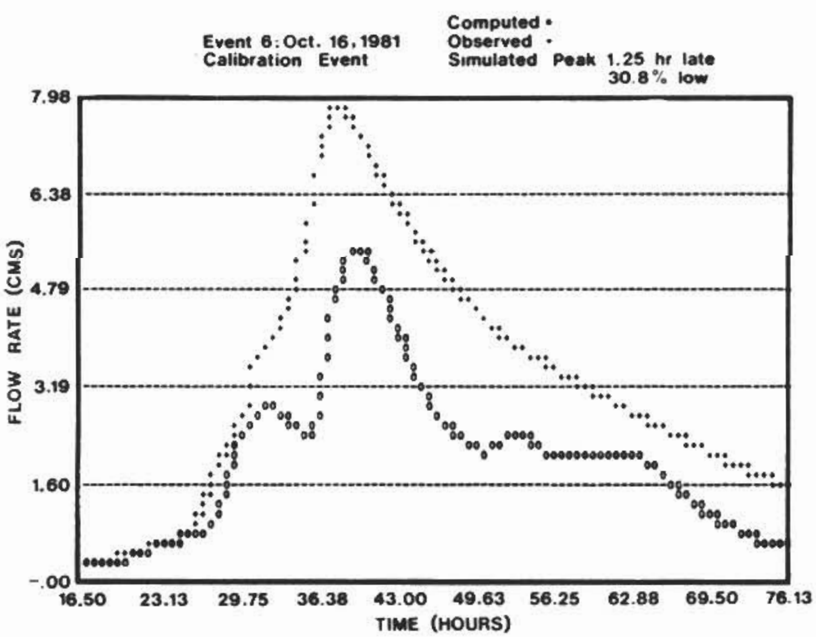
CALIBRATION AND VERIFICATION EVENTS

WATERFORD RIVER AT DONOVANS

AMC III







NO DATA AVAILABLE
FOR NOV. 6, 1981

Figure 4.4

10 percent high. The times to peak are generally within one hour of those for the recorded hydrographs, and the shapes of both the rising and recession limbs are fairly close to those for the observed hydrographs. To provide a better match on average for the hydrograph volumes, CN would have to be increased somewhat. This is the opposite of what would be required for Kilbride.

For Donovans, the volumes of the simulated hydrographs are consistently less than those for the observed hydrographs, by about 25 percent on average. The timing of the peak and the shapes of the rising and recession limbs are good, however.

To improve the fit for Donovans and Mount Pearl, CN would have to be increased significantly. This cannot be done, however, since the average CN for the hydrologic units is about 95. This is only slightly less than the maximum value of 100. For many of the hydrologic units, the estimated value of CN is nearly 100.

The fact that a good match in hydrograph volume could not be made for Donovans and Mount Pearl seems to indicate that the streamflow data for those two sites may not be very accurate, at least for high flow events. This is quite possible since the hydrometric stations at these sites were established specifically for this study. Accordingly, the relationships between water level (stage) and streamflow (discharge) at those two sites are most likely poorly defined under high flows/stages. The fairly good fit for the Kilbride hydrometric station's storm flows is probably the result of a better defined stage versus discharge relationship.

4.2.2 AMC I Calibration and Verification

The calibration for the AMC I (very dry initial conditions) events also consisted of a trial and error procedure, adjusting the hydrograph recession coefficients (K), the time to peak (t_p) and CN. As was the case for the AMC III calibration, the parameters were determined for each hydrologic unit (HU) by multiplying the initial values for each of the three parameters (contained in Table 4.1) by a common value. In this case, the event with the lowest peak flow was reserved for verification.

The parameters K and t_p for each HU were ultimately set at 18 and 8, respectively, times that computed by Williams' equations. The initial estimates of CN had to be increased, however, by 52 percent, resulting in an average CN for the HUs of about 87. The final set of parameters for the AMC I calibration are presented in Table 4.4.

Figures 4.5, 4.6 and 4.7 display the observed and simulated hydrographs for each of the three locations at which streamflows were recorded. As for the AMC III case, the AMC I calibration is fairly good for Kilbride, fair for Mount Pearl and poor for Donovans.

For Kilbride, the volumes of water represented by the simulated hydrographs are about the same as those for the observed hydrographs. The simulated peaks are, on average, just slightly greater than observed, and they occur within plus or minus one hour of the recorded peaks. Furthermore, the shape of the rising and recession limbs of the hydrographs are similar to those for the recorded hydrographs. The tendency to underestimate flows toward the end of each event is probably due to the simplistic method used to estimate the baseflow component.

For Mount Pearl, the volumes of the simulated hydrographs are between about 5 and 20 percent below those for the recorded hydrographs. The simulated peaks for two events were fairly close to recorded peaks; however, for one event, the simulated peak was only about one-half of the recorded peak flow. The simulated peaks occurred from 0.5 to 2 hours

TABLE 4.4

PARAMETERS FOR CALIBRATED EVENTS (1981)

AMC I

Hydrologic Unit	CN	K (hours)	t_p (hours)
301	76	29.898	6.816
302	82	12.816	4.624
303	88	10.782	3.832
304	87	15.444	5.600
305	93	8.010	2.792
306	87	7.362	2.832
307	91	7.434	3.048
308	93	8.982	2.904
309	88	3.510	1.936
310	96	11.682	4.112
311	85	3.672	1.920
312	91	16.056	5.184
313	88	10.602	3.304
314	84	7.362	2.912
315	94	5.958	2.888
316	91	14.382	3.744
320	78	22.068	7.672
321	87	21.366	7.616
322	94	17.460	4.344
323	88	9.972	3.840
324	91	14.382	4.232
325	83	13.014	4.120
326	84	6.786	3.304
327	75	2.178	0.960
Average:	87		

CALIBRATION AND VERIFICATION EVENTS

WATERFORD RIVER AT KILBRIDE

AMC I

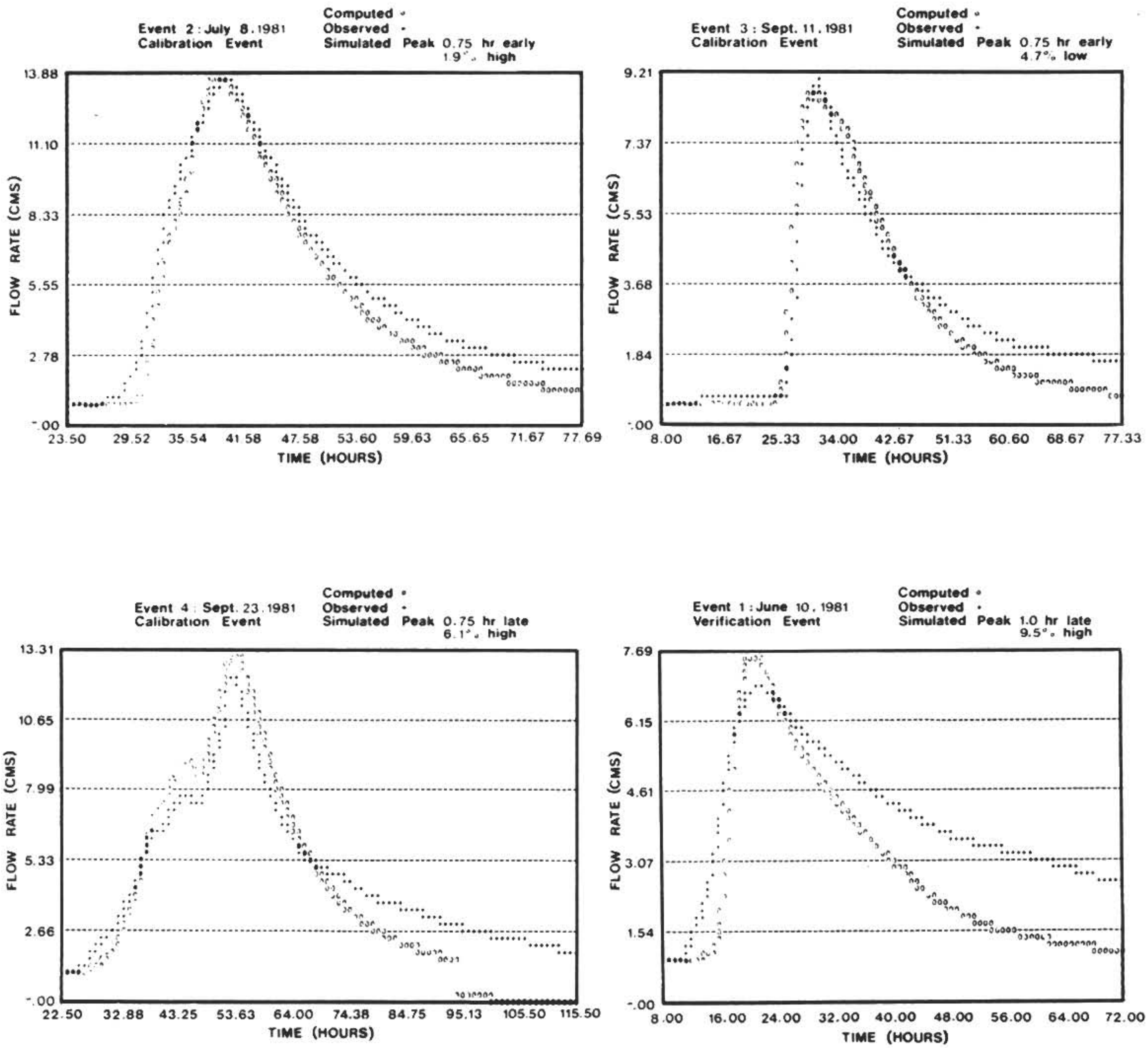
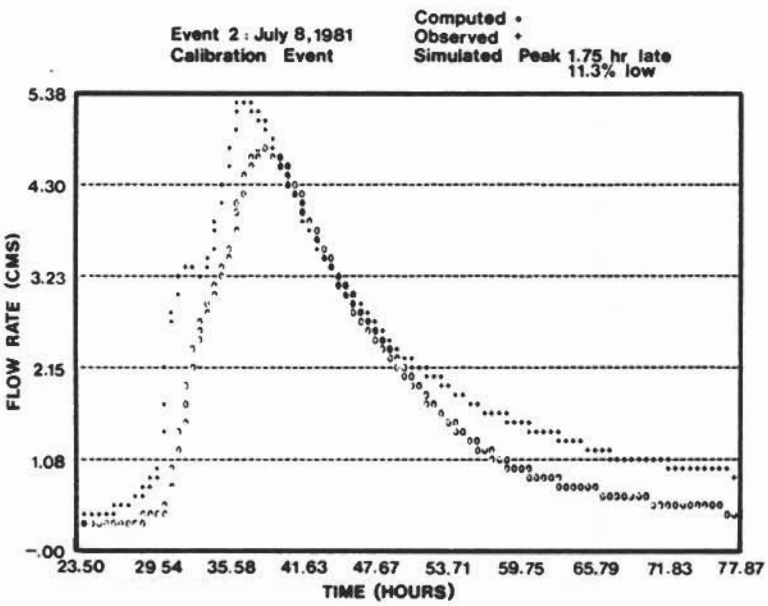
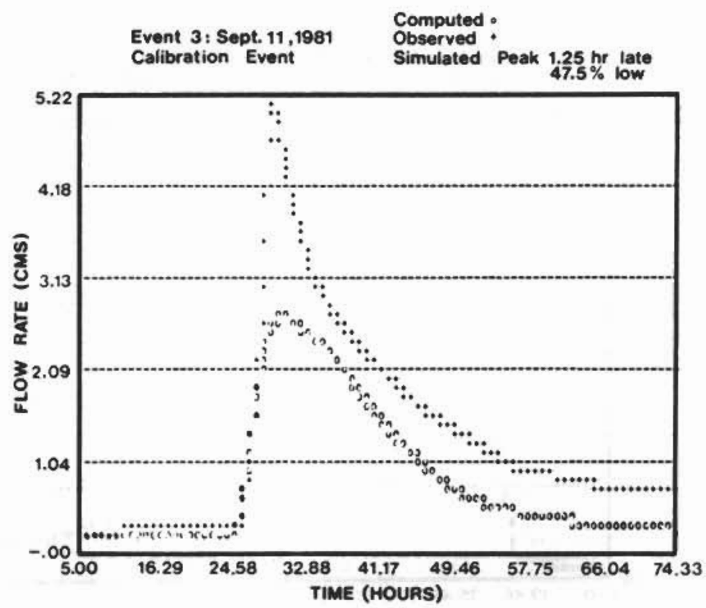
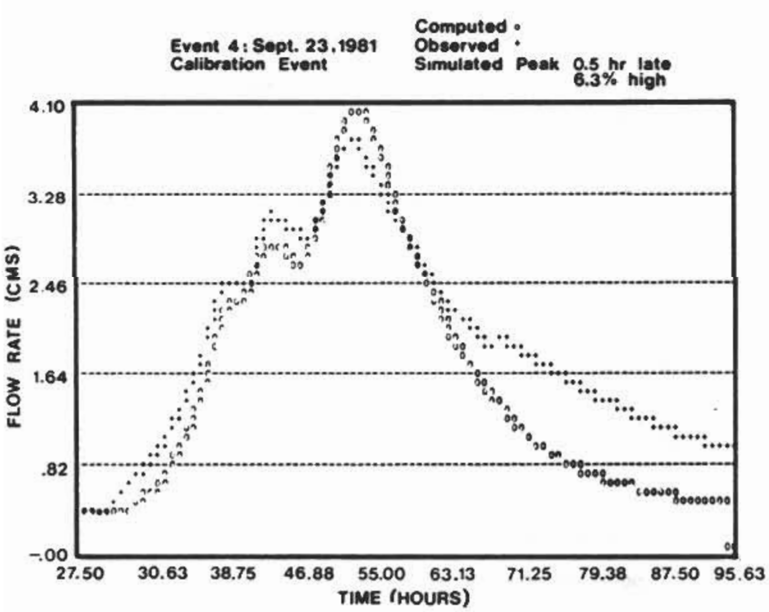


Figure 4.5

CALIBRATION AND VERIFICATION EVENTS
WATERFORD RIVER AT MOUNT PEARL
AMC I







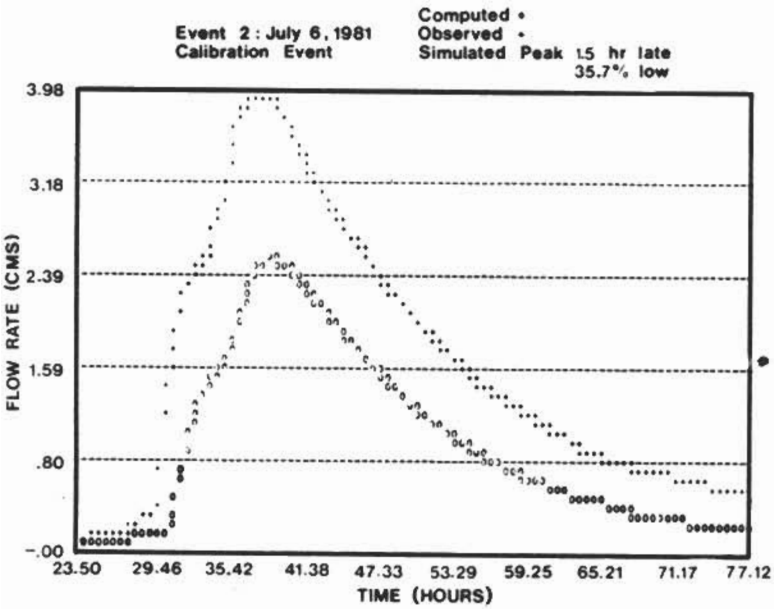
NO DATA AVAILABLE
FOR JUNE 10,1981

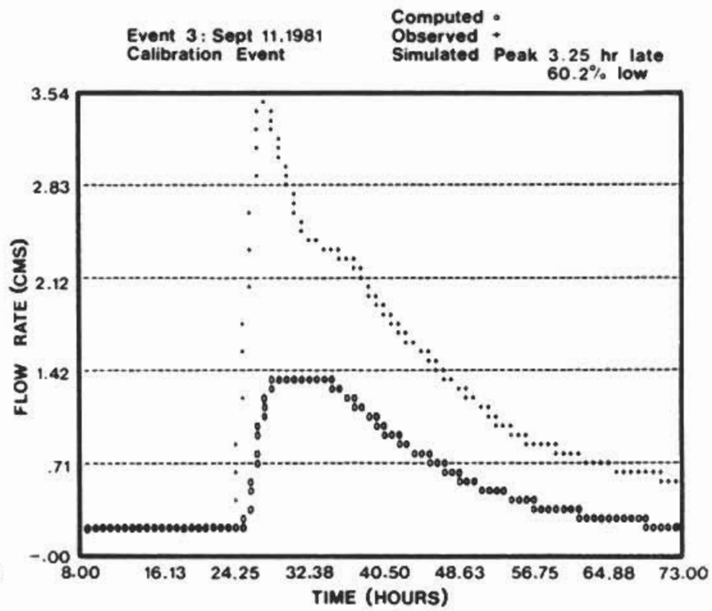
Figure 4.6

CALIBRATION AND VERIFICATION EVENTS

WATERFORD RIVER AT DONOVANS

AMC I





NO DATA AVAILABLE
FOR SEPT. 23,1981

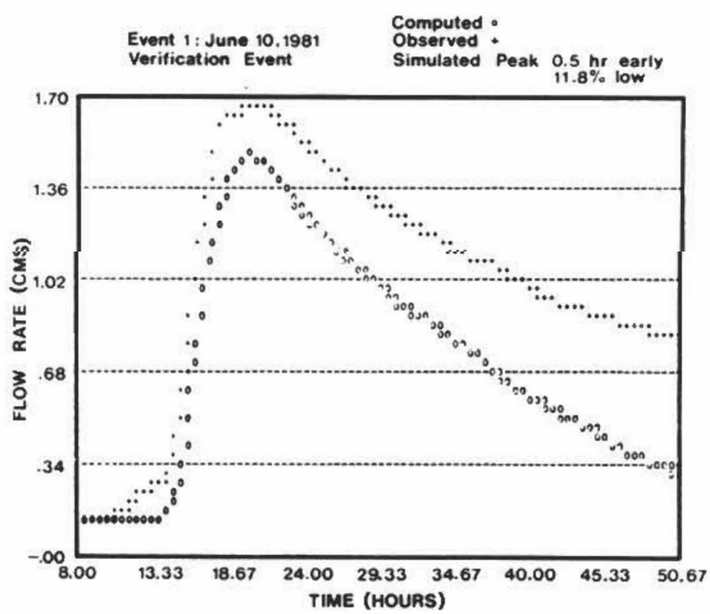


Figure 4.7

later than the recorded peaks. The shapes of the rising and recession limbs are fairly close to those of the observed hydrographs.

For Donovan's, the volumes and peak flows of the simulated hydrographs are significantly lower than for the recorded events. The timing of the peaks varied from one-half hour early to perhaps as much as three hours late. The shapes of the rising and recession limbs are reasonably good, however.

As was the case for the AMC III events, it would be necessary to significantly increase the runoff coefficient, CN, in order to improve the simulation for Donovans and Mount Pearl. Again, this cannot be done since the average CN for the hydrologic units is already quite high at 87. Increases in CN for the HUs upstream of Donovan's could not be justified without increasing the CNs for all the HUs. This would then produce a poorer fit for Kilbride.

As was suspected for the AMC III calibration, the streamflow data for Donovans and perhaps Mount Pearl may not be very accurate, at least for high flows. As stated previously, it is not unreasonable to suspect a significant amount of error in the recorded high flows since the relationships between water level and streamflow at those two sites are most likely poorly defined for high flows.

4.2.3 Comments on Parameter Values

The increases which were made to the initial estimates of parameters CN, K and t_p seem high. A closer examination of a surficial geology map produced for the Waterford River Basin Urban Hydrology Study, subsequent to the assignment of the soils of the basin to the SCS hydrologic soils groupings, provides justification for the increases made in CN during calibration. The subject map (ref. 2) clearly indicates that the overburden in the Waterford Basin is very thin (0 to 5 metres) -

much thinner than was assumed in setting the initial estimates of CN. The overburden is composed of a very compact lodgement till which has a significant (10 percent) silt-clay content. This till is often overlain by another till which is looser, coarser and therefore permeable in nature, derived by a supra-glacial or melt-out process (ref. 2). For these reasons, the initial estimates of CN presented in Table 4.1 could be justifiably revised by as much as 5 to 10 percent for AMC III conditions and by 30 percent or more for AMC I. This is comparable to the adjustments which were found to be required during calibration.

It is reasonable that a very thin overburden material, which is compact, poorly sorted and has a high silt-clay content, would result in a large portion of storm rainfall contributing to direct runoff.

Initially, it was believed that the large adjustments required to the parameters K and t_p were also related to the overburden. It was felt that the overlying, more permeable till deposit, in association with the relatively impermeable lodgement till and bedrock, may be producing a large amount of flow just under the surface (interflow). A subsequent simulation of one event using the entire study area as one hydrologic unit, however, indicated that values of K ant t_p, only approximately two times those provided by Williams' equations, were required to obtain a good match to the observed flows at Kilbride. It was concluded then that a possible explanation for the large adjustments required to the initial estimates of K and t_p might be related to the size of the HUs. The drainage areas of the HUs into which the study area was divided were about the size of the smallest ones used in the derivation of Williams' equations. In any event, the calibration and verification procedure consistently indicated that large adjustments to the intial estimates of these parameters were necessary.

4.3 Sensitivity Analysis

A sensitivity analysis of the calibrated watershed model was conducted to determine the sensitivity of simulated peak flows to changes in calibration parameters as well as the recorded rainfall and streamflow data.

The parameters/variables considered in the sensitivity testing were the rainfall amount, the curve number (CN), Manning's 'n', the recession constant (K), the time to peak (t_p), and streamflow. The testing was done by varying one parameter or variable at a time while holding the others at their previously determined values for two AMC III and two AMC I events.

The results of the sensitivity analysis are shown in Table 4.5. Since the findings were similar, only one table was used to give the results for both AMC III and AMC I conditions. As would be expected, the model is very sensitive to the variation of the rainfall amount and the CN number. This indicates the importance of having a reliable rain gauge network and adequate soils and land use information. However, the results also show that the peak flows are not significantly affected by 10 percent changes in K, t_p and Manning's 'n'.

The sensitivity of the model to errors in streamflow data used for calibration was determined by the adjustment in CN required to obtain a good fit for the peak. As shown in Table 4.5, by decreasing the streamflow by 30 percent, the CN's had to be decreased by 7 percent. Understandably, a realistic water balance could not be obtained when the streamflows were increased by 30 percent since that would have resulted in all CNs being much greater than 100.

TABLE 4.5

SENSITIVITY ANALYSIS

% of Change in specified Parameter/Variable	Variation of Peak Flow at Donovans		Mount Pearl		Kilbride	
Rainfall +25%, -25%	+25%	-32%	+25%	-31%	+23%	-30%
'n' +10%, -10%	0%	+0.1%	-0.5%	+0.4%	-1.1%	+12%
CN +10%, -10%	41%	-43%	+40%	-43%	43%	-40%
K +10%, -10%	-1%	+1.1%	-1.4%	+1.1%	-1.8%	+1.6%
t_p +10%, -10%	-5%	+5.6%	-5%	+4%	-4.7%	+5.8%

	Variation in CN		
Streamflows -30%	-7%	-7%	-7%
Streamflows +30%	N/A	N/A	N/A

5.0 EVALUATION OF IMPACT OF HISTORIC URBANIZATION ON PEAK FLOWS

The typical consequences of urbanization include, among other things, decreases in the amount of pervious area and depression storage. These changes increase the capacity of the area to convey stormwater runoff quickly through and out of the watershed, often resulting in a significant decrease in the amount of water which infiltrates into the soil and consequently a corresponding increase in total volume of surface runoff.

To assess the historic impact of urbanization on peak flows in the Waterford River Basin, two storm events which produced high streamflows in 1974, the year the hydrometric station was established at Kilbride, were simulated using the calibrated model. If there had been a significant effect on the streamflow in the Waterford River Basin, as was postulated in the proposal for the overall Waterford River Basin Study (ref. 27), this should be seen by comparing the simulated hydrographs for the two 1974 events (simulated using the same calibration parameters: K, t_p and CN determined based on calibration to 1981 conditions) and the two storm hydrographs actually recorded by the Water Survey of Canada. It would be expected that the events, simulated in this way, would have higher peak flows, a shorter time to peak and greater runoff volume than observed.

The two events selected for simulation were August 28 and September 18, 1974. The antecedent moisture conditions for both were I. Unfortunately, there were no AMC III events available. The simulation was carried out using the parameters for the 1981 events as shown in Table 4.4. It should be noted that the calibration to flows recorded at the Kilbride streamflow gauge during 1981 was good.

It can be seen from Figure 5.1 that the streamflow peak at Kilbride for August 28, 1974 was underestimated by 21% and the time to peak was two hours late. This is exactly the opposite of what would be expected if significant urbanization had occurred. No comparison could be made for Donovans and Mount Pearl since hydrometric stations were not operational at those sites in 1974.

For the event of September 18, 1974, the peak at Kilbride, as shown in Figure 5.2, was overestimated by 10 percent, while the time to peak was one hour early. These are the type of changes which would be evident as a result of urbanization. However, their magnitudes are within the range of error associated with calibration. Again, no comparison was possible for Donovans and Mount Pearl since hydrometric stations were not installed at those locations until 1981. Based on these two simulations, it would appear that the urbanization which occurred between 1974 and 1981 has probably had no significant impact (i.e. greater than the error range associated with model calibration) on high streamflows at Kilbride.

The fact that these simulations seem to indicate that urbanization since 1974 has had no major impact on peak flows in the Waterford River at Kilbride corroborates the findings of the graphical and tabular analyses of annual extreme high and low flows presented in Chapter 2. As stated previously, however, the amount of additional urbanization which occurred in the study area between 1974 and 1981 was not great.

SIMULATION OF AUGUST 28, 1974 STORM
WATERFORD RIVER AT KILBRIDE

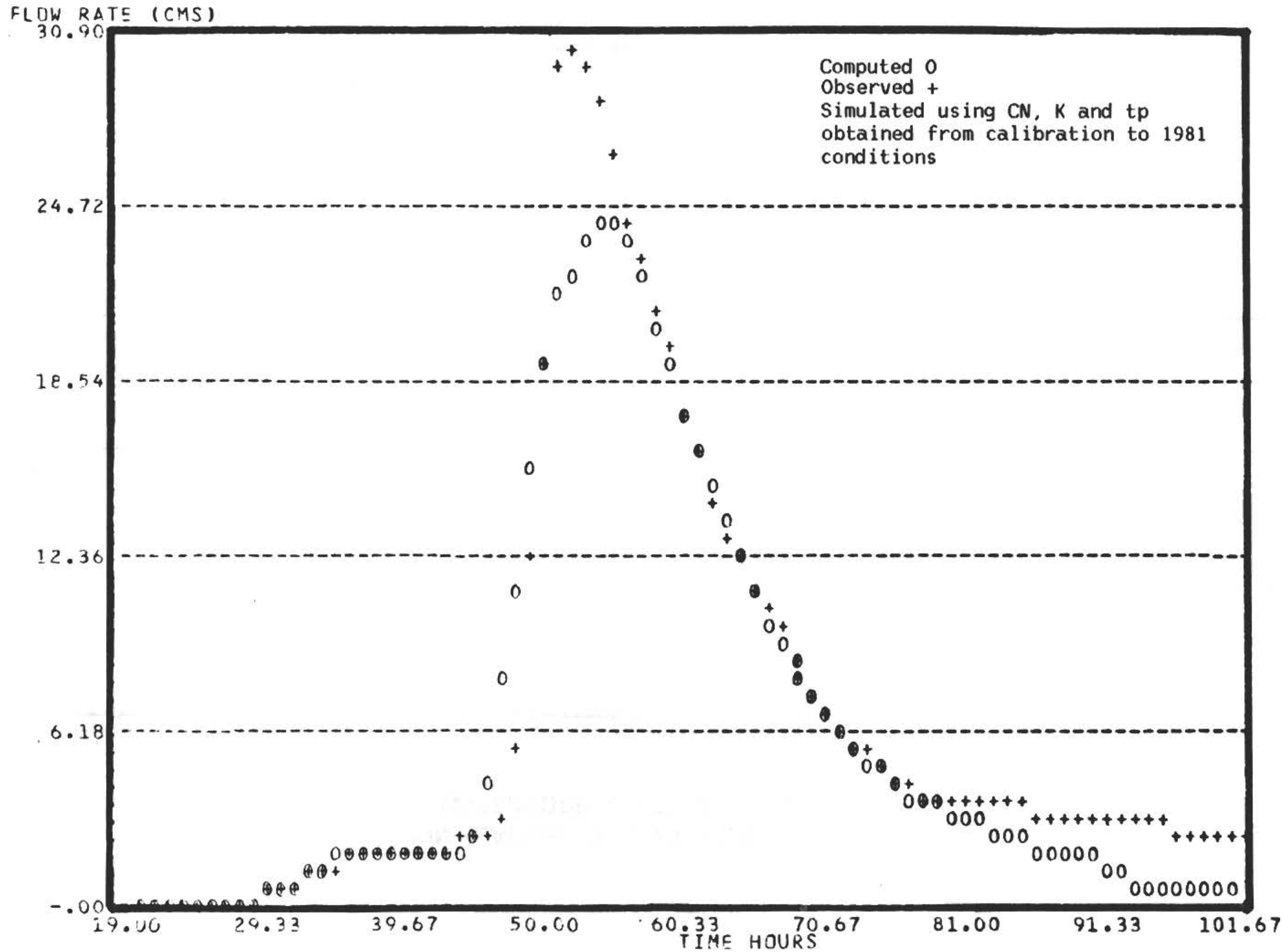


Figure 5.1

SIMULATION OF SEPT. 18, 1974 STORM
WATERFORD RIVER AT KILBRIDE

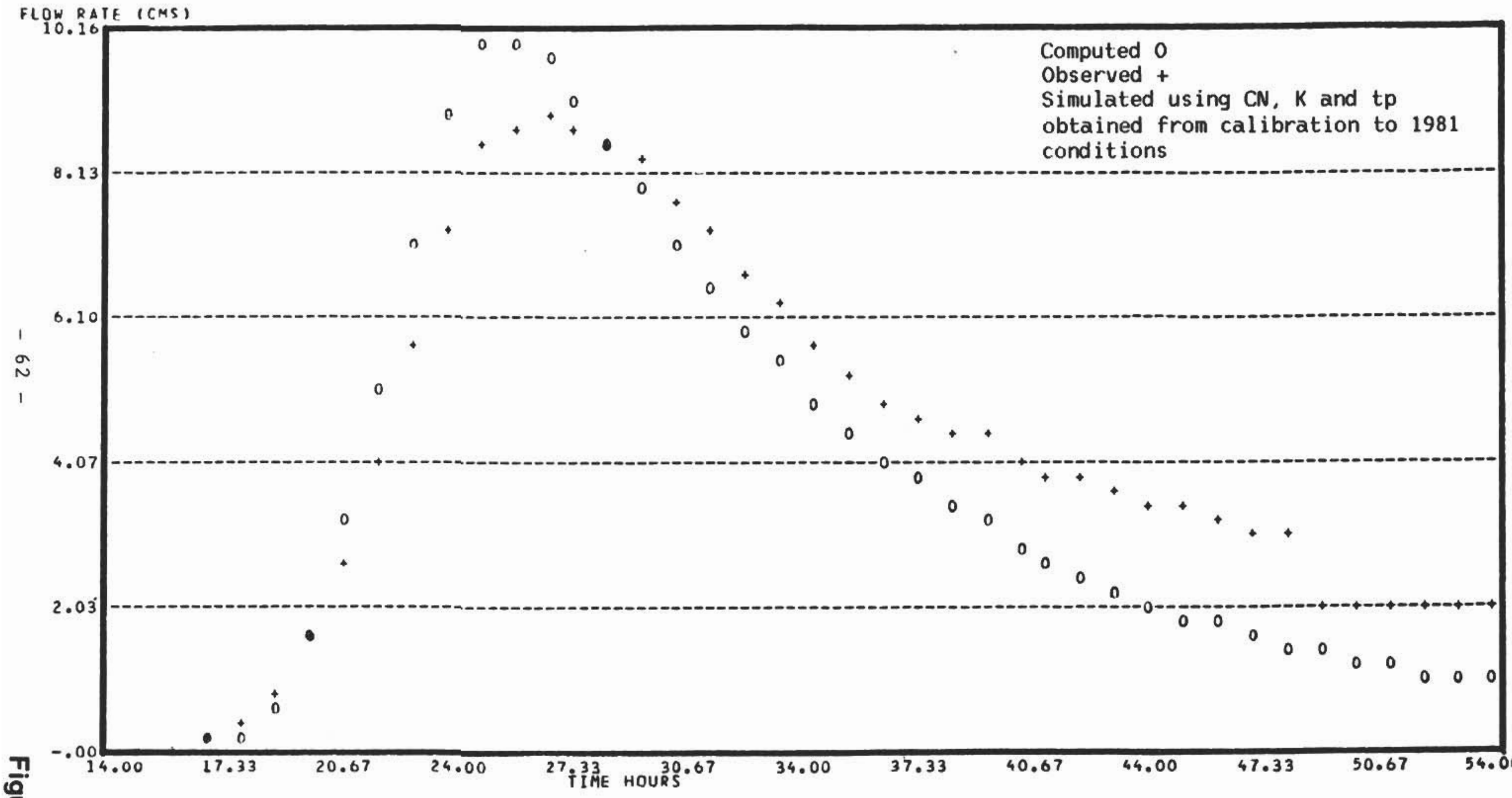


Figure 5.2

6.0 DESIGN STORM AND RESULTING STREAMFLOWS

In addition to the objective of determining the effects of urbanization on peak flows in the Waterford River Basin, the watershed modelling was also undertaken to provide estimates of the 20 and 100 year recurrence interval peak flows for use in the flooding component of this study. These flows were required so that estimates could be made of the areal extent and depth of flooding along the Waterford River.This Chapter describes the work undertaken to support the Flood Study.

6.1 Introduction

Ideally, flood flows for specific recurrence intervals are estimated from the historical records for a hydrometric station located at or near the point of interest. In practice, however, this is often not possible since it is unlikely that a hydrometric station will be located at the point of interest or even a short distance up or downstream. Occasionally, a hydrometric station will be located on the stream; however, the period of record may be too short to permit valid statistical analyses.

When no suitable hydrometric records are available, other techniques must be used to estimate flood flows. These include the use of various statistical techniques using the records from local or regional hydrometric stations located on other streams as well as rainfall/runoff models such as HYMO. In the latter case, it is generally assumed that the T year recurrence interval rainfall storm will produce the T year recurrence interval flood flow. This is not always the case since factors such as varying antecedent soil moisture conditions and vegetative cover, whether or not the ground is frozen, and whether or not a snowpack is present, can diminish or worsen the effects of storm rainfall.

On the Waterford River, the hydrometric station at Kilbride has been in operation since the latter part of 1973. Flood flow estimates could have been computed from the records at this site; however, the station had been in operation for just 10 years when this study was being undertaken.

An alternate method could be to use equations developed from a regional analysis of streamflow peaks and pertinent physiographic and climatic parameters. For the purpose of the flooding component of the study, however, it was decided to use the calibrated HYMO model in association with an analysis of the 22 years of short duration, tipping bucket gauge, rainfall data collected at St. John's Airport by the Atmospheric Environment Service (AES) of Environment Canada. Airport data were used since long term records of storm rainfall do not exist for the St. John's West CDA. The applicability of St. John's Airport rainfall data to the study area was verified, in part, by comparing the storm volumes for 13 storm events. As shown in Table 6.1, the difference in volume varies from one storm to another, but on the average, they were quite similar.

Flood flow estimates computed in the above manner for Kilbride were then compared with estimates determined using a statistical frequency analysis of the streamflow data. The assumption was made that, based on the findings in Chapters 2 and 5, the streamflows recorded at Kilbride since 1974 were essentially unaffected by urbanization between 1974 and 1981. Equations from a recently completed, regional flood frequency analysis could not be used as a comparison since one of the parameters used in that analysis, ACLS, which indicates the proportion of the drainage area controlled by lake and swamp (ref. 21), fell outside the range for which the equations are applicable. Furthermore, the regional technique was derived using data from completely natural (non-urban) watersheds.

The procedure used to estimate design flood flows began with the determination of the storm parameters, which include the time distribution, the rainfall amount and the storm duration. Then, using the calibrated watershed model and an appropriate antecedent moisture condition (AMC), the peak flows were simulated. Details of the procedure are given in the following sections.

TABLE 6.1

STORM VOLUMES - AIRPORT vs CDA

Date of Event	St. John's Airport (mm)	St. John's West CDA (mm)
May 2 - 4, 1981	14.6	10.6
May 23 - 26, 1981	25.8	28.4
June 7 - 12, 1981	83.1	70.6
June 15 - 19, 1981	12.6	10.4
June 21 - 23, 1981	22.0	16.8
June 27 - 30, 1981	16.7	12.9
July 5 - 12, 1981	97.5	101.2
July 14 - 15, 1981	22.0	25.8
August 4 - 9, 1981	25.6	29.6
August 17 - 20, 1981	37.4	48.8
September 6 - 8, 1981	25.6	29.9
September 23 - 26, 1981	55.2	58.4
October 3 - 4, 1981	20.8	29.9
Average	35.3	36.4

6.2 Time Distribution

The temporal distribution of rainfall of short duration, in particular when the duration is much less than 12 hours, is a critical factor in selecting an appropriate design storm. It has been found that characteristics of short duration storm rainfall vary significantly across Canada (ref. 8). For these reasons, the Atmospheric Environment Service (AES) of Environment Canada developed a set of curves showing storm distribution statistics for one-hour and twelve-hour rainfall durations for various regions of Canada. The curves applicable to the Canadian east coast are presented in Figures 6.1(a) and 6.1(b) (ref. 8).

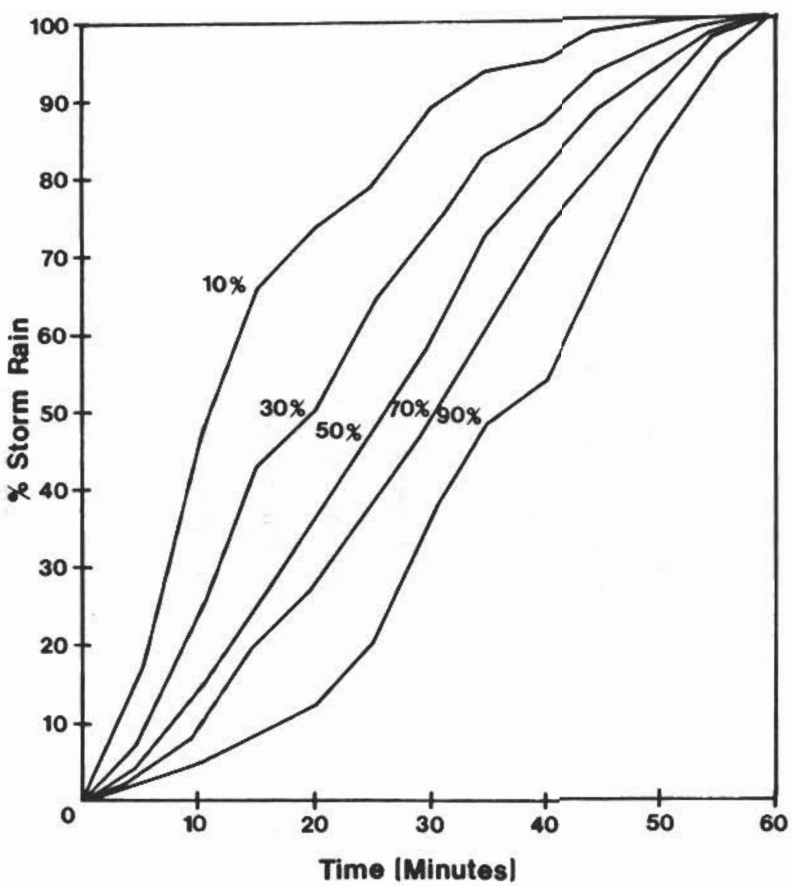
More recent (1982) AES studies also included an investigation to determine the most appropriate storm rainfall distribution for each region for use in design work. The resulting distributions were calculated so as to provide simulated peak streamflows similar in magnitude to those obtained from frequency analyses of simulated streamflow data. The resulting distribution for the Canadian east coast was very similar to the 30 percent curve shown in Figure 6.1.

The AES also compared their storm distribution against several other well known design storm distributions used in Canada (Huff 3rd Quartile, Chicago, SCS Type III, and Huff 2nd Quartile). It was concluded that the AES distribution was the most appropriate for use on the Canadian east coast.

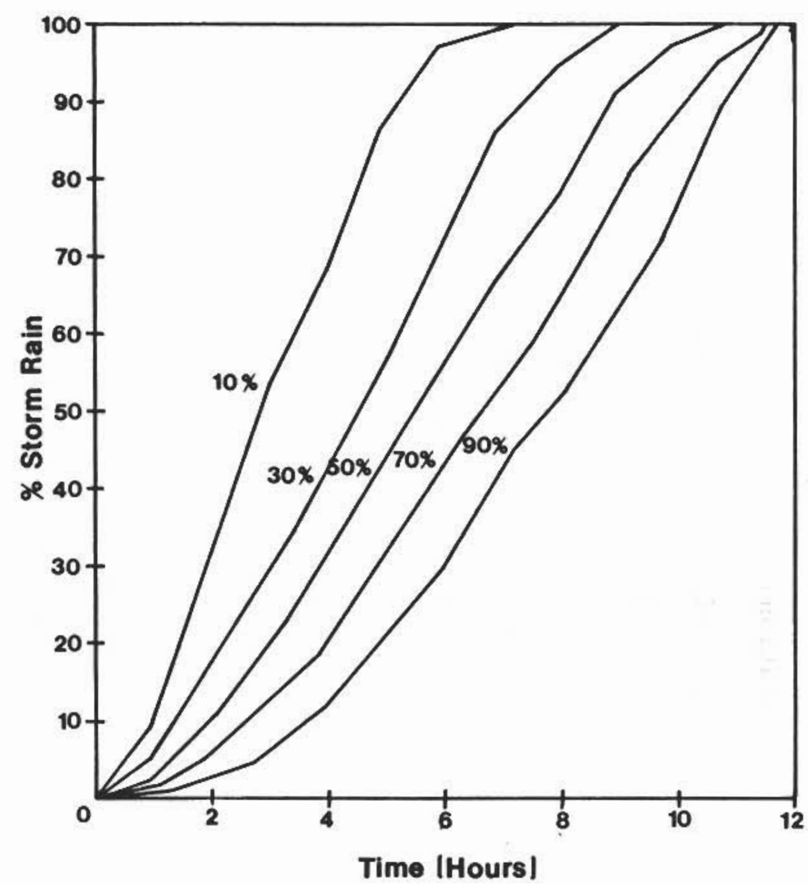
The AES report (ref. 8) concluded that, for storms with a duration in the vicinity of 12 hours, the relative differences in peak flows due to the choice of time distribution, between the various methods are no more than about 4 percent.

For the purposes of the present study, the 50 percent distribution curve was initially adopted (in 1981). When the more recent work of

STORM RAIN DISTRIBUTION
CANADIAN EAST COAST
[ref. 8]



1-HOUR STORM RAIN DISTRIBUTION



12-HOUR STORM RAIN DISTRIBUTION

Note: Curves show % of events with % storm rain ≥ values plotted.

Figure 6.1[a]

Figure 6.1[b]

the AES was discovered (after the modelling work was completed for this study in 1982), the simulations of design flows were repeated, based on the 30 percent distribution curve. Use of the 30 percent distribution curve produced design flood flows which were in the order of 1 to 2 m^3/s lower than those obtained using the 50 percent distribution curve. The more conservative peak flows resulting from the simulations based on the 50 percent distribution were retained, however, as the design flows.

In order to compare the one-hour and the twelve-hour distributions, the 50 percent curves for both storm durations (from Figure 6.1) were plotted as shown in Figure 6.2, with the storm duration reduced to a dimensionless scale. As can be seen, the two distributions are quite similar. It was then assumed, for the purposes of this study, that the various storm durations between one and twelve hours have similar time distribution, and that the time distribution for other durations could be found by interpolating between these curves.

6.3 Storm Duration

It is often assumed, but is not always the case, that the greatest streamflow for a given rainfall/snowmelt intensity would occur at the time when runoff from the entire drainage area is contributing to the streamflow. That time, referred to as the time of concentration (t_c) is defined as the time required for surface runoff from the most remote part of the drainage basin to reach the point being considered. In hydrograph analysis, t_c is computed as the time from the end of "excess" rainfall to the point on the falling limb of the hydrograph where the recession curve begins (often assumed to be the point of inflection).

The SCS handbook (ref. 10) recommends that, if the time of concentration is less than six hours, the duration of the design storm

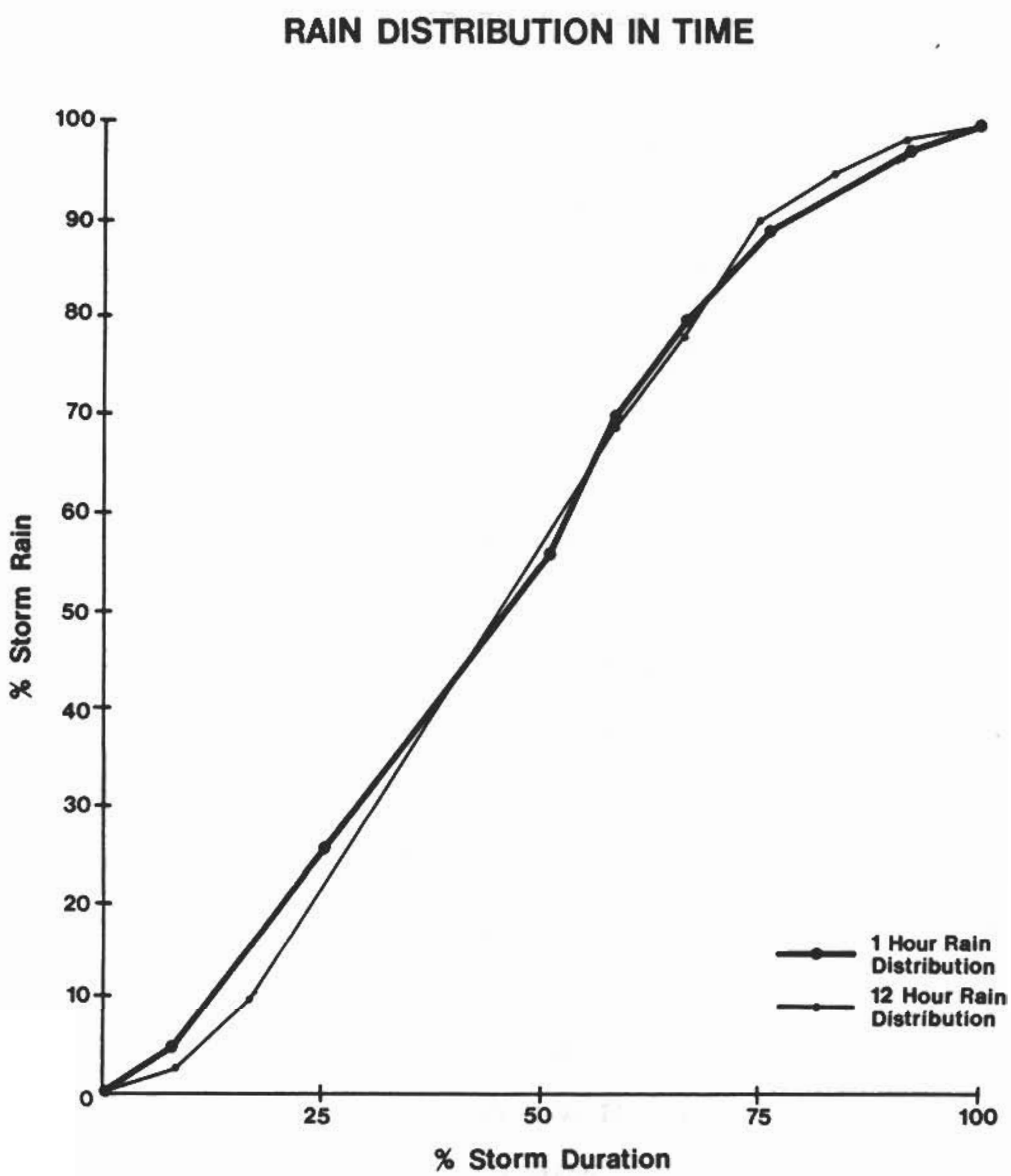

RAIN DISTRIBUTION IN TIME
100
90
80
70
60
50
40
30
20
10
0
% Storm Rain
25
50
75
100
% Storm Duration
1 Hour Rain Distribution
12 Hour Rain Distribution

Figure 6.2

should be made equal to six hours. Otherwise, it is made equal to the time of concentration of the watershed.

In this study, two techniques have been used to evaluate the time of concentration. The first was the Bransby Williams method (ref. 6). This method is especially useful for drainage areas where the defined channel extends to 30 percent or more of the watershed length. It takes into consideration the shape, length and slope of the watershed. Details of the calculations are shown in Appendix E. The values of time of concentration obtained for Donovans, Mount Pearl and Kilbride are respectively 2.6 hours, 3.3 hours and 4.9 hours. Experience using this equation in Ontario (ref. 6) has shown that this method tends to underestimate times of concentration.

The second approach used in evaluating the time of concentration was by comparing the rainfall distribution (end of rainfall) with the streamflow hydrograph inflection point for the calibration/verification events, according to the definition of the time of concentration as stated above. The results obtained are shown in Table 6.2. The fact that the times of concentration determined in this manner are much larger than obtained by the Bransby Williams method seems to substantiate to some degree the requirement to use such large values of t_p.

In order to determine the appropriate storm duration for the drainage basins to each of Donovans, Mount Pearl and Kilbride, the calibrated HYMO model for AMC III was run using 100 year return period rainfall intensities for various rainfall durations. The corresponding rainfall amounts for the 100 year return period, from the intensity-duration-frequency analysis (refer to Section 6.4), were then distributed according to the 50 percent distribution developed by AES. The critical storm duration for each sub-basin should be the one that produces the

TABLE 6.2

TIME OF CONCENTRATION

Date	AMC	Donovans	t_c (hours) Mount Pearl	Kilbride
08-07-81	I	10.0	12.2	13.8
23-09-81	I	-	11.5	12.3
10-10-81	III	6.3	7.0	8.8
11-10-81	III	9.0	8.3	9.5
26-11-81	III		6.3	9.8
	Average (all)	8.4	9.1	10.8
	Average (AMC I)	10.0	11.8	13.0
	Average (AMC III)	7.7	7.2	9.4

highest peak flow at the outlet. The resulting hydrographs for durations of 11, 12 and 13 hours for Kilbride are shown in Figure 6.3. This Figure shows that the highest peak flow at Kilbride is produced by the 12-hour storm.

Furthermore, the peak flows at Donovans and Mount Pearl also occurred for the 12 hour storm duration, even though the times of concentration determined from the analysis of historic streamflows are somewhat less than 12 hours. The differences in peak flows, however, were only approximately four percent for each change in duration of one hour; therefore, the blanket use of a 12 hour storm duration for simulating peak flows at Donovans, Mount Pearl and Kilbride is not expected to produce significant errors.

6.4 Rainfall Amounts

Based on rainfall data collected at St. John's Airport over a period of 22 years, AES has performed an intensity-duration-frequency analysis. From the resulting curves, presented in Appendix D, the rainfall amounts for the 20 and 100 year return period storms were computed. These are presented in Table 6.3 along with the upper and

100 YEAR STORM HYDROGRAPHS PRODUCED BY DIFFERENT RAINFALL DURATIONS WATERFORD RIVER AT KILBRIDE AMC III (Excludes Baseflow)

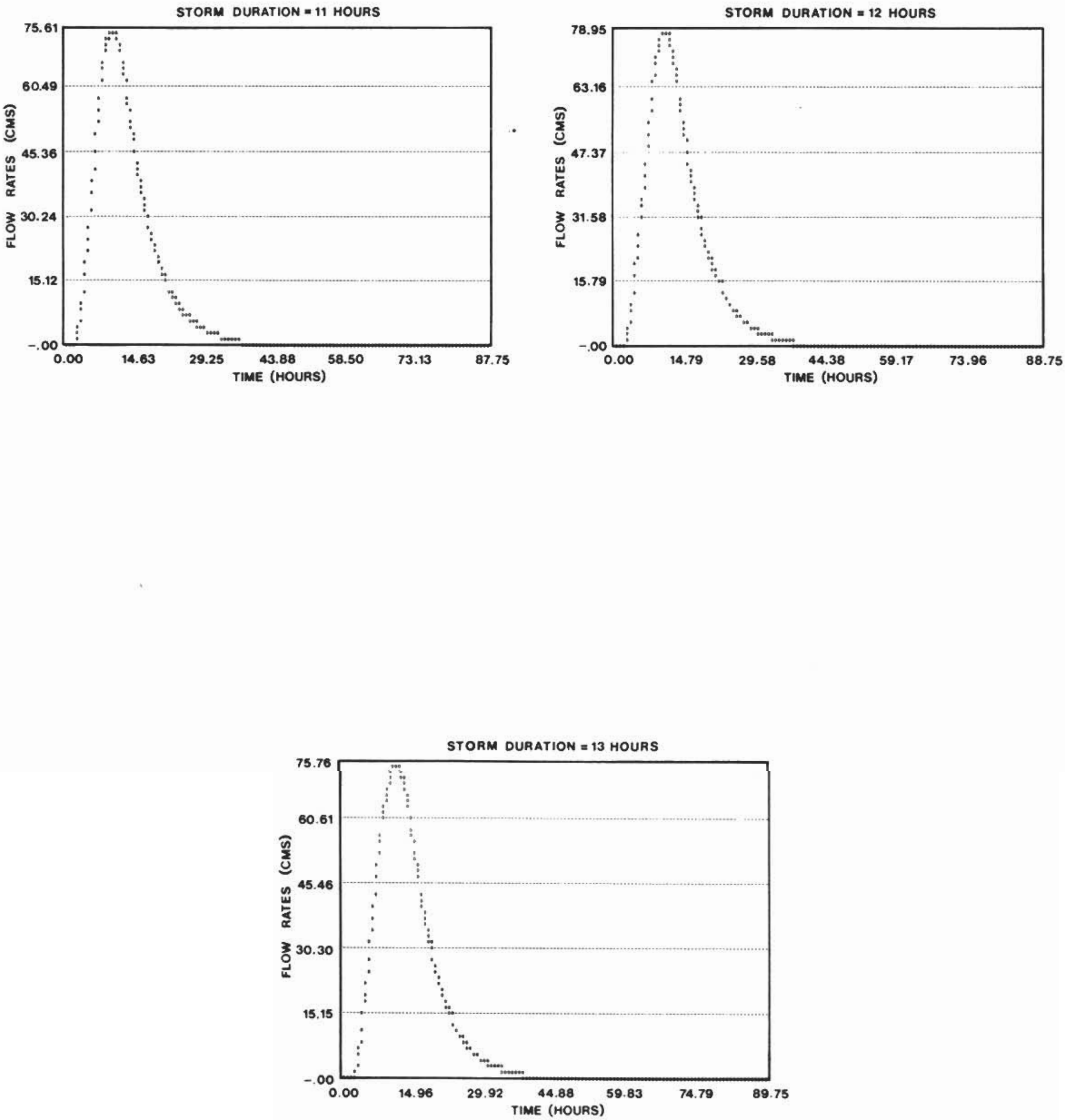


Figure 6.3

lower 95 percent confidence limit values. In keeping with the findings reported in Section 6.3, a twelve-hour storm duration was employed to produce the estimates.

TABLE 6.3

RAINFALL AMOUNTS FOR PEAK FLOW SIMULATION

Return Period	Rainfall Amount (mm)
20 year	76.8
20 year, upper 95 percent confidence limit	90.7
20 year, lower 95 percent confidence limit	62.9
100 year	94.4
100 year, upper 95 percent confidence limit	116.0
100 year, lower 95 percent confidence limit	72.7

6.5 Peak Flow Simulation

Prior to undertaking the peak flow simulations for the 20 and 100 year events, it was necessary to determine what antecedent soil moisture (AMC) condition to employ. As a result of an analysis of AMC for annual peak flows on the Waterford River, it was concluded that the very wet condition (AMC III) was appropriate for peak flow estimates. All annual peaks investigated were associated with AMC III conditions.

The calibration parameters presented in Table 4.3 (which reflect 1981 land use conditions) were employed.

The baseflows for the simulation runs were assumed to be equal to the average of the baseflows used in the calibration and verification events for AMC III. As shown in Table 6.4, the differences in baseflows among the events are not significant and these flows are certainly small in relation to peak flows. Therefore it was decided that use of the average baseflows would be appropriate.

TABLE 6.4

BASEFLOWS USED FOR CALIBRATION/VERIFICATION EVENTS

AMC III (m^3/s)

Date of Event*	Donovans	Mount Pearl	Kilbride
October 10, 1981	0.491	0.760	2.600
October 16, 1981	0.334	0.662	2.640
November 26, 1981	0.243	0.575	2.460
Average	0.356	0.666	2.570

*N.B.: The October 11, 1981 event could not be used in baseflow estimation since its baseflow was affected by the October 10 event.

****

TABLE 6.5

PEAK FLOWS

(m^3/s)

Location	Donovans		Mount Pearl		Kilbride	
Recurrence Interval:	20	100	20	100	20	100
Method Used	Years	Years	Years	Years	Years	Years
Simulation (HYMO) (baseflow included)	11.5***	14.7***	19.9	25.3	64.6	81.5
Single Station Analysis of Streamflow Data					59.6* to 65.1	75.3** to 101.

* 95 percent confidence limits are at least ± 28 percent

** 95 percent confidence limits are at least ± 32 percent

*** For the purposes of the flooding component of the Waterford River Basin Study (ref. 29), the peak flows presented in this table for Donovans had to be increased by 25 percent. Refer to Section 6.5 for some explanation.

A summary of the 20 and 100 year peak flows simulated for the three hydrometric stations along the Waterford River is presented in Table 6.5. Peak flow estimates obtained from a single station flood frequency analysis for Kilbride are also presented in Table 6.5 for the purpose of making a comparison with the simulated peak flows. The flood frequency analysis results and resulting frequency curve are presented in Appendix F.

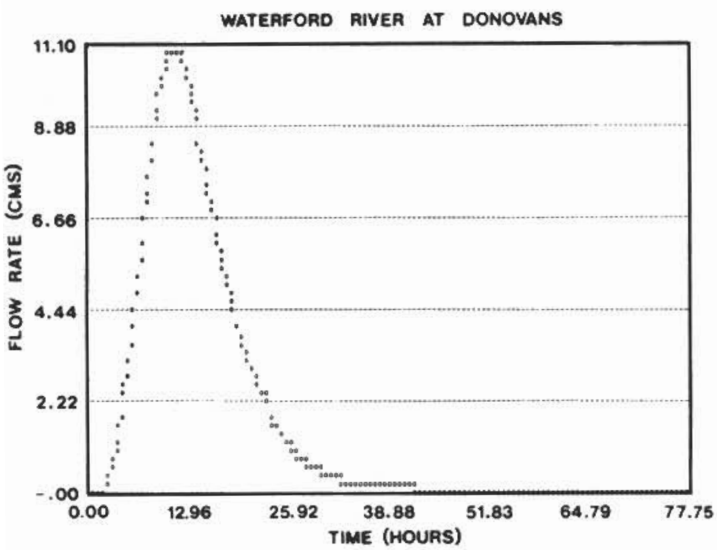
Figures 6.4 and 6.5 display the resulting hydrographs (excluding baseflow) for the 20 and 100 year return period simulated streamflows for the Waterford River at Donovans, Mount Pearl and Kilbride.

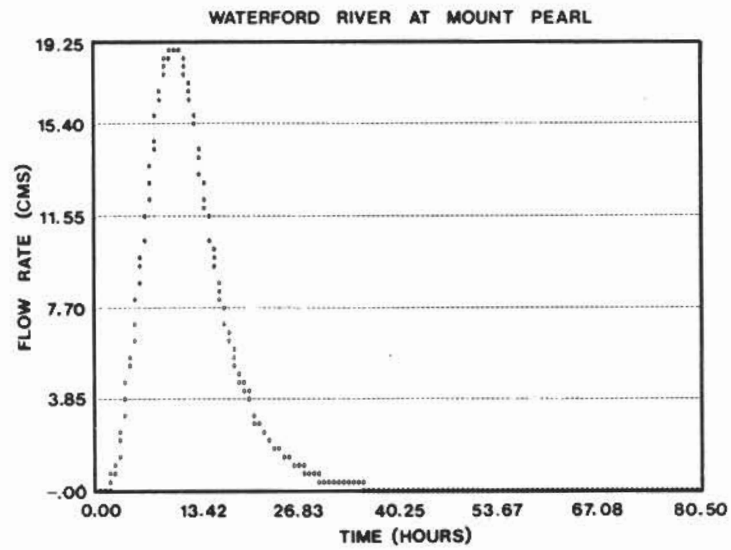
It should be noted that, for the flood component of the Waterford River Basin Urban Hydrology Study, the flows presented in Table 6.5 for Donovans had to be increased by 25 percent before simulating the flood profiles along the Waterford River near Donovans. This was necessary since: (i) the peak flows published for Donovans by the Water Survey were used to calibrate the hydraulic model; and (ii) the peak flows simulated by the HYMO model for Donovans were on average 25 percent less than those published by the Water Survey of Canada.

6.6 95 Percent Confidence Limit Flows

The upper and lower 95 percent confidence limit flows were also calculated based on the results of the intensity-duration-frequency analysis of St. John's Airport rainfall data. It was determined that the 12-hour upper 95 percent confidence limit rainfall totals for the 20 and 100 year recurrence interval storms were 90.7 mm and 116 mm,respectively, while the lower 95 percent confidence limit storm rainfall totals were 62.9 mm to 72.7 mm, respectively.

20 YEAR STORM HYDROGRAPHS
AMC III t_c=12 hours
(Excludes Baseflow)





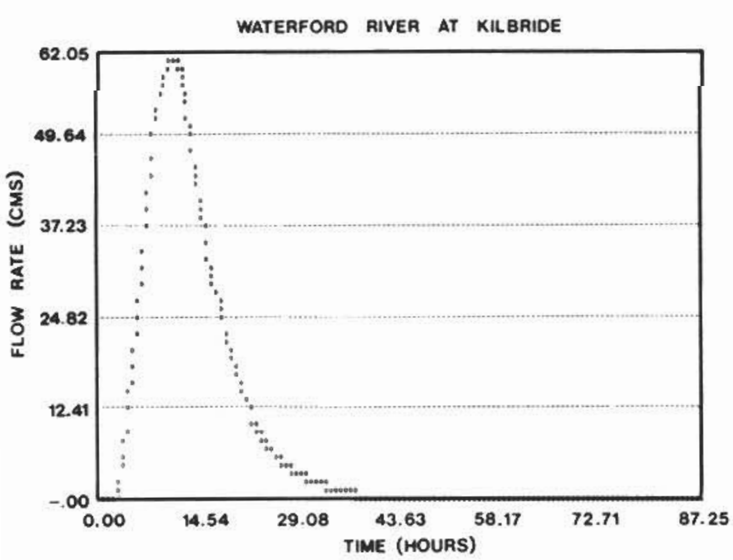
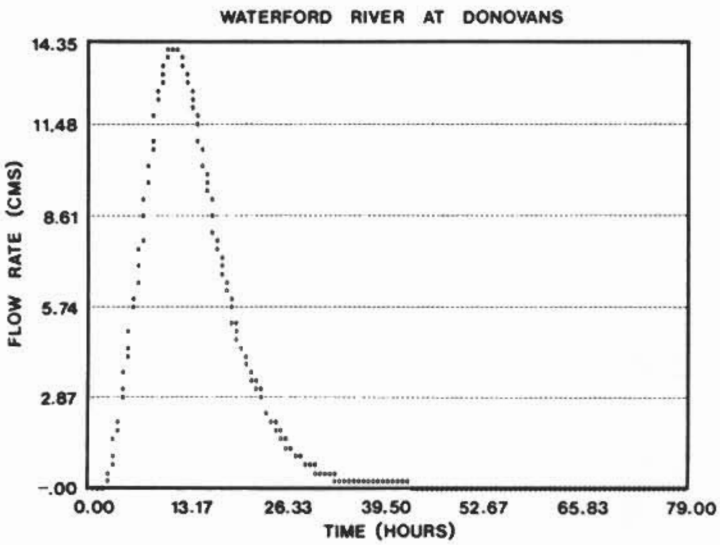
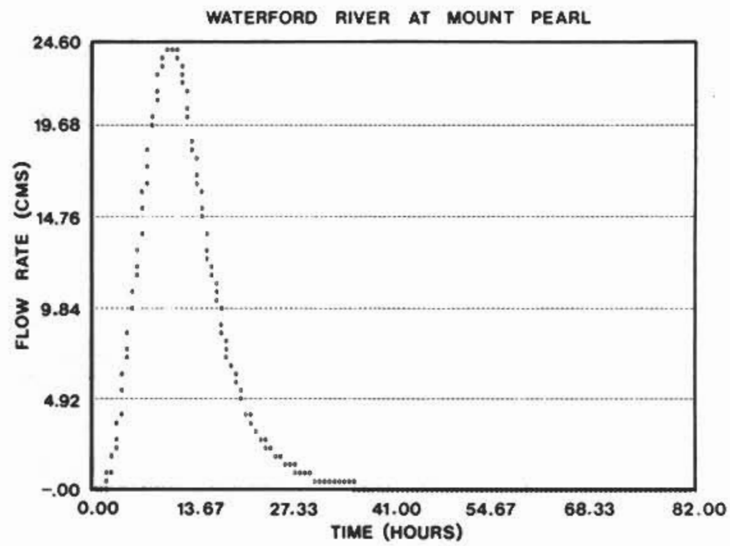


Figure 6.4

100 YEAR STORM HYDROGRAPHS
AMC III t_c=12 hours
[Excludes Baseflow]





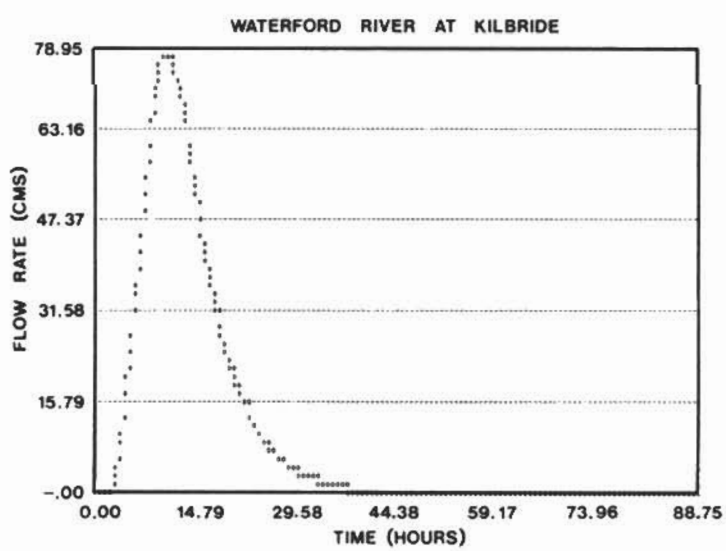


Figure 6.5

These rainfall totals were then distributed in time using the average of the rainfall distributions presented in Figure 6.2. The calibrated HYMO model (AMC III) for the Waterford River Basin was then used to generate the flows corresponding to the upper and lower 95 percent confidence limit 20 and 100 year recurrence interval storms. The resulting peak flows are presented in Table 6.6.

TABLE 6.6

95 PERCENT CONFIDENCE LIMIT PEAK FLOWS

(m^3/s)

(Baseflow included)

	Donovans	Mount Pearl	Kilbride
1:20 year			
Peak Flow	11.5	19.9	64.6
Upper 95% CL*	14.1 (+24%)	24.1 (+21%)	78.0 (+21%)
Lower 95% CL	8.6 (-25%)	15.8 (-21%)	51.4 (-20%)
1:100 year			
Peak Flow	14.7	25.3	81.5
Upper 95% CL	18.8 (+28%)	31.9 (+26%)	102 (+25%)
Lower 95% CL	10.7 (-27%)	18.7 (-26%)	60.7 (-26%)

* CL refers to confidence limit

7.0 FUTURE URBANIZATION SCENARIO

One of the objectives of the Waterford River Basin Study was to evaluate the effect of various development scenarios on peak flows and water levels on the Waterford River. This chapter provides an indication of what might happen to peak flows along the main stem of the Waterford River. The separate report of the flooding component of the study (ref. 29) provides an indication of the effect of future urbanization on peak water levels.

Urbanization usually results in increased peak flows and hence water levels, the major causes of which are an increasing amount of impervious area as well as a reduction in depression storage and the construction or improvement of drainage systems. The last two factors can actually have more significant impact than the first factor.

Reducing the amount of impervious area decreases the potential for infiltration and hence can greatly increase the amount of direct runoff during storms. The smoothing of land surfaces, including infilling of depressions (including swamps) reduces the capability of the area to naturally control the rate of supply to streams. Construction of drainage systems facilitates the rapid transmission of storm runoff.

As presented previously (Chapter 4), the average runoff curve number (CN) for the study area is 95 (Table 4.3) for wet antecedent conditions. This implies that, under wet conditions, almost all storm rainfall contributes to direct runoff. Even under dry antecedent conditions, the average CN is 87 (Table 4.4). Assuming that the calibration of the watershed model to the Kilbride hydrometric station is based on reliable data and hence the CN values are accurately estimated, then these high CN values imply that there is not likely to be much increase in storm runoff volume at the outlet of the study area, even with complete development.

If the streamflow data used to calibrate the watershed model were in error (and high) by 30 percent, then, according to the results of the sensitivity analysis (Table 4.5), the CNs for the calibration events would be lowered by about 7 percent. The high CN values, however, probably reflect the relatively thin and relatively impermeable overburden materials, and exposed bedrock within the study area.

In order to determine what the effect on flood peaks might be as a result of further development, one hypothetical future development scenario was developed (for the year 1991). That scenario, presented in Appendix G, was based on projecting the urbanization which occurred between 1973 and 1981 - the period for which information on urbanization was readily available from the land use component of this study. The resulting average CN for wet antecedent conditions was 97. This represents a 2 percent increase in the average CN for the study area, compared to 1981 conditions.

Peak flows are affected, not only by the volume of direct runoff, but also by the reduction in the time of concentration. Urbanization generally reduces the time of concentration of a basin. This can have a very significant effect on peak flows. While procedures exist (e.g. ref. 28) for estimating the change in time of concentration as a result of urbanization, they are often subjective and/or subject to considerable error. For this reason, in the 1991 future development scenario, a conservative (high) estimate of the impact of urbanization on time of concentration was made. This was implemented by reducing the times to peak (t_p) for each of the hydrologic units by 0, 15 or 25 percent as well as the recession constant (K), depending on the development expected, while assuming little change in the routing parameters in the main channel. This implies that new development would include storm drainage piping systems, the discharge from which would pass into the unaltered channels of the Waterford River and South Brook. The resulting parameters used in the hypothetical future urbanization scenario are presented in Table 7.1

Flow simulations (excluding baseflows) were then undertaken, assuming that the 12 hour rainfall would still produce the highest peak. Figure 7.1 shows the hydrographs for the 20 year recurrence interval storm hydrographs at Donovans, Mount Pearl and Kilbride. The peaks are 14.1 m^3/s, 23.6 m^3/s and 74.8 m^3/s for each station respectively (when baseflows are added on), compared to 11.5 m^3/s, 19.9 m^3/s and 64.6 m^3/s for present conditions. The 100 year recurrence interval hydrographs for the future development scenario are shown in Figure 7.2. The peak flows are 17.7 m^3/s, 29.5 m^3s and 93.3 m^3/s for Donovans, Mount Pearl and Kilbride, respectively, when baseflows are added on. For present conditions, the simulations for these three stations produced peak flows of 14.7 m^3/s, 25.3 m^3/s and 81.5 m^3/s. It can be seen that the hypothetical increase in the peak streamflows, although not very large, varies from 14 percent at Kilbride to 23 percent at Donovans. The hypothetical increases are largely related to the stated arbitrary reductions in the time to peak parameter. The peak flows resulting from the development scenario are summarized in Table 7.2. It is relevant to note that, as would be expected, a greater relative increase is projected for the less developed areas.

It was considered reasonable to investigate the impact of the peak streamflows (as reported by the Water Survey of Canada) being too high by, for example, 30 percent. The Water Survey of Canada has indicated that that would not be unreasonable for the hydrometric stations on

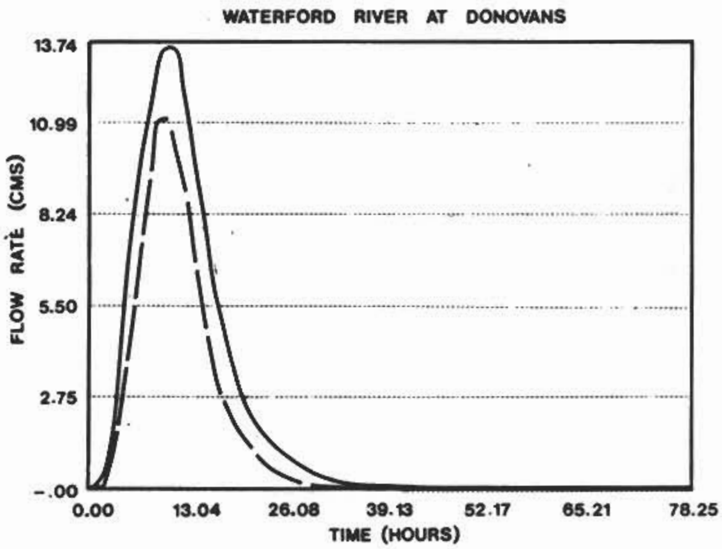
TABLE 7.1

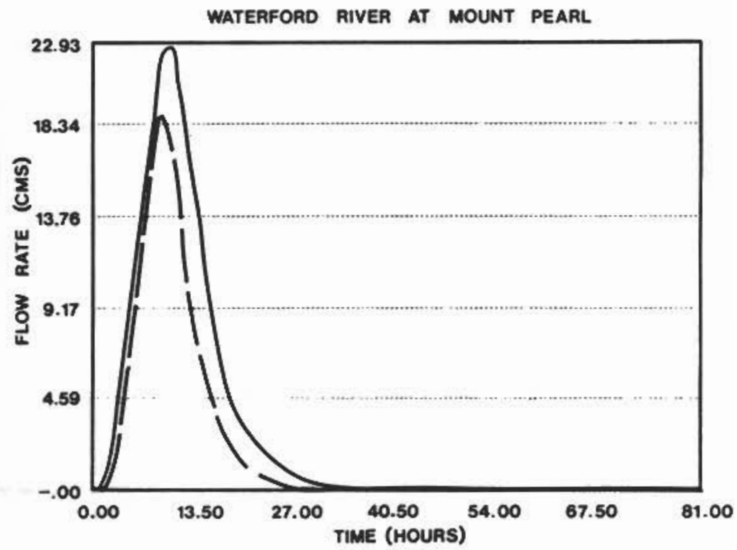
PARAMETERS FOR FUTURE DEVELOPMENT SCENARIO [1]

Hydrologic Unit	CN	K (hours)	t_p (hours)
301	90	13.29	5.96
302	95	4.27	3.03
303	96	4.79	3.35
304	95	5.15	3.68
305	99	3.03	2.08
306	97	2.45	1.86
307	100	2.48	2.00
308	100	3.39	2.16
309	98	1.17	1.27
310	100	4.41	3.06
311	98	1.22	1.26
312	100	5.35	3.40
313	97	4.71	2.89
314	97	2.45	1.91
315	100	1.99	1.90
316	98	5.43	2.78
320	92	9.81	6.71
321	97	8.07	5.66
322	100	6.60	3.23
323	98	3.77	2.86
324	99	5.43	3.15
325	97	4.34	2.70
326	97	2.26	2.17
327	92	0.97	0.84
Average:	97		

1. K and t_p were reduced from the values presented in Table 4.3, by 25 percent where significant development was expected, by 15 percent where some development was expected, and remained the same where none was expected.

20 YEAR RETURN PERIOD PEAK FLOWS FOR FUTURE DEVELOPMENT SCENARIO
[Excludes Baseflow]





Legend

——— FUTURE CONDITIONS
— — PRESENT CONDITIONS

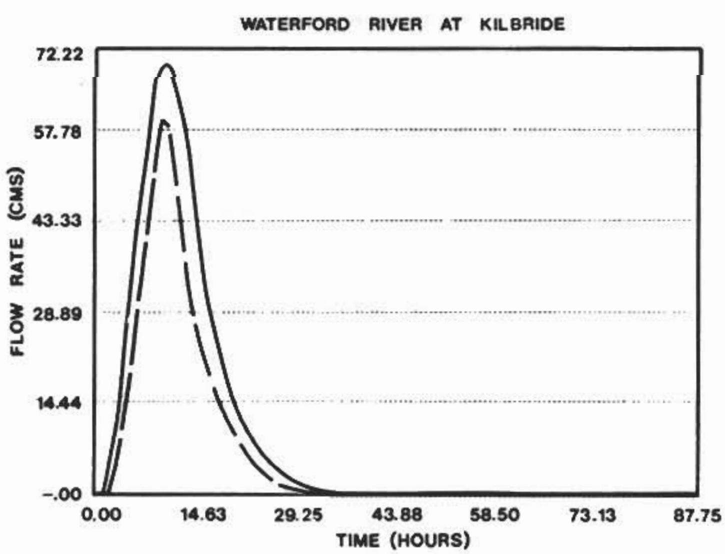
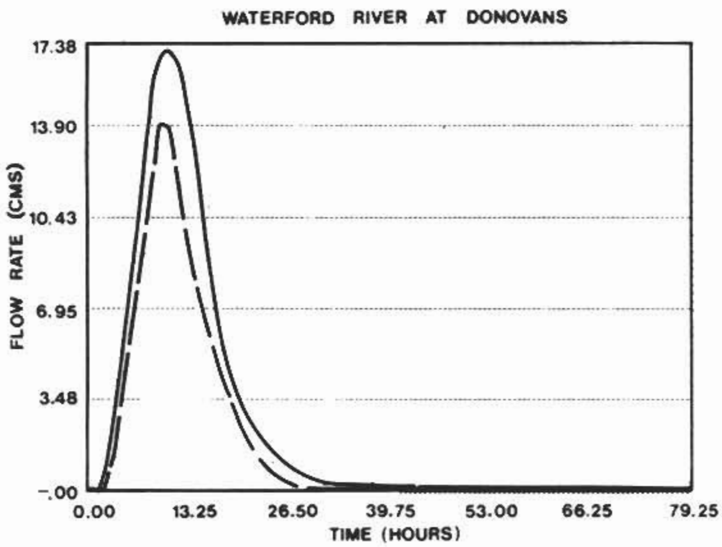
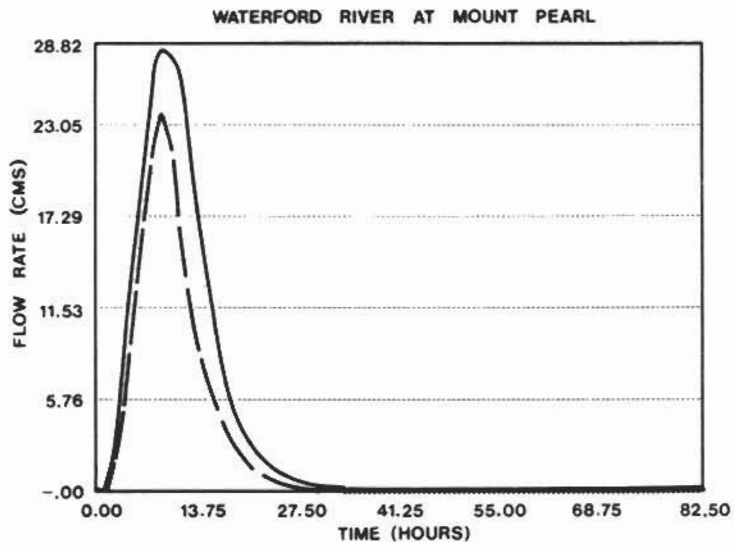


Figure 7.1

100 YEAR RETURN PERIOD PEAK FLOWS FOR FUTURE DEVELOPMENT SCENARIO (Excludes Baseflow)





Legend

——— FUTURE CONDITIONS

— — PRESENT CONDITIONS

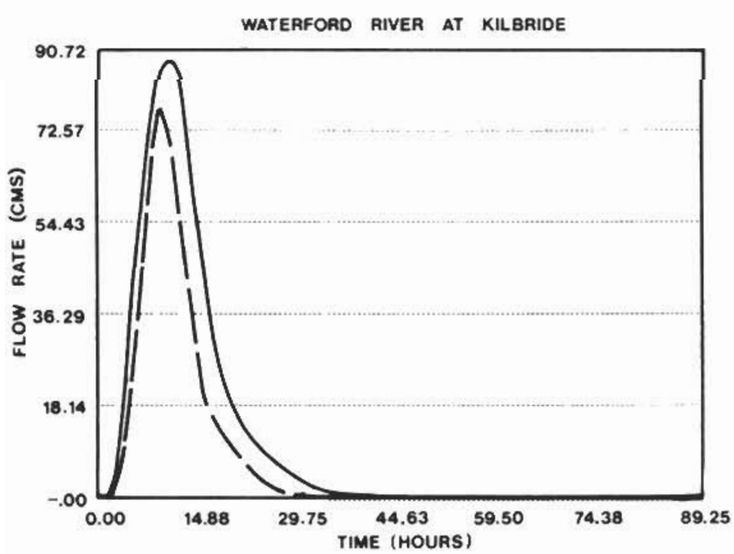


Figure 7.2

TABLE 7.2

SUMMARY OF PEAK FLOWS FOR FUTURE DEVELOPMENT SCENARIO

(m^3/s)

(baseflow included)

Recurrence Interval (year)	Donovans	Mount Pearl	Kilbride
20	14.1	23.6	74.8
100	17.7	29.5	93.3

the Waterford River. According to the sensitivity analyses, the CNs for the calibration events would then be lower by 7 percent (refer to Table 4.5).

The new average CN for the present level of development for AMC III conditions would be 88. In this case, the range of possible increases in average CN would be from 88 to 100 instead of from 95 to 100. Assuming a new average CN for the future development scenario of 94 (half way between 88 and 100), the resulting 20 year and 100 year return period peak streamflows would be approximately 12 percent lower than those presented in Table 7.2. Compared to the design flood flows presented in Table 6.5, however, they would be an average 7 percent higher (varies from 2 to 13 percent, depending on location and return period).

8.0 CONCLUSIONS AND RECOMMENDATIONS:

8.1 General

The principal objectives of the watershed modelling work using the HYMO model were:

(1) to determine if urbanization in the Waterford River Basin, above the hydrometric station at Kilbride, has significantly affected peak streamflows in the Waterford River.

(2) to provide an indication of the potential increases in peak flow which can be expected as a result of future urbanization, and

(3) to provide estimates of the 20 and 100 year recurrence interval peak discharges for use in the flooding component of the Waterford River Basin Urban Hydrology Study.

It was hoped at the onset of this study that the various influences on streamflows of specific land uses (e.g. commercial, residential, forest, agriculture) could be discerned through the use of the nested streamflow gauges on the Waterford River at Donovans, Mount Pearl and Kilbride. The drainage areas to these gauges are 11.4, 16.6 and 52.7 km^2, respectively.

It was also anticipated that the application of the HYMO model (as well as the ILLUDAS, SWMM and HSP-F models, the results of which will be presented in separate reports) would provide information which would prove useful when these and other hydrologic models are applied to other areas of the Province.

8.2 Conclusions

The amount of urbanized land in the study area is not great (as of 1981, approximately 17 percent). Of that amount, only a portion consists of pavement, rooftops and other impermeable surfaces directly connected to storm drains. Coupled with that fact, however, is the important aspect that the overburden material in the basin is very thin, and, for the large part, includes a relatively impermeable layer just below a fairly permeable surface layer. Accordingly, it is unlikely that urbanization has had, or could result in, a very significant (e.g. perhaps greater than 20 percent) increase in the volume of direct runoff associated with high streamflow peaks.

Urbanization can increase peak flows, however, without significantly changing the volume of direct runoff. Due to the smoothing of land features and the addition of man-made drainage systems, urbanization can alter the time of concentration, especially on very small streams with concentrated development.

No significant change in the time of concentration could be discerned between 1973 and 1981 by applying the HYMO model calibrated to 1981 land use conditions in conjunction with rainfall and streamflow data for two events in 1974. However, it must be realized that the amount of developed land increased from just 12 to 17 percent during that time.

Projections of the change in high peak flows for a fairly complete level of development in the basin suggest the possibility of an increase in the order of 14 percent at Kilbride to 23 percent at Donovans. This, however, is based on the arbitrary assumption of a maximum 25 percent reduction in both the time to peak (t_p) parameter and recession constant (K) for each hydrologic unit (sub-basin). Review of reductions in time to peak reported for basins with similar geology, topographic features and land use may provide a better estimate of the potential influence of this parameter (e.g. ref. 28).

Estimates of the 20 and 100 year recurrence interval flood flows, along with the associated 95 percent confidence limits, were prepared for the sites of the hydrometric stations near Donovans, Mount

Pearl and Kilbride. Those values, which are summarized in Chapter 6, were based on the level of development existing in 1981.

8.3 Potential Influence on Conclusions of Errors in Rainfall and Streamflow Data

In any study of this nature, an amount of uncertainty is introduced due to, among other things, errors in the data base. In order to determine the sensitivity of the conclusions, presented above, to such errors, some testing of the calibrated HYMO model was undertaken. This included, among other things, its sensitivity to plausible errors in rainfall and streamflow data.

As noted in Chapter 3, all six standard rain gauges established for this study recorded, on average, less rainfall for storm events than was recorded at the permanent standard gauge at the Canada Department of Agriculture (CDA) Research Centre, located in the study area. The amounts of undercatch varied, for each site, on average, up to 21 percent. These findings are supported by a review conducted by staff of Environment Canada's Atmospheric Environment Service, which indicated that several of the temporary gauges would experience some undercatch due to the influence of vegetation and buildings. This problem was partially accounted for by employing adjustment factors to all data used in calibration and verification so that, on average, the amount of rainfall recorded at the temporary gauges would be no less than that recorded at the CDA site.

A sensitivity analysis indicated that peak flow simulations are, as would be expected, very sensitive to errors in rainfall. Furthermore, since many of the temporary rain gauges are located on higher land approaching the steeper edges of the study area, it would be expected that, if anything, more rain would have fallen than the adjusted data may indicate. The net effect of this is that runoff coefficients (CN) may have been overestimated somewhat. This would translate into a somewhat greater potential for increases in direct runoff.

The modelling work indicates that significant problems were encountered in the simulation of streamflows at Donovans and, to a much lesser extent, at Mount Pearl. The problem may be related to undercatch by the standard rain gauges, as discussed above, or, which is more likely, a poorly defined streamflow rating curve for high flows. During the course of this study, the rating curve for Kilbride was adjusted downward by the Water Survey of Canada by as much as 30 percent for high flows, as a result of recent field measurements. This type of uncertainty in rating curves is not unreasonable when peak flows, of the magnitude used in the present study, have not been measured in the field and when the metering section contains shifting gravel beds. This implies then that it is not unreasonable to conclude that the peak streamflow data for Donovans and Mount Pearl may be in error, and low, by the amount by which simulated and observed streamflows differed for these two sites (25 percent and 10 percent respectively).

The impact of streamflow errors of the magnitude of those reported in the previous paragraph (30 percent), on the calibration of the HYMO model was evaluated. It was determined that, for wet antecedent conditions, the average CN for the hydrologic units would be about 7 percent lower. The net effect of this requirement to lower CN would be to produce significantly (30 percent) lower design peak flows for the present state of development. For future hypothetical development scenarios, however, somewhat significant increases in peak flows would be possible relative to design flows.

8.4 Recommendations

During the course of this project, a detailed surficial geology map was produced for the groundwater component of the Waterford River Basin Urban Hydrology study. Initial estimates of the runoff coefficients (CN) for the HYMO model were developed, however, using the 1:100,000 scale soils mapping available at the time. As a result of the calibration procedure, it was found that significantly higher values of CN were warranted for the study area. This requirement was supported by the results of the detailed surficial geology survey. For this reason, it is highly desirable, if the HYMO model is to be applied for a location with no streamflow records, that detailed surficial geology information be provided.

Initial simulations of storm streamflows for the study resulted in hydrographs that peaked much too early and with peak flows much greater than actually recorded. Those initial simulations were based on the equations for estimating the hydrologic parameters (K and t_p) suggested for use with the HYMO model for rural watersheds in the southern United States. Projects undertaken in Canada have indicated that the estimates of K and t_p, provided by the HYMO equations, often had to be increased by up to 300 percent. Since even greater increases were required for this study, it is reasonable to conclude that HYMO should not be applied to basins, for which there are no streamflow and precipitation data available for calibration, without the existence of a better procedure for estimating the recession constant (K) and time to peak (t_p) parameter.

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