Fenco Newfoundland

FENCO NEWFOUNDLAND LIMITED
189 WATER STREET, P.O. BOX 248
ST. JOHN'S, NEWFOUNDLAND, CANADA A1C 5J2 · TELEPHONE: (709) 754-1480
TELEX: 016-4936 · CABLE: FENCOENG

48160-C2-380000

June 6, 1988

Canada-Newfoundland Flood Damage
Reduction Program
c/o Department of Environment and Lands
Confederation Complex
4th Floor West Block
P.O. Box 4750
St. John's Newfoundland
A1C 5T7

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

RE: WATERFORD RIVER AREA - HYDROTECHNICAL STUDY

Dear Dr. Ullah:

We are pleased to submit 50 copies of our final report of this study. Accompanying this submission are 10 copies of our technical appendix and a complete set of flood risk maps for the study area.

This report describes the results of field surveys and our studies to identify flood-prone areas and flood damage reduction alternatives in the study area - from St. John's Harbour to Donovans Industrial Park. The first Chapter presents a concise summary of our findings and subsequent chapters provide the details.

It has been a distinct pleasure to work with you and the other members of the Flood Damage Reduction Technical Committee during this project. We particularly wish to thank you, Ms. E. Langley, and Mr. R. Picco for your considerable assistance and constructive comments throughout the course of this study.

We look forward to being of service to you again in the near future.

Yours very truly,
FENCO Newfoundland Limited

Eric Gray

Eric Gray, P. Eng.
President

:hc
Encl.

Lavalin

TABLE OF CONTENTS

TABLE OF CONTENTS (CONT'D)

TABLE OF CONTENTS (CONT'D)

LIST OF TABLES

LIST OF FIGURES

LIST OF FIGURES (cont'd)

HYDROTECHNICAL STUDY
WATERFORD RIVER AREA

1.0 SUMMARY OF FINDINGS, CONCLUSIONS AND RECOMMENDATIONS

1.1 Introduction

The Waterford River, which passes through growing urbanized areas of Mount Pearl, Donovans Industrial Park and the western portion of St. John's, has been the subject of several recent studies. These studies examined the hydrology, water quality, surficial geology, effect of urbanization in the basin, and other factors relating to the potential for flooding along the course of the river.

One previous report was the "Flood Study" technical report of the Urban Hydrology Study of the Waterford River Basin, prepared by Newfoundland Department of Environment and Environment Canada. The report examined three flood-prone sections along the river, and pointed to the need to designate flood risk areas in the studied reaches.

The following report describes the hydrotechnical study leading to the delineation of flood risk areas for the entire river from Donovans Industrial Park to St. John's Harbour. This project began in February 1987 with a streamflow monitoring program, and collection and review of previous studies and hydrometric data. This information was then combined with field surveys along the river to develop a watershed model which provided accurate information about historic flood flows. These flows were evaluated to enable projection of the 1:20 year and 1:100 year flood levels and preliminary evaluation of flood damage reduction alternatives.

1.2 Hydrology Summary

Historical records since 1934 report about 40 incidents of flooding which have resulted in loss of life and a significant number of bridge washouts. Many of the bridges have been replaced or repaired but recent urban development and pressures for future development along the waterway are combining to increase the potential for flood problems. Generally, flooding in the watershed results from about two days of rainfall totalling 70mm or more.

Progressive urban development in the watershed has changed its surface runoff characteristics. Hence, historical records from times when there was less development, and regional flood flow statistics derived from undeveloped watersheds may not provide realistic estimates of flood flows in the Waterford River. A mathematical modelling approach employing the current hydrologic system was consequently selected herein for determining flood flows. The model is called QUALHYMO.

The hydrologic model was prepared using:

- physical watershed characteristics derived from detailed topographic mapping, field surveys, sewer network drawings, and surficial and bedrock geology reports

- land use data maps for 1984, updated by our team using recent aerial photography and land use mapping by the City of St. John's

- a continuous 28 year record of hourly data assembled from precipitation and temperature data monitored at St. John's west CDA and Dalhousie Crescent in the study area, and the nearby St. John's Airport Station. Daily snow depth data was obtained from St. John's West CDA for 1964 to date

Hourly streamflow data for verifying the model simulations of precipitation runoff were obtained from three monitoring sites in the study area. One site at Kilbride has monitored flow since 1973 and another site at Mount Pearl has data since 1981. Data from the third site near Donovans is considered unreliable and was only employed qualitatively. There was no streamflow monitoring for the 15 years between 1959 and 1973 when hourly rainfall data is available to simulate streamflow.

Streamflow simulations were conducted using two versions of the QUALHYMO model. The first version was prepared to identify the periods of peak flow in the past when streamflow peaks were not monitored (1959-73). The second model was developed to give accurate estimates of peak flows in these periods. Overall, the objective was to develop a long period (28 years) of streamflow data - based on today's land use - in order to estimate 1:20 and 1:100 year flood flows which could now occur in the watershed.

Model calibration and verification gave excellent results for the important parameters of runoff peak, runoff volume, peak flow timing and flow recession following the peak. In all, these gave considerable confidence about the model's ability to simulate streamflows from precipitation inputs.

The verified model was then employed to simulate peak flows for the 28 year period of precipitation record (15 more years than is currently available from streamflow monitoring). The peak flows from all the years were then subject to a frequency analysis to determine the 20 year and 100 year return period flood flow peaks. The Three-Parameter Lognormal Probability distribution was the best fit to the data, and was employed to give flood flow estimates at 18 locations along the river. The following lists these flood flows at four locations of interest:

Location of	Flood Flow Estimate (m^3/s)	
Flow Estimate	1:20 year	1:100 year
Donovans Park (downstream bridge)	16.9	22.8
Mount Pearl (Commonwealth Ave.)	26.2	36.2
Kilbride	83.3	118.0
River Mouth	102.0	145.0

Additional study was undertaken to determine the basin's response to major rainfall storms. Runoff from the 1:20 year and 1:100 year 12-hour storms was simulated and found to be similar, but slightly less, than the runoff generated by the previous approach. This indicates that rainfall storms result in very significant flows, but that combinations of rainfall with snowmelt combine statistically to produce slightly higher 1:20 year and 1:100 year flood flows. These latter, higher flows were selected for determining flood profiles in the river.

1.3 Hydraulics Summary

The purpose of the hydraulic investigations was to derive the 1:20 and 1:100 year open water surface profiles along the study reach using the results of the above hydrologic information.

A mathematical modelling approach was also selected for this work. The selected model, called HEC-2, has been successfully used in similar applications throughout North America and was used in earlier Flood Studies conducted as part of the Urban Hydrology Study of the Waterford River Basin (1986).

The principal data for the HEC-2 model are cross sections of the river channel and dimensions of bridges and other structures in the river. These data were obtained by field surveys in mid-1987, surveys conducted from 1981 to 1983 for the Urban Hydrology Study, and bridge drawings and topographic mapping. In all, 209 cross sections were employed in the model.

A water level monitoring program was established in early 1987 in order to obtain levels and flows for calibrating the model. This program included snow surveys because it was anticipated that the 1987 snowmelt period would produce a large peak flow. High flows did not materialize in the spring, summer or fall and the level monitoring was discontinued in November.

Despite the absence of a high flow period in 1987, the results of the 1987 monitoring were employed to calibrate the model because the data was available for the full study reach. The calibration was successfully completed using values of channel roughness and other parameters within the range of anticipated values.

Model verification was undertaken using water level data from monitoring conducted for the Urban Hydrology Study, Flood Study Report (1986). This monitoring focussed on the Kilbride, Mount Pearl and Donovans area and three flow periods were taken from that data to verify the model.

The model verification demonstrated that computed water levels were in close agreement with observed levels. It was concluded that the model is capable of accurately predicting open water surface profiles for 1:20 and 1:100 year flood flows.

Sensitivity analyses were then undertaken in order to test the sensitivity of computed water levels to variations in the model parameters. The parameters changed in the model were channel inverts (± 0.15m), discharge (± 30%), starting water level (± 0.15m), channel roughness (± 10%) and expansion and contraction coefficients. The model was found to be insensitive to these variations in the parameters, lending additional confidence to its use.

The final step before computing the river flood levels involved analysis of water levels at the river mouth - the starting point for the backwater modelling. The mean high tide level (0.62m GSCD) was selected as an appropriate starting level as it is quite possible that a high tide would be present during the course of high flows.

Instantaneous high water levels at the mouth were also subjected to frequency analyses to determine the 20 year and 100 year levels. It was determined that these levels are: 1.45m GSCD for the 1:20 year case and 1.68m for the 1:100 year case. Since river flooding or high levels at the outlet can occur at the river mouth, the highest of these two levels determines the flood hazard area at the mouth.

The calibrated/verified HEC-2 model was run in two final simulations to produce 1:20 year and 1:100 year flood levels at all of the cross sections in the model. These levels were plotted on six, 1:2500 scale topographic maps (prepared for the Canada-Newfoundland Flood Damage Reduction Program) to delineate the flood risk area.

Photo-reductions of portions of the maps are contained in Chapter 7 of this report. They indicate that there are a number of individual structures at the edge of the flood plain along the course of the river, and potential flood damage to buildings at Leslie Street Bridge, Symes Bridge, Kilbride (Corpus Christi Church area) just upstream of Brookfield Road Bridge, and at the Fiberply Plant area at Donovans Industrial Park. Other locations (e.g. Mt. Pearl) have extensive areas which are within the flood plain, but these areas contain no structures.

It is evident that the lessons learned from previous damaging floods have been applied through the years. Historical reports confirm that washed out bridges have been replaced by larger structures, which can now safely pass flood flows (e.g. the high level arterial bridge replacing Job's Bridge Crossing).

Similarly, it appears that channelization at the river mouth and channels in the newer portions of Donovans Industrial Park, for example, are reasonably sized to carry flood flows.

In total, about 19 or 20 structures are contained in the 1:100 year flood zone along the Waterford River.

1.4 Summary of Flood Damage Reduction Alternatives

The final step of the report was preliminary identification of flood damage reduction alternatives which could be employed along the Waterford River. Proven ways to reduce flood damages can be grouped into:

- those which accept that high water levels will occur from time to time but mitigate flood damages by floodplain zoning, acquisition of properties at risk, or flood proofing of structures.

- those which attempt to reduce the flood level by structural means such as flood control dams, channelization or dyking, or bridge opening expansions.

It is always desirable to pursue alternatives that are economically justifiable as well as feasible. As the 20 buildings in the flood zone of the Waterford River are distributed along the entire study reach, the latter group of options would be very expensive to implement and would likely present highly unattractive benefit - cost ratios. Hence, our focus for damage reduction was the first group of alternatives.

Brief descriptions of each option is given in Chapter 8 of the report where it is concluded that:

1) the flood elevations determined in this study be adopted by municipalities along the river, so that developable areas which are prone to flooding can be zoned as flood risk areas or for special, structural design considerations (e.g. flood proofing by elevation).

2) five existing houses in the flood zone be considered for flood proofing by elevation.

3) three buildings be examined for the installation of permanent or automatic closures at low level flood entry openings.

4) the potential for protecting five buildings in the flood zone by using low berms, fill or road regrading be examined.

5) the area near Corpus Christi Church in Kilbride (containing six structures in the flood zone) cannot be easily/economically protected by major works, such as a flood wall. It was concluded that further flood damage reduction investigations should include examination of:

- closures for low level openings at the church
- flood proofing by raising other buildings on piers or reinforced walls
- gradual acquisition of some of the most damage prone buildings

In conclusion, there appear to be relatively inexpensive damage reduction options which may be applied to reduce future and existing problems along the Waterford River. Most may be carried out by the individual owners, but zoning to minimize future developing problems rests in the hands of local government.

2.0 INTRODUCTION

Floodplain lands adjacent to rivers and streams have always represented attractive centres for development. Such developments have historically occurred because of the use of rivers as transportation routes, sources of power and water, and because much of the best agricultural land is located within floodplains. The resulting conflict with the river at flood times has led to a variety of approaches to controlling flooding. The earliest records of such attempts in North America to modify the relationship between man and floods stretch back to 1617 when early French settlers used dykes to protect areas for agricultural purposes.

In light of trends towards increases in flood disaster assistance payments, greater pressure for floodplain development because of increasing urban population coupled with escalating land costs, and the potential environmental problems associated with structural flood control measures such as dams, it has been recognized that a new and more comprehensive approach to floodplain management is required. Policies based on a full evaluation of both non-structural alternatives, such as restrictions on flood vulnerable development in high flood risk areas, as well as traditional structural approaches are necessary.

Given this increasing awareness, and in view of the potential for loss of life and damages resulting from floods, the Province of Newfoundland and the Government of Canada entered into a "General Agreement Respecting Flood Damage Reduction" on May 22, 1981. The objective of this Agreement is to reduce the potential flood damages on floodplains along the shores of lakes, rivers and the sea. This Agreement also recognizes that the potential for flood damages can be reduced by controlling the uses made of flood hazard areas. This involves the identification and delineation of flood prone areas and ultimately the designation of these areas wherein only certain conforming developments could take place.

As part of this initiative, a flood risk mapping program is being undertaken in Newfoundland. The mapping of a flood risk area consists of four main components: hydrology, hydraulics, topographic mapping and public information. The main purpose of this investigation is to provide the hydrologic and hydraulic components for the identification of flood prone lands for the main branch of the Waterford River Basin from Donovans Industrial Park to the river mouth in St. John's.

2.1 Study Area

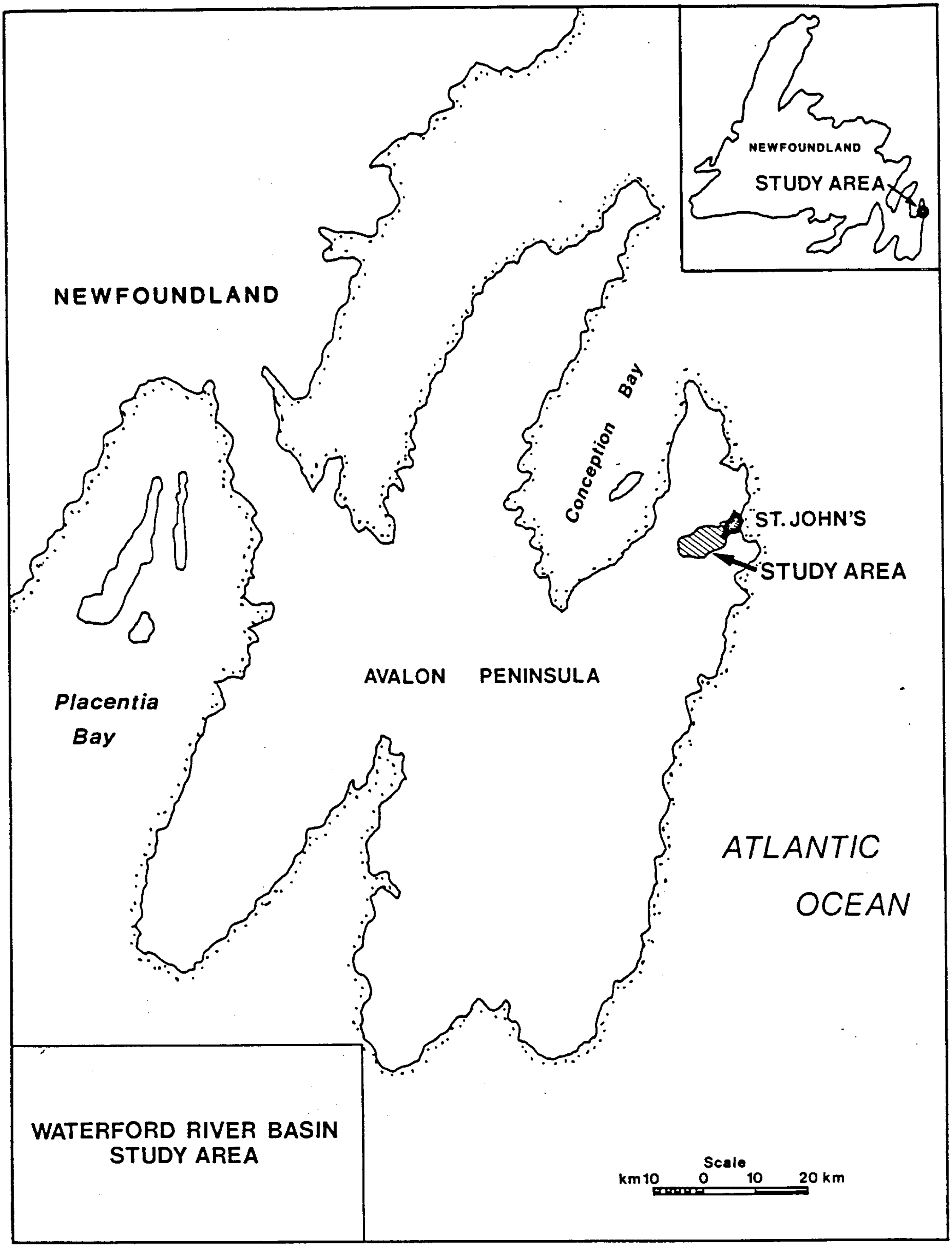
The study area is located in the northeastern portion of the Avalon Peninsula. The eastern extremity of the river basin drains the western part of the City of St. John's, and the western edge of the basin is about 13 km west-southwest of St. John's Harbour. Figure 2-1 outlines the drainage area in relation to the Avalan Peninsula and Figure 2-2 shows the area in more detail.

There are two principal branches of the river. The main branch extends westward from the harbour, through Mount Pearl and further west toward headwaters near Bremigens Pond. This branch and a small tributary which parallels the Trans-Canada Highway (through Donovans Park) are the focus of this study. South Brook, which joins the main branch below Bowring Park at Kilbride, is part of this basin-wide hydrology study but is not mapped herein for identification of flood prone lands.

2.2 Study Objectives

The purpose of this study is to examine the flooding potential from the western boundary of Donovans Park to the river mouth, and to provide reliable estimates of open water flood profiles for the 1:20 year and 1:100 year floods.

The objectives of the study were prepared by the Technical Committee of the Canada-Newfoundland Flood Damage Reduction Program. Their requirements are briefly summarized below:



WATERFORD RIVER BASIN
STUDY AREA

FIGURE 2-1

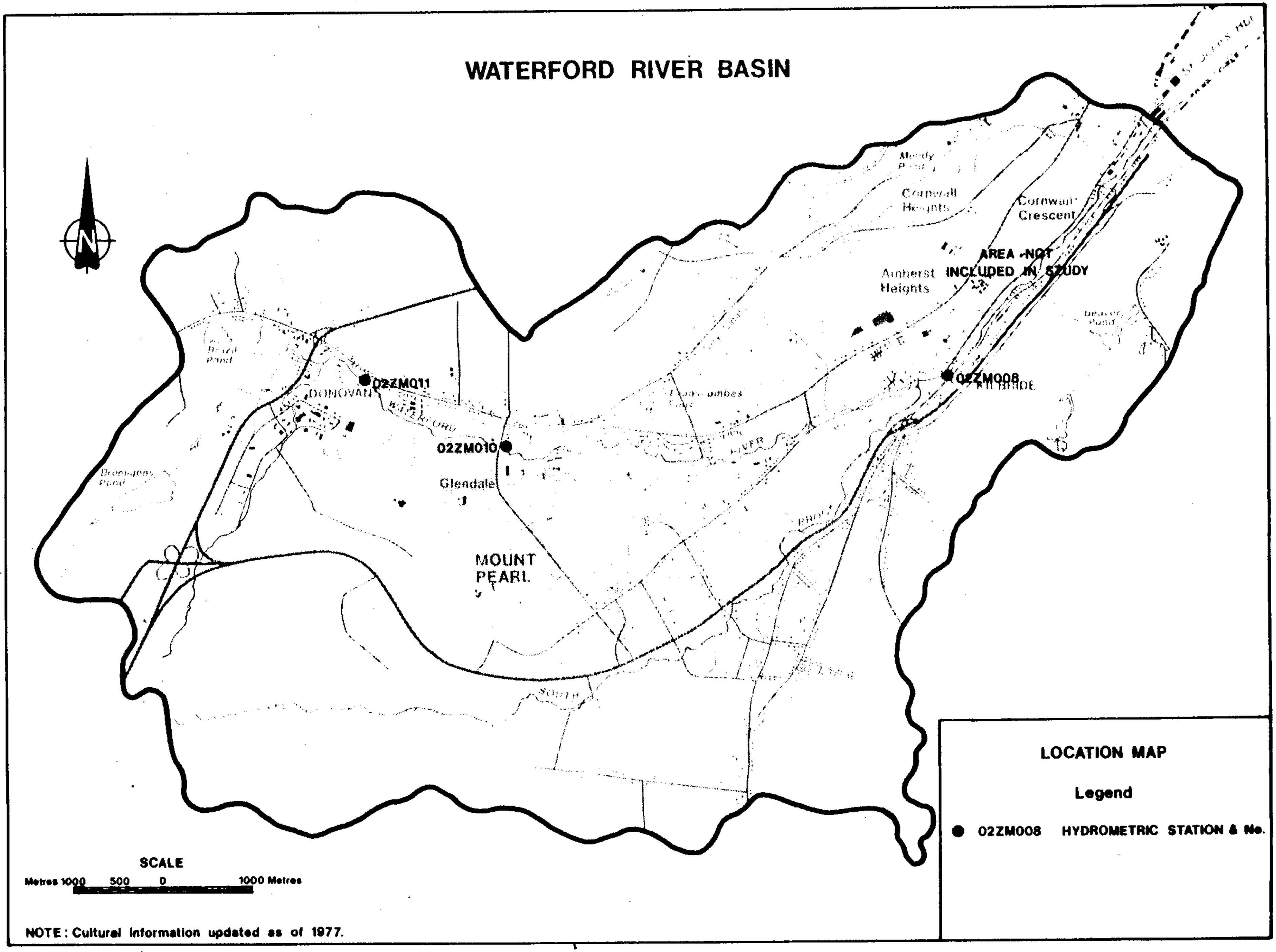

WATERFORD RIVER BASIN
N
Cornwall Heights
Cornwall Crescent
AREA NOT INCLUDED IN STUDY
Amherst Heights
02ZM011
DONOVANS
02ZM010
Glendale
MOUNT PEARL
02ZM008
RIVER
LOCATION MAP
Legend
02ZM008 HYDROMETRIC STATION & No.
SCALE
Metres 1000 500 0 1000 Metres
NOTE: Cultural information updated as of 1977.

FIGURE 2-2

1. Assemble and review existing information and previous studies to gain a good appreciation of factors which affect flood flows in the study area.

2. Prepare an appropriate mathematical model of the river basin to reliably simulate the flood flow hydrology of the river:
   - considering effects of urban development
   - employing historical hydrometeorological data to determine watershed conditions and the 1:20 and 1:100 year flood flows

3. Prepare a mathematical model to provide flood flow hydraulic profiles, using:
   - field surveys to accurately characterize the geometry of the river channel
   - water level data collected to calibrate and verify the model
   - information on tides, changes in channel morphology, and man-made features such as structures, infilling and retaining walls.

4. Produce 1:20 and 1:100 year return period open water flood profiles following sensitivity analysis of various model parameters. Plot the flood lines on large scale maps provided by the Technical Committee (1:2500 scale with 0.5 m contours).

5. Suggest remedial measures which may be appropriate for future studies to reduce flood damage potential in the study area.

2.3 Study Approach

After project initiation on 25 February 1987, the flow of the project proceeded with the following activities:

- a field reconnaissance and several snow surveys to gather hydrologic information pertaining to snowmelt and the general physiographic characteristics of the watershed.

- monitoring of flood elevations with crest gauges put in place at five locations within the main study reach. Several gauges were maintained in the study area from 26 March 1987 until mid-November 1987.

- extensive review of all pertinent background records and reports related to hydrology, hydraulics, and flooding. This included a considerable number of reports from previous hydrotechnical studies conducted by the Provincial and Federal Governments during the "Waterford River Basin Urban Hydrology Study" (1980 - 1985)

- hydrometric data analysis. Hourly precipitation and snow accumulation at monitoring locations within the watershed and at St. John's airport were assembled for a 28 year period from 1959 to 1986.

- development of several hydrologic models to provide streamflow estimates from the precipitation data. The first model used the hydrometric data to give information on soil moisture conditions in the watershed since 1959. The second model grouped/lumped the smallest subcatchments into larger units to simulate streamflow and provide the dates of historical high flow periods. The final model employed all of the smaller discretized subcatchments to provide accurate estimates of peak flows in the high flood periods identified in the lumped modelling.

- model calibration and verification to ensure that the hydrologic model was correctly simulating streamflows. Flows monitored by the Water Survey of Canada were employed to check results at three locations along the main branch.

- computation of flood flow estimates based on frequency analyses of the peak flows generated by the hydrologic model (1959-1986). These 1:20 and 1:100 year design flows were found to be similar but slightly larger than flood flows generated by 1:20 and 1:100 year rainfall storms.

- flood profiles within the study reach were obtained by using the HEC-2 backwater model, established from field-surveyed cross sections and topographic mapping prepared for this study. The model was calibrated prior to calculation of final 1:20 and 1:100 year profiles using water level observations obtained during flow monitoring in 1978, 1982, 1983 and 1987.

- sensitivity analyses were carried out to determine the impact of variations in flow, Manning's roughness, expansion and contraction coefficients and downstream conditions on levels in the study reach.

- the flood risk areas for the 1:20 and 1:100 year flows were delineated on topographic mapping of the study reach. The location of the field surveyed sections was also shown on these maps.

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

The following sections of this main report discuss these tasks and the results of each activity in greater detail. A second volume, which is a compendium of technical notes, survey data and computer outputs, is available for review by the interested reader from the Canada-Newfoundland Flood Damage Reduction Program office at Newfoundland Department of Environment and Lands, St. John's.

3.0 HISTORICAL OVERVIEW

3.1 Introduction

The history of flooding on the Waterford River between Donovans' and St. John's Harbour has been drawn together from a variety of sources for this study. Principal use was made of:

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

- a listing of similar information for flood events from 1951 to 1986, prepared by the Technical Committee of the Flood Damage Reduction Program as part of the Terms of Reference of this study.

- newspaper reports giving additional details of flood events. These reports are archived on microfiche at Memorial University.

- reports of recent flood events compiled by the Newfoundland Department of Environment.

3.2 Historical Flooding

It is evident from review of the literature that flooding in the Waterford River Basin has been frequent and damaging. There has been loss of life, numerous washouts of bridge crossings, extensive erosion and repeated flooding of certain areas over the past years. As noted in Table 3.1, there have been approximately 40 flooding events in the basin over the 54 year period from 1934 to late 1987.

TABLE 3.1

SUMMARY OF HISTORICAL FLOODING AND HIGH FLOW PERIODS - 1934 TO PRESENT WATERFORD RIVER BASIN

Date	Comment
1934 Oct. 12-14	· about 94 mm rainfall in 3 days caused flooding from Symes Bridge to Donovans area
1941 Aug. 02-04	· heavy rainfall resulted in local flooding but apparently none from Waterford River
1942 Jan. 29-30	· almost 128 mm rainfall in 2 days resulted in flooding along Water Street at the mouth
1942 Oct. 04	· 100.8 mm rainfall on this day caused flooding of suburban ponds and outlying rivers
1942 Dec. 02-03	· 63.5 mm rainfall reported in 3.5 hr caused flooding along Southside Hills and at the river mouth
1944 Dec. 06-07	· 87 mm rainfall in 2 days cause flooding along the river and mud/gravel slides down Southside Hills
1945 Jun. 23-24	· almost 100 mm rainfall in 2 days led to flooding on the outskirts of St. John's
1946 Jul. 27-29	· 121 mm of rain on 27 July resulted in flooding in Bowring Park and damage/destruction of some bridges
1946 Dec. 01-02	· over 68 mm rainfall in two days led to extensive flooding along the Waterford River
1948 spring	· uncertain causes (perhaps rainfall with snowmelt) resulted in railway washout at Symes Bridge
1948 Sep. 14-15	· over 117 mm rainfall in two days result in flooding near the mouth and the death of a child
1950 Oct. 31 Nov. 01	· over 77 mm rainfall in two days led to general overbank flooding, particularly in Bowring Park

1951 Apr. 10-13	· 177 mm rainfall over three days caused high flows on Waterford River and flooding at Mundy Pond
1951 Nov. 30- Dec. 01	· 73 mm rainfall over two days caused flooding at St. John's Bridge and houses at Waterford Bridge
1953 Oct. 06-07	· 92.5 mm rainfall in one day filled Southside Hills' gullies, but apparently caused few problems
1953 Dec. 26-27	· 85.1 mm rainfall in one day caused destruction of Steady Waters Bridge, washouts at Donovans and near Mill Bridge
1955 Feb. 12-13	· 51.1 mm rainfall on wet soils led to flooding at mouth and at the Old Mill Bridge (Leslie Street)
1959 Nov. 11	· 83.6 mm in two days followed by 70.1 mm a week later resulted in local flooding
1962 Nov. 19-20	· 48 mm rainfall and 78.7 mm snowfall caused flooding along Mundy Pond outlet stream
1962 uncertain	· flooding in Donovans Industrial Park, Mount Pearl and Bremigans Pond area
1963 Jan. 01-03	· melting snow and ice jams flooded areas at several bridges (Dunn's, Steady Waters, Commonwealth, Donovans)
1964 Aug. 04-05	· 76 mm over two days caused flooding along Waterford River banks and in Bowring Park
1966 Dec. 20-22	· almost 103 mm in two days reported to have resulted in considerable local flooding
1968 uncertain (Jan.?)	· houses, roads, rail line reported flooded, as well as a washout of Steady Waters foot bridge
1970 Feb. 27- Mar. 02	· approx. 114 mm rainfall in four day period reported to have caused flooding around St. Bride's College
1971 Jan. 31- Feb. 01	· about 49 mm rainfall with snowmelt cause Waterford flooding at several sites (e.g., St. Bride's College)

1971 Nov. 14-15	·	local flooding reported in St. John's and within Bowring Park
1972 Nov. 10-12	·	about 116 mm rainfall resulted in local flooding in St. John's and road washouts (near Waterford River mouth)
1973 Feb. 22-23	·	68.3 mm in one day effected flooding near the river mouth (Southside Road, Water Street west)
1973 Jun. 17	·	50 mm rainfall caused isolated flooding and land-slides down Southside Hills at Kilbride
1974 Aug. 31	·	stormwater runoff led to basement flooding in Mount Pearl
1975 Aug. 23-24	·	approximately 75 mm rainfall in one day caused localized flooding and debris slides down South-side Hills
1976 Jan. 08-10	·	55 mm rainfall in two days caused flooding problems in St. John's (e.g., Mundy Pond, Victoria Park)
1976 Jan. 25-26	·	snow followed by snowmelt and rain brought river to bankfull stage, but with little flooding
1976 Dec. 24-26	·	31 mm rainfall with snowmelt caused Mundy Pond to overflow and slides down Southside Hills
1977 Dec. 27-29	·	45.7 mm rainfall with snowmelt flooded parts of Bowring Park, Kinsman Park and Squires Avenue
1978 Jan. 27	·	rainfall and snowmelt combined to flood many streets, but not significantly along the river
1979 Jan. 28-30	·	91 mm rainfall over 6 days caused local flooding and high water levels on the river
1980 November	·	unconfirmed report of flooding in Bowring Park, possibly resulting from melt of record snowfalls
1981 Oct. 10-11	·	over 121 mm rainfall in 2.5 days resulted in serious flooding from Southside Hills and flooding near Mundy Pond
1981 Nov. 26	·	76 mm rainfall in one day caused overbank flooding in a number of areas (e.g., Kilbridge, Southside Roads)

1982 Oct. 03-05	·	over 100 mm rainfall reported on one day during severe storm and high winds, but little or no flooding
1985 May 24-25	·	85 mm rainfall in 33 hours caused flooding from Donovans to Kilbride and along a South Brook Tributary
1986 Apr. 11	·	70 mm rainfall in 22 hours on frozen soil led to flooding at Dunn's and Waterford Road Bridges and other areas
1987 April	·	melt of major snowfall with rainfall resulted in no reports of flooding

It is possible to draw several preliminary conclusions from review of Table 3.1 and the above-listed reports. Generally, flooding problems result from synoptic rainfall events with durations of about two days. A rainfall total of about 70 mm appears to be the threshold value which will initiate flooding, although lesser amounts following earlier wet periods have also led to flooding. Overall, it is appropriate that the Technical Committee suggested that a precipitation runoff model which accounts for antecedent rainfall and soil moisture be used for this study.

It is also noteworthy that frequent, local flooding problems are reported (e.g. full gullies, runoff from the Southside Hills). It is often difficult to distinguish between flooding from the river and local drainage difficulties. However, it is clear that the most frequent reports of flooding have been from the Bowring Park area, through Kilbride to the river mouth.

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

This study draws on some of the results of early work which are summarized in the following section.

3.3 Urban Hydrology Studies

In 1978, Environment Canada and the Province of Newfoundland initiated the "Waterford River Basin Urban Hydrology Study". Its general objective was to assess the effect of urbanization on the water resources of the basin and evaluate solutions to various water management problems.

The main study was subdivided into a number of smaller, connected packages including evaluations of land use, flooding, storm drainage, and groundwater. Of particular value to this hydrotechnical study are the "Land Use Report" (1986), the two volume "Flood Study" (1986), the two volume "Data Summary Report" (draft 1987), and the draft of the "Watershed Modelling - HYMO" report prepared in 1985.

These four reports contain many similar elements to those prepared for this study. The watershed modelling study, for example, employs a hydrologic model (HYMO) from which several concepts were taken for use in the model selected for this study (QUALHYMO). The modelling report also details the development of hydrometeorological data and simulations of flood flows for the basin above Kilbride (for storms in 1981 and 1974, and for design precipitation storms). Watershed subcatchment delineations and hydrologic soils classifications were retained from that work for this study.

The data summary reports provide a wealth of information on rainfall, snow depth, and streamflow measurements taken in the early 1980's. It is clear that considerable effort was placed on the development of sound, accurate data and we have made good use of the reports as reference documents. Certain precipitation events and streamflow stage-discharge data were taken directly for use in this study.

The land use report provides particularly interesting data outlining the progression of urbanization throughout the western portion of the basin since the early 1970's. Extremely detailed mapping of every land use accompanies that report and the most recent (1984) mapping was adopted for our study.

The two volume flood study summarizes the development of a backwater model (HEC-2) for three reaches of the river; and the approach, measurements and data used to calibrate a number of model parameters. Our hydrotechnical study (Chapters 6.0 and 7.0) employs much of this work unchanged in our evaluation of backwater conditions. The absence of a significant flood peak during 1987 (January to October monitoring period) makes the monitoring information from the flood study of particular value to this study, which also uses the HEC-2 model.

The value of the Urban Hydrology Study goes well beyond just the data which can be extracted from various reports. Its true value lies in discussions, conclusions and findings of the works which identify characteristics of the basin and problems and solutions which have benefitted the enclosed study - and will benefit other, future studies. In many ways the Urban Hydrology Study provides the stepping stones to the successful completion of this Hydrotechnical Study.

4.0 HYDROLOGIC STUDIES

4.1 Introduction

The Terms of Reference for this hydrotechnical study of the Waterford River area note that the basic purpose of the study is to provide reliable estimates of open water profiles for 1:20 year and 1:100 year return period floods. The following sections discuss the derivation of these flood flows, which are later used to determine water level profiles (Chapter 7.0).

Flood flow estimates can be derived by various techniques including:

- single station frequency analysis of flow records at the site of interest (or from a nearby site on the same stream by transposition)
- regional frequency analysis of flow records within the general area of interest
- computer simulation using a mathematical watershed model and long-term weather records.

The choice of method in a particular situation is governed by the availability and length of streamflow record at or near the point of interest, as well as by the "stationarity" of the watershed (i.e. the nature of changes in land use, development, the channel).

The Terms of Reference make note of the ongoing, urban development in the watershed which is shown to have been most significant in the Donovans and New Town area of Mt. Pearl since 1973 (Land Use Report). Such changes in the surface characteristics of a watershed usually result in changes in surface runoff, and thus, single station frequency analysis using historical records or regional frequency analysis may not provide realistic estimates of flood flows. Hence, a computer simulation using mathematical models of the hydrologic system was selected as the principal method for determining 1:20 year and 1:100 year flood flows.

4.2 Data Base

Hydrologic models which simulate streamflow runoff require input data describing:

- physical watershed characteristics
- land use and surface soils
- precipitation and other climate data (e.g. air temperature)

Monitored steamflow response to those inputs is also required to calibrate various parameters in these models and to verify that the simulations are correct.

The following sub-sections briefly describe the data used for modelling and some of the characteristics of that data and the watershed itself.

4.2.1 Physical Characteristics

Available topographic mapping was employed as the principal input defining watershed slopes, areas of tributaries and ponds, channel slopes and channel contours. Principal use was made of 1:2,500 scale and 1:12,500 scale mapping developed by the Provincial Department of Forestry and Agriculture, and 1:2,500 scale mapping of the river channel prepared by McElhanney Mapping Services in 1985 for the Canada-Newfoundland Flood Damage Reduction Program.

Figure 4-1 delineates the entire Waterford Basin and the boundaries of several of the principal subcatchments. Catchment 301, which drains the western portion of the watershed (including Donovans Industrial Park), slopes from headwater elevations of about 200 m to a channel elevation of about 133 m in approximately five kilometres.

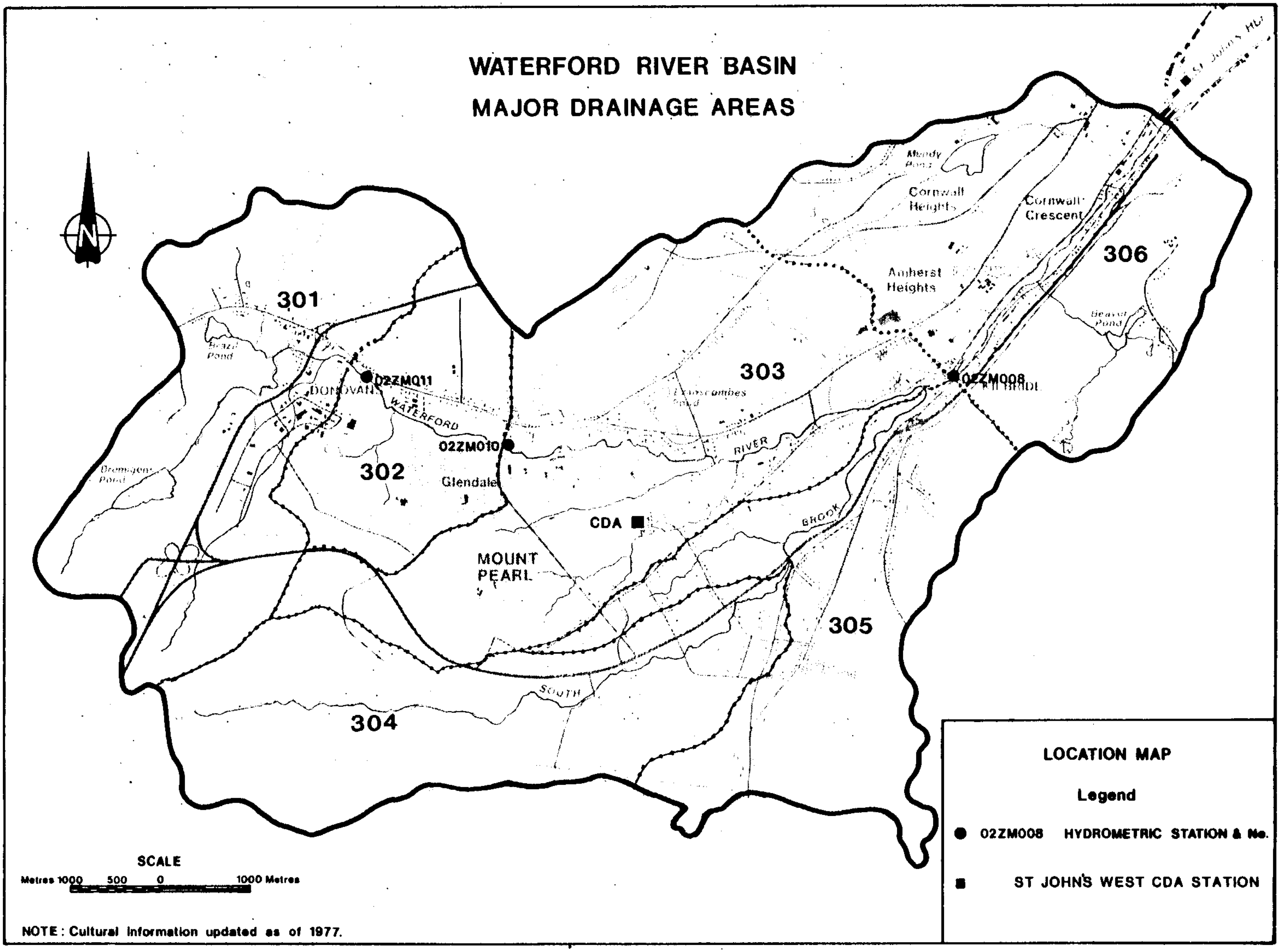

WATERFORD RIVER BASIN
MAJOR DRAINAGE AREAS
N
301
302
303
304
305
306
02ZM011
02ZM010
02ZM008
DONOVANS
WATERFORD
RIVER
SOUTH
BROOK
Glendale
MOUNT PEARL
CDA
Amherst Heights
Cornwall Heights
Cornwall Crescent
LOCATION MAP
Legend
02ZM008 HYDROMETRIC STATION & No.
ST JOHN'S WEST CDA STATION
SCALE
Metres 1000 500 0 1000 Metres
NOTE: Cultural information updated as of 1977.

FIGURE 4-1

Catchment 302 drains portions of the strip development along Topsail Road north of the river, and the western part of Mount Pearl to Commonwealth Avenue. Catchment 303 includes the majority of Mt Pearl, New Town developments south of Mt. Pearl, and the western section of St. John's. Although extensively urbanized, the northwestern segment remains undeveloped.

Catchments 304 and 305 mark the drainage area of South Brook which joins the main branch at Kilbride. This tributary is somewhat elongated in shape and bounded on the east by a range of steep hills lying between Bay Bulls Road and Petty Harbour Long Pond.

The Southside Hills extend from Kilbride into Catchment 306, with the river marking the distinct topographic break between steep, hillside slopes on the east and urban St. John's on the west. Drainage from the Southside Hills is generally through short, steep channels (some channelized) or overland. Runoff from the developed area of St. John's is carried to the river via sewers and the road network, as well as overland and through several channels.

The sewer network drawings for St. John's, Mt. Pearl and New Town were examined in considerable detail to characterize drainage areas in Catchments 303 and 306, and the eastern portion of Catchment 302. In addition, aerial photography from 1966, 1977, 1984 and 1985 were employed to determine drainage details not shown on the topographic mapping. Extensive use was also made of the photographic records from earlier Environment Canada and Newfoundland Department of Environment studies.

Last, a profile of the channel was prepared from the upper catchment to the river mouth (Figure 4-2) to provide information relating to channel storage and routing of flood flows. The figure shows a relatively steep channel (average slope 0.012 m/m) having a terraced form. Comparatively gently sloping reaches averaging about 0.0065 m/m (for example, from Commonwealth

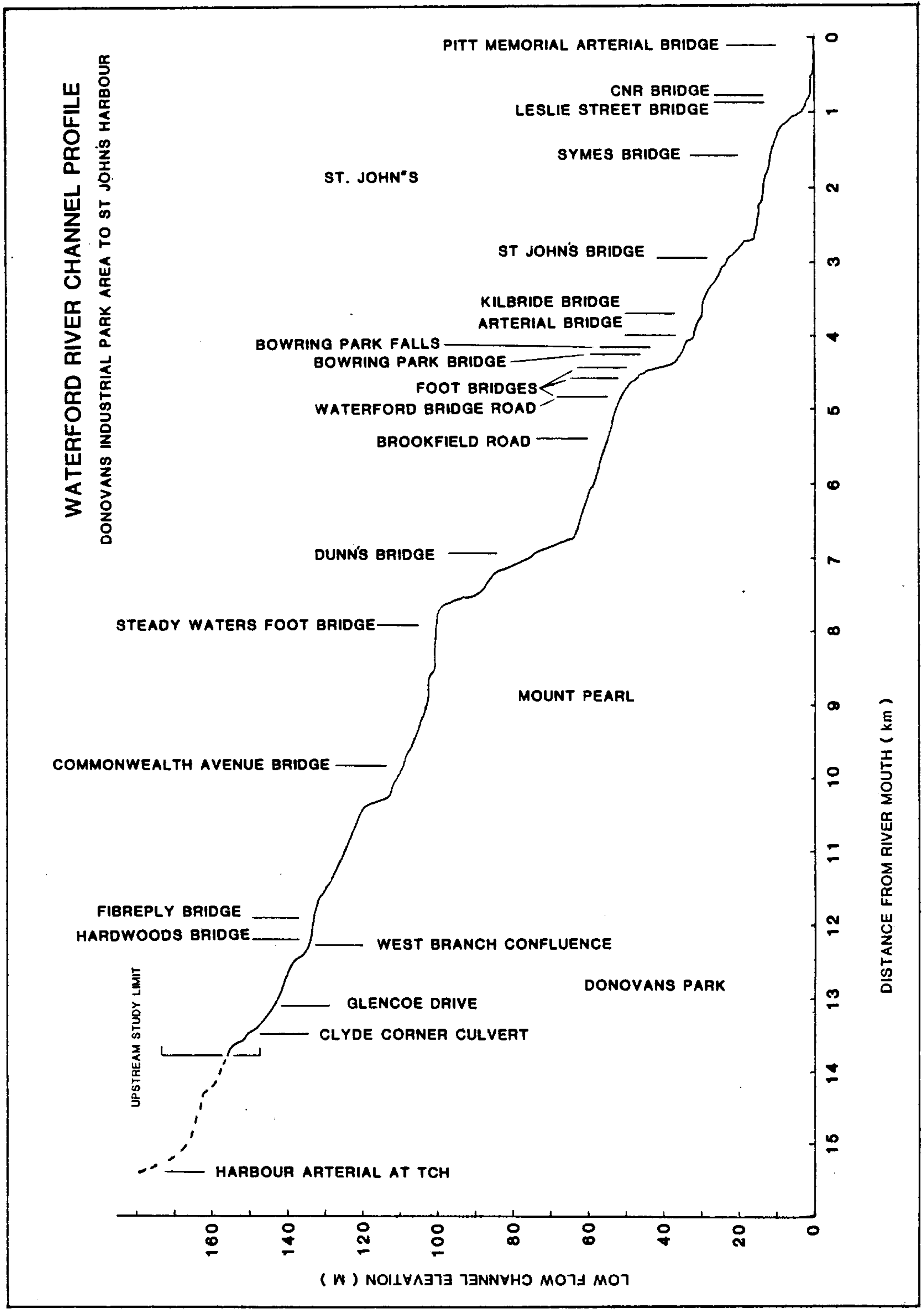

WATERFORD RIVER CHANNEL PROFILE
DONOVANS INDUSTRIAL PARK AREA TO ST JOHN'S HARBOUR
PITT MEMORIAL ARTERIAL BRIDGE
CNR BRIDGE
LESLIE STREET BRIDGE
SYMES BRIDGE
ST. JOHN'S
ST JOHN'S BRIDGE
KILBRIDE BRIDGE
ARTERIAL BRIDGE
BOWRING PARK FALLS
BOWRING PARK BRIDGE
FOOT BRIDGES
WATERFORD BRIDGE ROAD
BROOKFIELD ROAD
DUNN'S BRIDGE
STEADY WATERS FOOT BRIDGE
MOUNT PEARL
COMMONWEALTH AVENUE BRIDGE
FIBREPLY BRIDGE
HARDWOODS BRIDGE
WEST BRANCH CONFLUENCE
DONOVANS PARK
GLENCOE DRIVE
CLYDE CORNER CULVERT
UPSTREAM STUDY LIMIT
HARBOUR ARTERIAL AT TCH
DISTANCE FROM RIVER MOUTH (km)
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
LOW FLOW CHANNEL ELEVATION (M)
160
140
120
100
80
60
40
20
0

FIGURE 4-2

Bridge to Steady Water Footbridge in Mt. Pearl), are separated by very steep reaches. These sleep reaches have an average slope of 0.043 and include several rapids and waterfalls.

Overall, the watershed channel is steep, as are the basin slopes - particularly along the eastern watershed boundary from St. John's Harbour to Kilbride and to Rocky Pond. As a result, it would be expected that flows concentrate quickly in the watershed following rainfall events and that flood hydrographs rise quickly to their peaks.

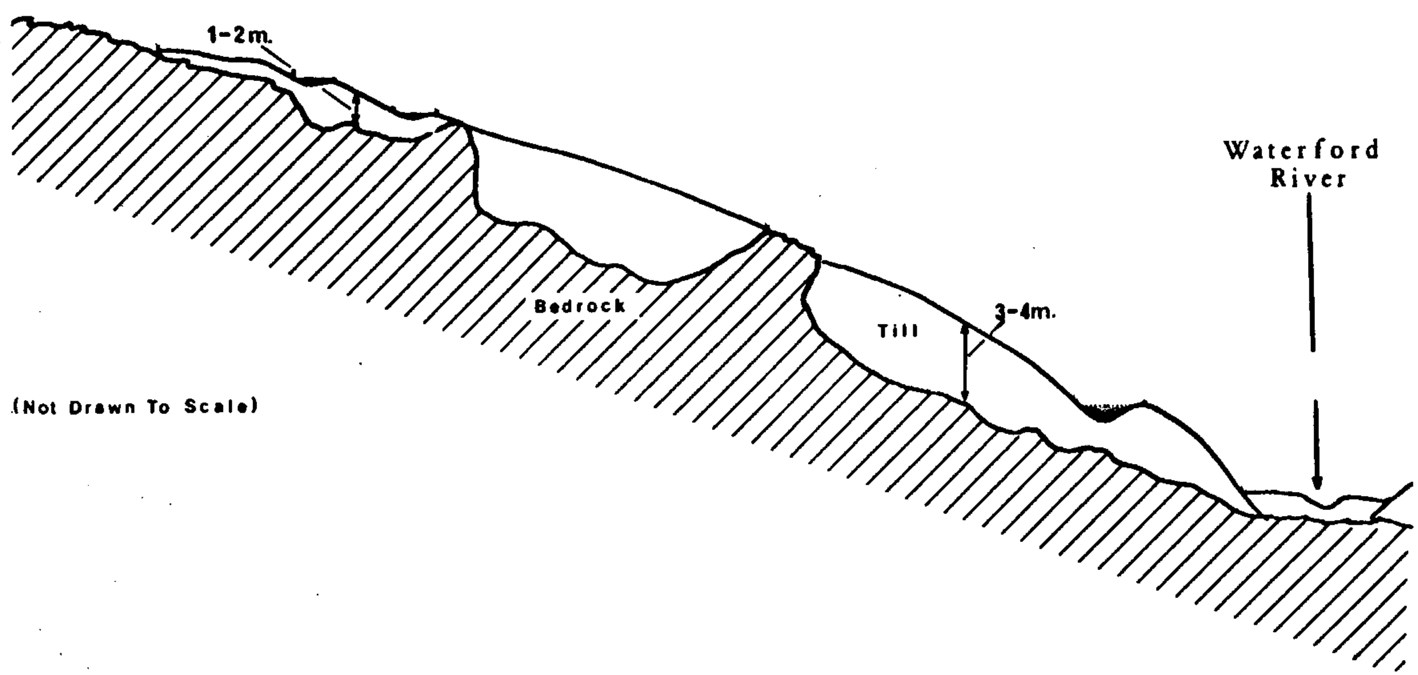
4.2.2 Surficial Geology

The surface soils and exposed bedrock in the basin reflect the results of glaciation. Soils are thin over the slopes around most of the basin perimeter (particularly between St. John's and Kilbride on the crown of the Southside Hills, on Kenmount Hill, and on the hill tops south of Donovans Industrial Park). Further down the slopes into the heart of the basin, soil depth increases but bedrock is generally never far from the surface and outcropping is not unusual. Figure 4-3 shows the soil bedrock relationship considered to exist in the basin (Batterson, 1984).

The soils are derived from underlying parent materials and are largely coarse to moderately-coarse in texture. Although the main stream is generally steep, gravels are observed to have deposited in bars in several locations (e.g. Symes Bridge, Bowring Park Pond). These bars and certain banks are reported to erode during periods of high flow (Baker, p.c. 1987).

Soils in the basin are frequently mixtures or "complexes" of the six principal soil types in the basin; Bauline, Cochrane, Organic, Pouch Cove, Red Cove and Torbay. The distribution of these soils and complexes have been mapped in detail in the Newfoundland Soil Survey Report (Heringa, 1981; Environment Canada, 1985) for the basin west of Kilbride. Their distribution is not well known in the urban portion of St. John's (drainage area 306

Effect of glacial deposition of topography.



Sketch illustrating the variability of the overburden thickness.

FIGURE 4-3

in Figure 4-1) but can be extrapolated into the city following land forms and distributions at the city boundary. Uncertainties with this approach are alleviated by the similarity in soil types in the area and the disruption and mixing of soils associated with urbanization.

Hydrologic soil group classifications are presented in Figure 4-4 (after Environment Canada, 1985). These classification (e.g. A, B, C, D) or combinations of classes (e.g. BC) describe the runoff potential of the soil, ranging from low (Class A) to high (classes C and D). Noteworthy in Figure 4-4 is the predominance of soils having moderate to high runoff potential.

There are two conclusions of interest regarding surficial geology. The presence of bedrock near the soil surface at higher elevations, and the predominance of soils with high runoff potential in the valley are expected to contribute to a relatively high rainfall-to-runoff ratio. Further, it is possible that runoff may begin from some of the areas of shallowest soil depths before the theoretical (infinite soil depth) water holding capacity of the soil is reached.

4.2.3 Land Use

The watershed modelling investigations of the Urban Hydrology Study employed 1981 land use conditions in the simulation of rainfall runoff. At the time of that study, 1981 information represented the most up-to-date status of watershed development which could be incorporated with other watershed data such as precipitation and streamflow.

This present study also uses the most reliable land use data, which is that of 1984. In the area west of Kilbride, this information was prepared by Environment Canada, Inland Waters Directorate (Atlantic) as a component of their recent land use report (1986). The area downstream of Kilbride in St. John's has been mapped by the City on 1:2,500 scale topographic drawings, which plot location and type of land use. These were assembled by our study

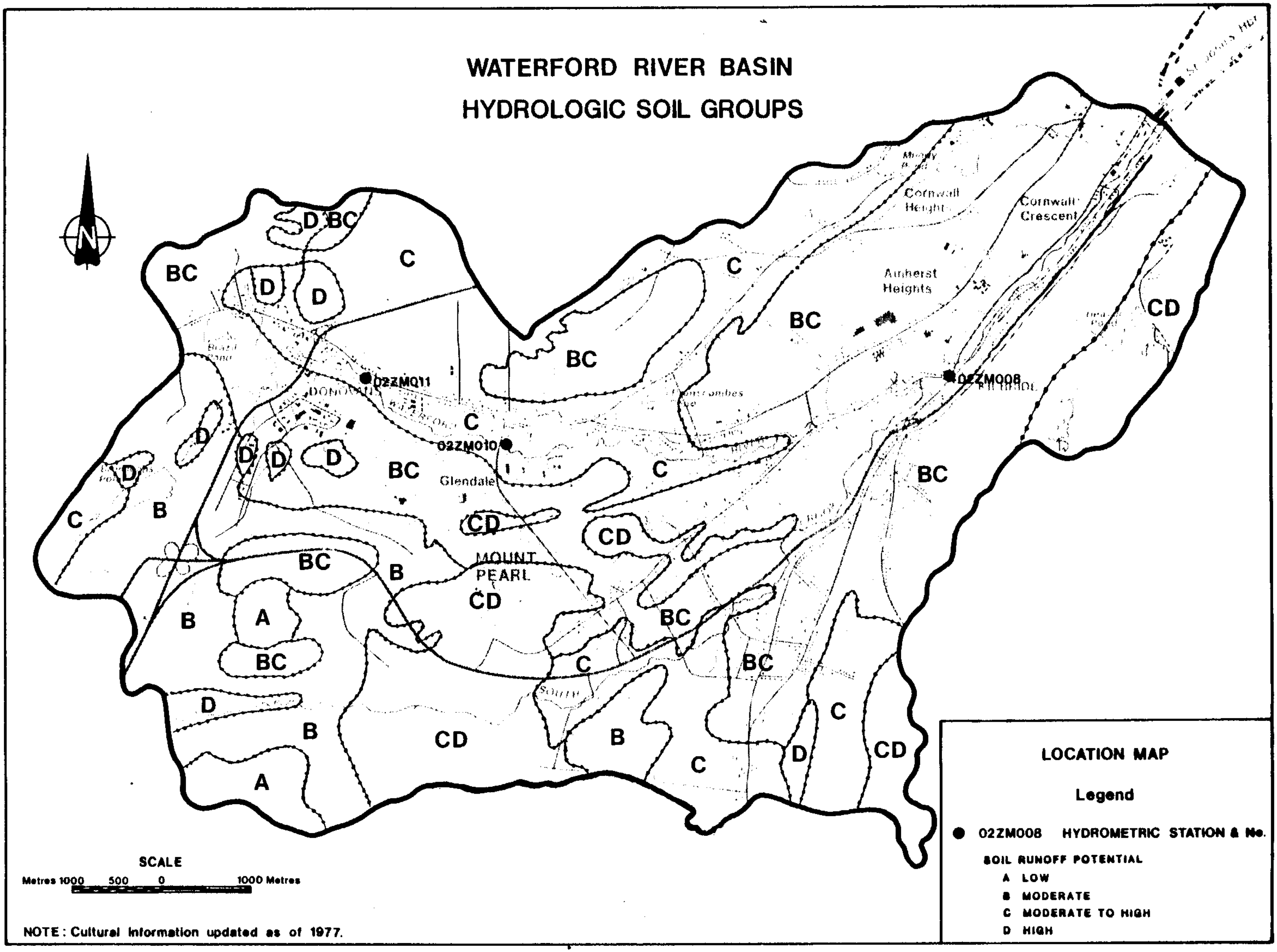

WATERFORD RIVER BASIN
HYDROLOGIC SOIL GROUPS
N
Cornwall Heights
Cornwall Crescent
Amherst Heights
02ZM008
02ZM011
02ZM010
Glendale
MOUNT PEARL
BC
C
CD
D
B
A
LOCATION MAP
Legend
02ZM008 HYDROMETRIC STATION & No.
SOIL RUNOFF POTENTIAL
A LOW
B MODERATE
C MODERATE TO HIGH
D HIGH
SCALE
Metres 1000 500 0 1000 Metres
NOTE: Cultural information updated as of 1977.

FIGURE 4-4

team for the western area of the City within the Waterford Basin. Aerial photography was also employed by our people to complete this inventory for the area containing the Southside Hills and southeastern segments of the watershed.

It is noteworthy that recent development in the watershed has largely focussed in the Donovans Industrial Park and in residential development of the Newtown area south of Mount Pearl. Both of these development areas are within regions overlain by soils having naturally high runoff contributions to the river. As a result, the increase in surface imperviousness accompanying development may not be reflected in substantial increases in runoff from these developing areas.

4.2.4 Hydrometeorology

Precipitation data is available in the Waterford area from the Atmospheric Environment Service, Environment Canada site at the St. John's Airport, and from the St. John's West CDA site located within the study area. The airport records are from a continuously recording, tipping bucket gauge which was first employed in 1959. Its data is reasonably unbroken since late 1963. Supplementing this is an excellent record of 24-hour rainfall since 1942 and 6-hour totals since 1957.

Precipitation data is available from 1954 at St. John's West CDA, but its distribution through time was not measured by a continuous recording gauge until 1981. At present, the charts from this gauge are available from Newfoundland Environment but are not digitized for computer access from Environment Canada. The location of the St. John's West CDA site is shown on Figure 4-1.

Similar, undigitized data is available from a gauge in Newtown (14 Dalhousie Crescent) and a recently established site (circa 1987) just west of Donovans Park. There are also some recording rainfall gauge record taken by the Geography Department at Memorial University since 1978.

The data base selected for modelling purposes is a blend of information from most of the above-mentioned sites. The years from 1959 to mid-1980 (22 years) are covered by data from St. John's Airport, and those from 1981-1986 (6 years) are taken from the record of St. John's West CDA. In instances where the CDA gauge was being tested or otherwise failed to provide a rainfall record, the sequence was replaced with data from the Airport or Dalhousie Crescent. This approach is justified because of the similarity between rainfall records at these sites (Environment Canada, 1985), their close proximity, and the synoptic nature of most rainfall events in the area.

No attempt was made to distribute rainfall in time or space over the basin in any manner different than that monitored at the continuous recording gauges. There was not sufficient data to attempt this in any but a very few storms, and no general conclusion could be drawn from these for use in the vast majority of storms.

It must be noted, however, that the rainfall gauges correctly describe rainfall at only one location - the gauge site. It is certain that differences in rainfall amounts and timing will have occurred at other locations in the basin and, consequently, that precipitation runoff cannot be perfectly simulated.

Snowfall data and temperature data for verifying snowfall and rainfall events and for assessing snowmelt were obtained on magnetic tape from St. John's Airport. Depth of snow on the ground was obtained from St. John's West CDA from 1964 to date, to validate snowfall data derived from airport data.

4.2.5 Streamflow Data

Streamflow has been monitored in the watershed at the following principal locations (shown in Figure 4-1);

- Kilbride - station 02ZM008 - late 1973 to present

- Mt Pearl - station 02ZM008 - March 1981 to present

- Donovans - station 02ZM011 - May 1981 to end 1984

Data from these locations was available to the study on magnetic tape for the period of record until June 1986. More recent data was obtained in paper copy. Stage-discharge curves, used to transform water level reading into streamflow were also provided (Figure 4-5). In 1986, a gauge was established on South Brook at Pearl Town Road (02ZM021), but no data from this site was available to the project. There is also runoff information from some storms at a storm sewer outfall at Mount Pearl (02ZM012). Because data from this latter site is also unavailable through the Water Survey of Canada, relates only to the storm sewer discharge from a small rapidly developed drainage area (0.4 km^2), and is regulated in passage through an outlet structure, it was not considered appropriate for general use at this watershed-scale of study (76.7 km^2).

Streamflow data is the only information which can be used to verify hydrologic simulations. Consequently, several discussions were held with Water Survey of Canada personnel in the St. John's and Dartmouth offices regarding the quality of data obtained from measurements at the three principal sites listed above. This was warranted in view of several revisions made by the WSC to their flow estimates (e.g. 1980) and the data given in Figure 4-5. The figures show that various measurements of stage and flow have lead to a range of flow for a given, measured level. The range is about ±15% from the average during high flow, high water periods.

Concern was expressed by the WSC about the reliability of data from the Donovans site (02ZM011), and it was noted that there were an unusual number of missing data for the station due to weed growth, backwater and estimated periods. The quality of data was described only as fair to good and it is

RATING CURVE FOR WATERFORD RIVER AT
KILBRIDE-HYDROMETRIC STATION 02ZM008

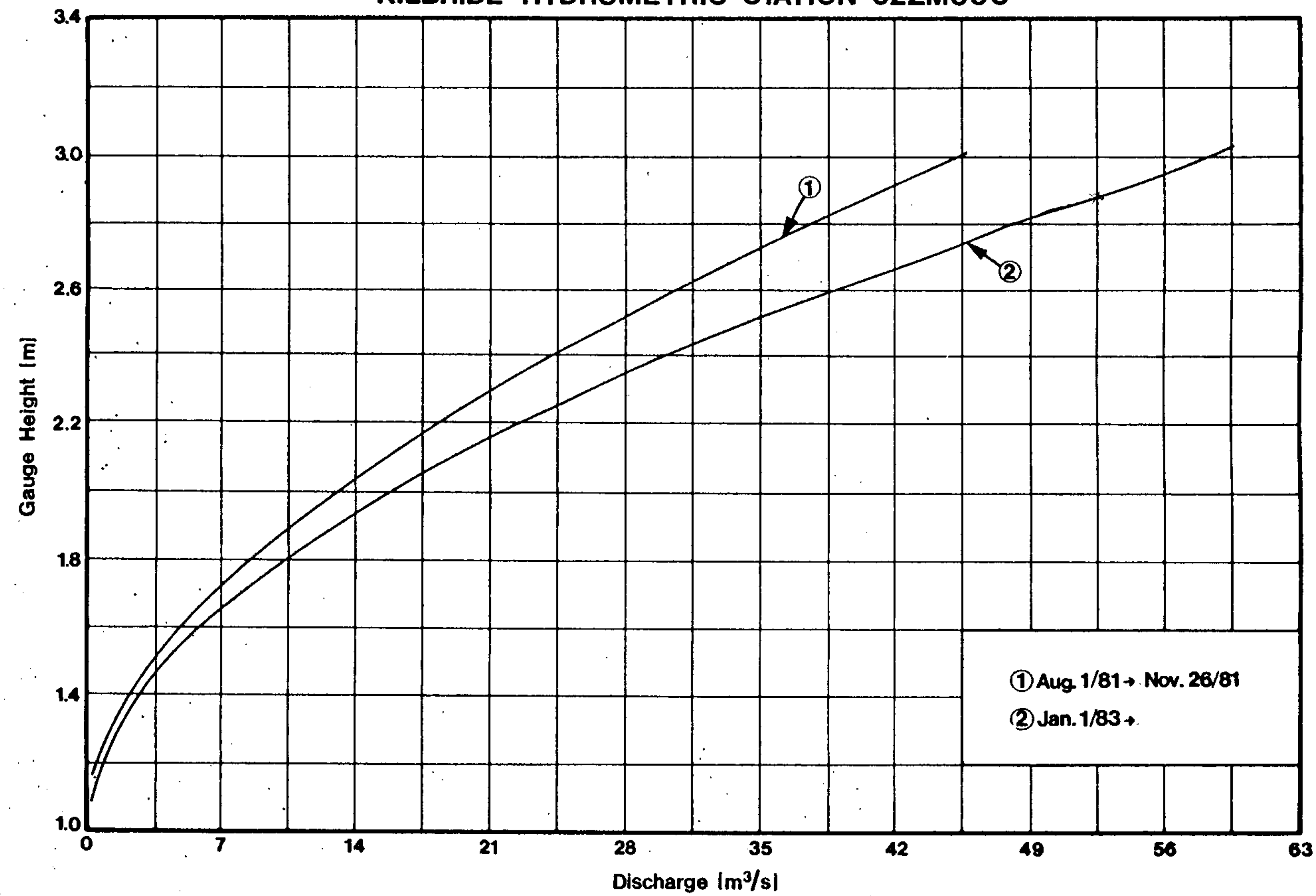
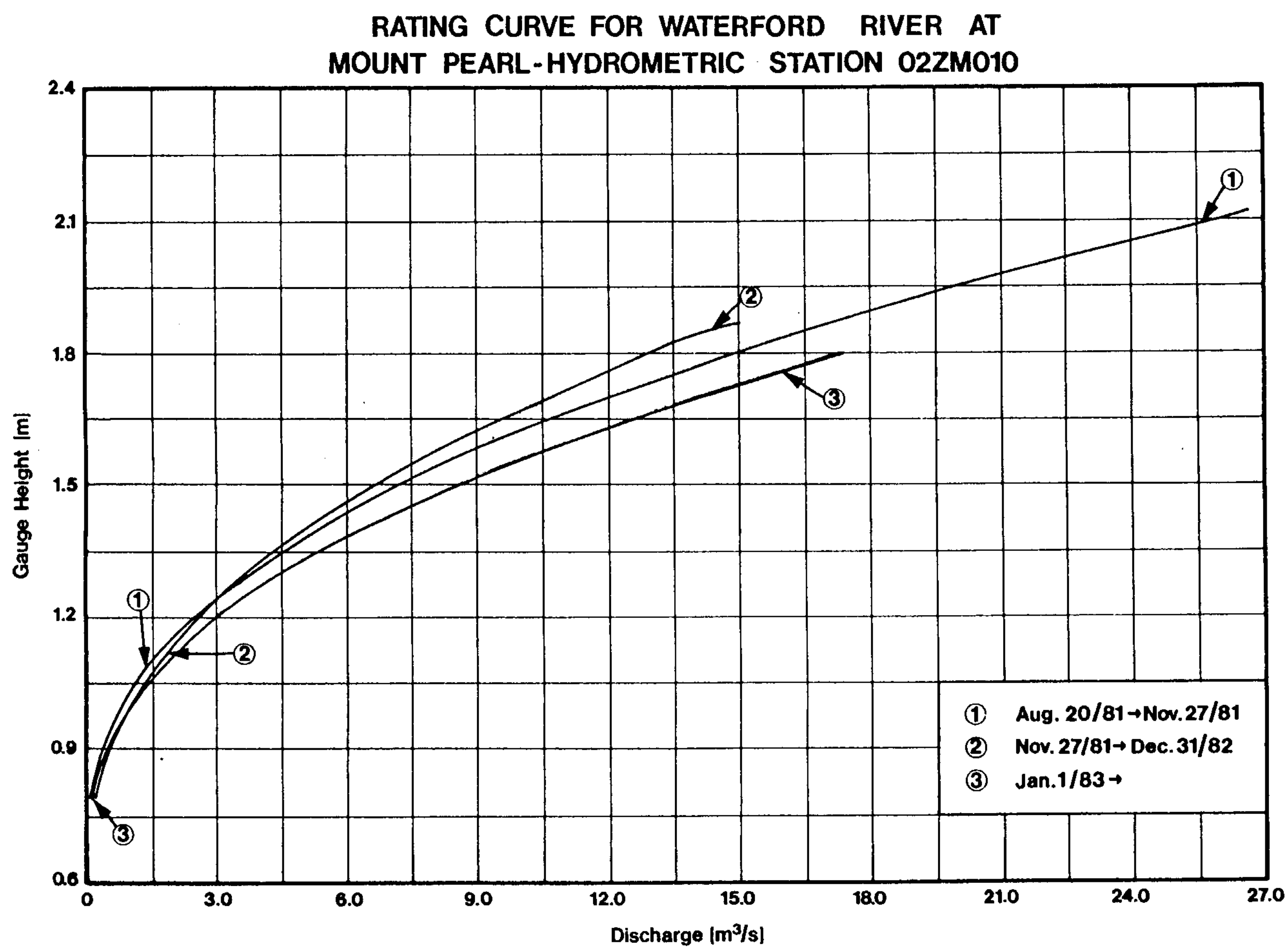


FIGURE 4-5a



RATING CURVE FOR WATERFORD RIVER AT
MOUNT PEARL-HYDROMETRIC STATION 02ZM010
Gauge Height (m)
2.4
2.1
1.8
1.5
1.2
0.9
0.6
0
3.0
6.0
9.0
12.0
15.0
18.0
21.0
24.0
27.0
Discharge (m³/s)
① Aug. 20/81→Nov. 27/81
② Nov. 27/81→ Dec. 31/82
③ Jan. 1/83→

FIGURE 4–5b

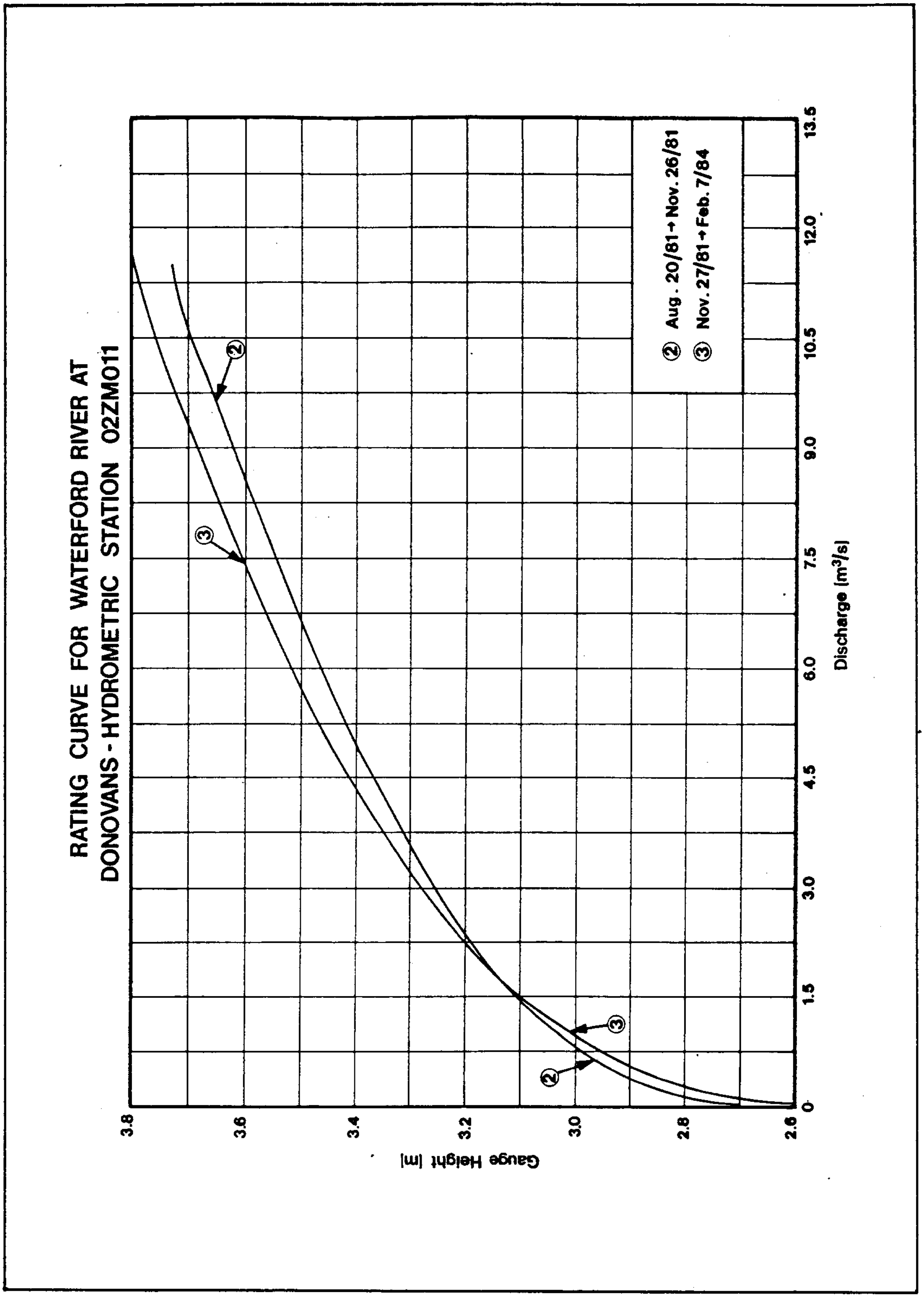

RATING CURVE FOR WATERFORD RIVER AT
DONOVANS - HYDROMETRIC STATION 02ZM011
② Aug. 20/81 → Nov. 26/81
③ Nov. 27/81 → Feb. 7/84
Gauge Height [m]
Discharge [m³/s]
3.8
3.6
3.4
3.2
3.0
2.8
2.6
0
1.5
3.0
4.5
6.0
7.5
9.0
10.5
12.0
13.5

FIGURE 4-5c

expected that reported flows may only be accurate to perhaps 15% of true values. Records from Mount Pearl and Kilbride were considered much more accurate and within about 10% of true discharge values, principally because the sites are good and flows are frequently measured.

In view of the above, Water Survey personnel advised use of the Kilbride and Mt. Pearl locations for model calibration and verification. The Donovan's data was considered unreliable by comparison.

5.0 HYDROLOGIC MODELLING

5.1 The QUALHYMO Model

After careful consideration of the requirement for hydrologic information concerning magnitude and frequency of flood discharges, the model, QUALHYMO (Wisner and Rowney, 1985), was chosen to estimate peak flows throughout the watershed under existing (1984) land use conditions. QUALHYMO is a simple continuous water quantity (and quality) simulation model which was developed in 1983 at the University of Ottawa for the analysis of stormwater detention ponds. The model can be used as a general tool for simulating rainfall runoff but is most suited to planning-level analysis of river basins where the land surface is developing from a rural or undeveloped state to an urban land use.

Several concepts in the QUALHYMO model have been retained from two earlier single event hydrologic models: HYMO (Williams and Haan, 1973) a runoff model which has been tested and extensively used for the hydrologic component of flood plain mapping studies; and OTTHYMO (Wisner et al, 1983), which is gaining widespread acceptance in planning-level studies of storm water management within urban areas. Unlike these single event models which require the modeller to approximate watershed conditions before each storm, the continuous simulation capability of QUALHYMO enables hour-by-hour updating of soil moisture, snowmelt and baseflow. As discussed below, QUALHYMO also improves upon the way that runoff is generated from impervious and pervious land use segments - important for accurate modelling of urbanized watersheds.

Meteorologic input to the QUALHYMO model consists of hourly precipitation and temperature records. During the winter period, precipitation is categorized as liquid or snow depending on air temperature relative to a specified threshold value at or near to 0° Centigrade. Snowpack accumulation and ablation is estimated by a temperature index equation.

Further technical discussion about the model and model enhancements is provided for the interested reader in Appendix 1.0.

5.2 Basin Discretization

Several decisions were made following the review of the watershed physical characteristics, surficial geology, land use, the previous watershed modelling study and the needs of present and future studies in the basin. It was determined that:

- the same discretization of subcatchments within the Waterford Basin as was used in the Urban Hydrology watershed modelling report was appropriate for the study area west of Kilbride (Figure 5-1). This selection is particularly appropriate to avoid confusion among reviewers of both studies. The following exceptions were made:

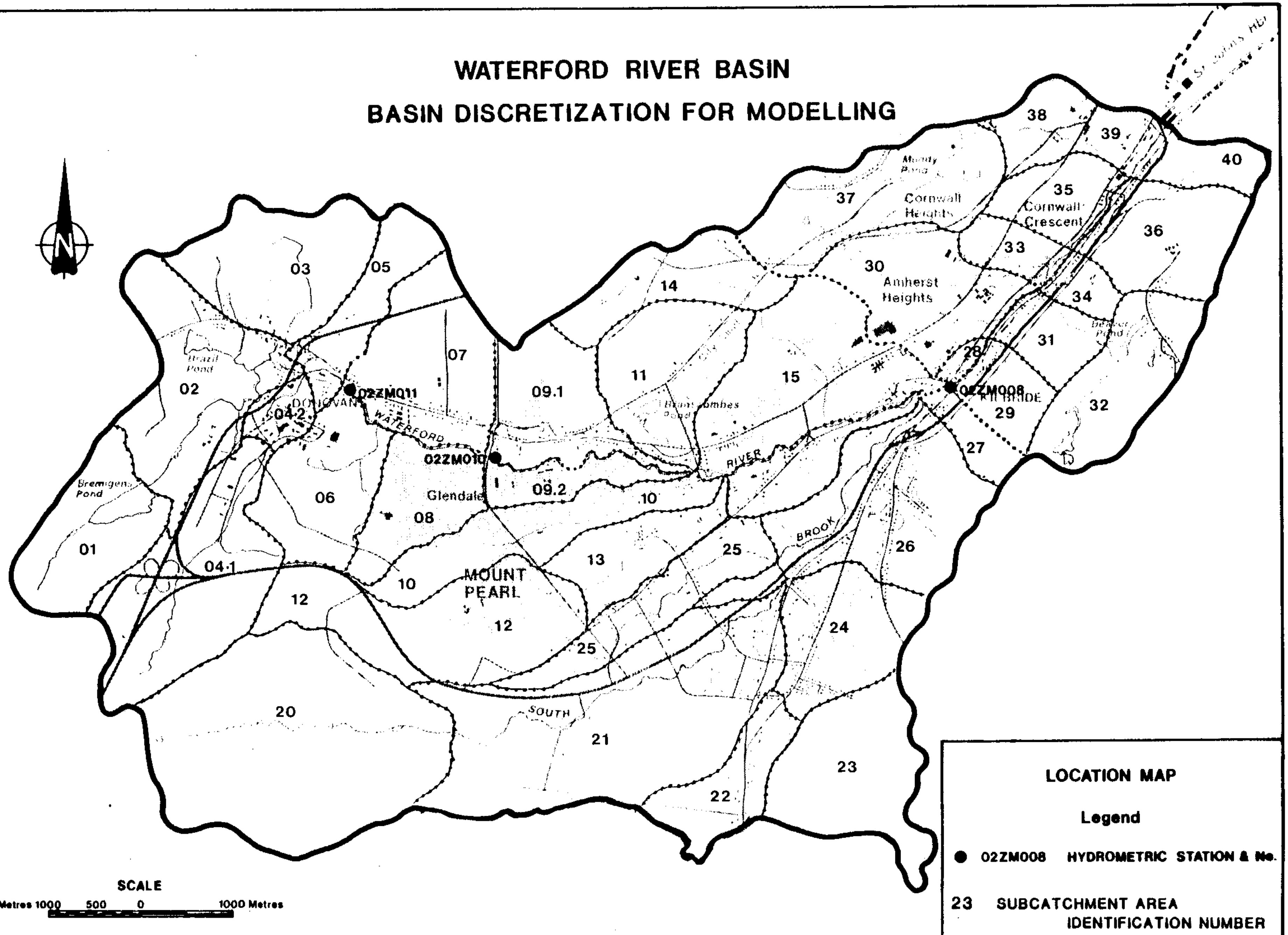
    - subcatchment #4 was divided into two (2) subcatchments to obtain an intermediate flow point near the upstream end of the study area at Donovans Industrial Park (subcatchments 4.1 and 4.2).

    - subcatchment #9 at Mount Pearl was broken into two (2) subcatchments to capture the predominantly rural character of this area on the north side of the river (9.1) and the urban character on the south side of the river (9.2).

- The area of the basin east of Kilbride, which was not modelled previously, was subdivided into 12 subcatchment areas to provide detailed flow points for input to the backwater modelling (Section 7.0). Particular use was made of sewer and stormwater layout drawings to assist in discretizing the subcatchments.

- elevation area-discharge relationships for Mundy and Beaver Pond outlets were derived from drawings, maps and surveys by the study team. A modification was made to storage elevation data developed in previous

WATERFORD RIVER BASIN
BASIN DISCRETIZATION FOR MODELLING



NOTE: Cultural information updated as of 1977.

LOCATION MAP

Legend

● 02ZM008 HYDROMETRIC STATION & No.

23 SUBCATCHMENT AREA IDENTIFICATION NUMBER

FIGURE 5-1

studies for Bremigans Pond to more accurately represent the stage-storage-discharge relationship.

Figure 5-1 shows the geographical distribution of the discretized subcatchments.

5.3 Imperviousness and Hydrologic Soil Complexes

This current study diverges from the previous Urban Hydrology Study in that pervious and impervious areas are determined separately for each subcatchment area.

In the previous watershed modelling study (Environment Canada, 1985), land use was combined with hydrologic soils classifications to obtain a weighted hydrologic soil cover complex number (i.e. CN number) for each subcatchment. The impervious components of each catchment were included in this CN number assignment according to established percent imperviousness categories, summarized in Table 5-1 (from Map K-2, Urban Hydrology Study, 1985 Draft report).

For this current study, pervious and impervious areas are treated separately, and were planimetered and tabulated independently (and only totalled for verification checks on total subcatchment areas computed earlier in the Urban Hydrology Study). Then, based on soil types, CN numbers were determined for each pervious area for subsequent transformation into water loss to the soil (a "soil loss parameter" discussed in detail in the next section).

The impervious areas were treated as follows:

- the surface area of various urban land uses (e.g. commercial, industrial, residential, etc.) were determined for each subcatchment

TABLE 5-1

LAND USE/IMPERMEABILITY CATEGORIES

Impermeability in the Following Categories Includes Roof Tops, Pavement, Hard Packed Ground, and Some Goods Stock-Piled Outside Which Allow Rapid Runoff of Surface Water.

AGRICULTURAL

A_1 Bog; used for grazing or improved for agricultural use.

A_2 Cropland, Close Grown, Improved Pasture and Forage Crops; includes lands used for associated farm buildings.

A_3 Cropland Grown in Rows.

A_4 Land Cleared for Agricultural Purposes; brush piles evident.

H Intensive Agricultural Activity; e.g. greenhouses

K Natural Grasslands; or unimproved pasture (where grazing may occur). Idle land which is not principally vegetated in shrubs or trees (bushes and trees may cover no more than 25%).

COMMERCIAL/INDUSTRIAL

C_1	Very Low Percentage of Impermeable Surfaces; Commercial/industrial property where very little of its surfaces can be interpreted as impermeable; e.g., initial stages of construction.	Impermeability: 0 - 15%
C_2	Small Percentage of Impermeable Surfaces; low building density.	Impermeability: 16 - 30%
C_3	Moderate Percentage of Impermeable Surfaces;	Impermeability: 31 - 45%
C_4	Moderately High Proportion of Impermeable Surfaces; Similar to C_2, but a greater percentage will be impervious.	Impermeability: 46 - 60%
C_5	High Percentage of Impermeable Surfaces; where close spacing of buildings and large areas of hard, impervious surfaces can be seen.	Impermeability: 61 - 75%
C_6	Very High Percentage of Impermeable Surfaces; where clearly most of the area is impervious.	Impermeability: 76 - 90%
C_7	Virtually Complete Impermeability; commercial property which, regardless of building size, has virtually all of its surface interpreted as impermeable.	Impermeability: 91% plus

INSTITUTIONAL

I_1 Institutional Where Less than 40% are Impermeable Surfaces; e.g. cemeteries schools, hospitals.

I_2 Institutional Where All Surfaces Account for Greater Than or Equal to 40% of the Area; e.g. some schools, hospitals community centres.

TABLE 5-1 (continued)

LAND USE/IMPERMEABILITY CATEGORIES

UNVEGETATED SURFACES

L_1 Unvegetated, Low Impermeability: where less than 40% is impervious. This may have resulted from removal of surface material clearing or areas to which fill has been brought in but not graded

L_2 Unvegetated, High Impermeability: where 40% or greater is impervious - surfaces are usually graded and land use activity tends to be in transition.

L_3 Rockland, Natural Bare Rock: may consist of minimal amount of vegetative cover.

Residential

R_1	Initial Stages of Construction	Impermeability: 0 - 15%
R_2	Low Density Housing: (in terms of space between houses) - this often includes rural dwellings.	Impermeability: 16 - 30%
R_3	Medium Density Housing:	Impermeability: 31 - 45%
R_4	Medium to High Density Housing: moderate area of impervious surfaces, either consolidated or separated. Most suburban housing falls here.	Impermeability: 46 - 60%
R_5	High Density Housing: close spacing of houses where much of the ground surface is impervious with either consolidated or separated portions of pavement.	Impermeability: 61 - 75%
R_6	Very High Density in Housing Group: where clearly most, if not all, the ground surface is impervious.	

Other Categories

X Highways: major transportation routes which are usually four lanes wide includes land on road right-of-way draining into highway runoff system.

E Excavation, Gravel Pits, Quarries: where there clearly appears to be significant removal of earth materials, e.g. Borrow pits, quarries, which contribute to depression water storage.

O Open Space, Parkland, Recreation Use: (grass covered) e.g. baseball, golf courses.

T Trees, Forest: Where crown closure is greater than 25% and trees are at least seven metres in height.

U_1 Unproductive Woodlands, Scrub: Wet Site; land with vegetative crown cover of less than 25% and shorter than seven metres in height. Stunted trees, bushes.

U_2 Unproductive Woodlands, Scrub: Messic to dry site; land with vegetative crown cover of less than 25%

M Marsh, Bogs: Low lying, level areas with characteristic vegetation appearing very low and evenly textures. Water may be visible.

Z Water Bodies: Ponds, river, lakes

- the percent impermeability for each of these urban uses was determined from Table 5-1. An average value for each range of percent impermeability was chosen to simplify matters (e.g. 68% impermeability was selected for a range given as 61% to 75%).

- the pervious component of each impervious area was then computed. Following the above example, where 68% of the surface was determined to be impervious, the remaining 32% was taken as a pervious surface.

- the pervious component (32%) was then weighted with the other pervious/rural area to obtain a weighted CN value for the pervious areas of each subcatchment.

- last, the total impervious area for each subcatchment was expressed as a percent of the total subcatchment drainage area.

Table 5-2 presents our initial estimates of weighted CN values and percent imperviousness computed for each of the 39 subcatchments.

5.4 Transformation of Hydrologic-Soil Cover Complexes (CN) for QUALHYMO Input

As discussed previously, the QUALHYMO model uses the U.S. Soil Conservation Service (SCS) procedure to determine runoff from pervious land segments. The technical procedure is outlined in detail in Appendix 1.0.

In summary, QUALHYMO employs an approach which continuously keeps track of soil moisture conditions for each hydrologic soil cover complex number (CN). Rather than using just the CN Number, the model transforms that number into a set of numbers which include a maximum, minimum and rate of change in soil moisture conditions (SMAX, SMIN and SK). The "S" values change each day depending on the antecedent precipitation.

TABLE 5-2

WATERFORD RIVER HYDROTECHNICAL STUDY

INITIAL ESTIMATES OF

SUBCATCHMENT AREAS, IMPERVIOUSNESS AND

COMPLEX NUMBER (CN) MODEL INPUT DATA

Sub-Basin No.	Area (ha)	Imp. Area (ha)	Imp. Area (%)	Perv. Area (ha)	Weighted CN (AMC II)
1	193.0	6.3	3.3	186.7	65
2	268.0	36.4	13.6	231.6	66
3	228.0	15.1	6.6	212.9	72
4.1	293.6	63.4	21.6	230.2	63
4.2	46.4	26.5	57.1	19.9	67
5	106.0	14.2	13.4	91.8	73
6	177.7	18.7	10.5	159.0	70
7	216.8	34.3	15.8	182.5	73
8	133.0	49.3	37.1	83.7	67
9.1	160.8	23.2	14.4	137.6	65
9.2	73.4	31.2	42.5	42.2	68
10	166.2	51.0	30.7	115.2	71
11	179.1	11.4	6.4	167.7	71
12	298.5	66.5	22.3	232.0	69
13	110.2	2.0	1.8	108.2	67
14	160.9	1.5	0.9	159.4	73
15	329.4	128.2	38.9	201.2	67
16*	84.2	21.0	25.0	63.2	66
20	547.3	0	0	547.3	67
21	515.3	42.4	8.2	472.9	74
22	109.0	2.0	1.8	107.0	79
23	221.8	0.8	0.7	221.0	75
24	175.3	30.0	17.1	145.3	72
25	135.5	2.7	6.5	38.6	72
26	306.1	46.3	15.1	259.8	70
27	41.3	2.7	6.5	38.6	69
28	17.0	6.5	38.0	10.5	67
29	71.0	0.85	1.2	70.2	67
30	19.5	63.3	32.0	131.7	67
31	79.0	3.6	4.5	75.4	68
32	191.0	2.3	1.2	189.7	78
33	56.0	20.2	36.0	35.8	67
34	37.0	3.0	8.0	34.0	68
35	84.0	30.2	36.0	53.8	67
36	170.0	17.7	10.4	152.3	75
37	255.0	78.9	31.7	176.1	71
38	99.8	37.2	37.3	62.6	67
39	48.0	21.4	44.6	26.6	67
40	69.0	5.1	7.4	63.9	74

* subbasins 17, 18, 19 were not defined in the Urban Hydrology Study Watershed Modelling-HYMO report (Env.Can., 1985)

5.5 "Lumped" Hydrologic Modelling

The discretized basins outlined in Table 5-2 were grouped, or lumped, to derive six (6) rather than 39 subcatchments shown in Figure 4-1. Five of these (301 to 305) were used in the lumped model. Flow points in the model are provided at the Water Survey of Canada (WSC) gauging stations at Donovans, Mount Pearl and Kilbride. Two lumped subcatchments were used to provide flows on South Brook.

The lumped model was assembled for two purposes:

- the main purpose was to generate a full sequence of streamflows from 1959 to the present - to identify peak flow periods in the 15 years before WSC streamflow gauges were in operation in the basin (1959 to 1973). The value of this work is that flood peaks from these peak flow periods (the focus of later, detailed modelling) double the amount of data used later in statistical analyses to estimate 1:20 and 1:100 year flood flows.

- a second purpose was to begin the model calibration-verification procedure in an economical manner (using 5 vs. 39 detailed catchments). The detailed, discretized modelling described later benefits from this strategy by beginning with a relatively well-defined range of calibration factors.

The year 1959 was selected as the beginning year for simulation as it was the first year that detailed precipitation began to become available in the St. John's area.

5.5.1 Model Parameters

Hydrologic-soil cover complexes for each lumped subcatchment were calculated from the CN values derived for the discretized basins. Impervious areas were combined with pervious areas to obtain weighted CN values. These are provided in Table 5-3, along with the drainage areas of each lumped catchment.

The longest river length and corresponding difference in elevation for each lumped subcatchment was measured from topographic mapping. These values are also shown in Table 5-3, and were input to the QUALHYMO model for calculation of hydrograph peak and shape parameters (K and Tp) using the Manning's equation. These parameters can also be computed externally to the model and provided as inputs.

Preliminary estimates of soil moisture parameters (SMIN, SMAX and SK) were obtained using the procedure outlined in Appendix 1.0 (transformation of CN values to SMIN, SMAX, SK). These estimates are also given in Table 5-3.

Estimates of water loss to deep storage in the basin were then obtained by conducting an annual water balance for the period 1982-1985 at the Donovans, Mount Pearl and Kilbride flow gauges. Annual estimates of precipitation and evapotranspiration were obtained from Environment Canada (AES), while annual estimates of streamflow runoff at the three site were obtained from WSC streamflow publications. The result of the analysis showed that deep storage losses are not a significant factor in the water budget of the Waterford River.

Lastly, preliminary estimates of the baseflow recession constant were developed by analyzing the recession of various observed hydrographs at Donovans, Mount Pearl and Kilbride.

5.5.2 Calibration Procedure

All of the above, preliminary estimates of model parameters were input to the QUALHYMO lumped model to initiate the calibration procedure. Three basic steps were then followed to develop parameters which correctly defined hydrograph shapes and volumes.

TABLE 5-3

LUMPED MODEL - INITIAL INPUT PARAMETERS

WATERFORD RIVER BASIN

Sub-Catch Number	Drainage Area (Km^2)	CN Value	Longest River Length (Km)	Difference Elevation (m)	Unadjusted K (hr)	Unadjusted Tp (hr)	Unadjusted Smin (mm)	Unadjusted Smax (mm)	SK
301	11.35	72	5.7	102	1.26	1.23	24	575	0.055
302	5.28	76	3.3	75	0.85	0.71	15	450	0.055
303	15.63	76	8.4	202	1.15	1.31	15	450	0.055
304	10.63	70	7.8	144	1.32	1.29	26	600	0.055
305	9.89	76	7.8	171	1.15	1.17	15	450	0.055

TABLE 5-4

LUMPED MODEL - FINAL CALIBRATION PARAMETERS

WATERFORD RIVER BASIN

Sub-Catch Number	Drainage Area (Km^2)	CN Value	Longest River Length (Km)	Difference Elevation (m)	Adjusted K (hr)	Adjusted Tp (hr)	Adjusted Smin (mm)	Adjusted Smax (mm)	Adjusted SK
301	11.35	72	5.7	102	2.35	2.45	50	950	0.055
302	5.28	76	3.3	75	1.69	1.52	45	900	0.055
303	15.63	76	8.4	202	2.30	2.61	45	900	0.055
304	10.63	70	7.8	144	2.65	2.58	53	980	0.055
305	9.89	76	7.8	171	2.30	2.34	45	900	0.055

Shape of the Recession Limb

The following series of long term runoff periods were selected for this purpose, because they contained an uninterrupted record of both streamflow and precipitation data:

Donovans:	1 Sept - 20 Oct 1982
Mount Pearl:	1 Sept - 20 Oct 1982
Kilbride:	1 July - 21 Dec 1983
	1 Dec/84 - 15 Jun 1985

Various base flow recession constants were tested until the shape of the recession limb was in close agreement with observed hydrographs. It was determined that a constant of 0.000007 mm/sec/mm provided fairly good agreement. During this phase of the calibration, no attempt was made to match the volumes with the observed hydrographs.

Volume of Runoff

Once the shape of the recession limb was reasonably established, initial estimates of SMIN and SMAX were adjusted to obtain correct volumes of runoff for the long-term runoff periods listed above. The adjusted SMIN and SMAX soil moisture values for each lumped subcatchment are shown in Table 5-4.

Shape of the Hydrograph

Once the shape of the recession limb and runoff volumes were correct, the shape of the hydrograph was adjusted by manipulating the K and Tp values for selected high flow events. This calibration procedure is outlined below.

5.5.3 Calibration of High Flow Periods

Rainfall

Calibration was initially undertaken with rainfall events in order to set certain model parameters before their influence is masked by the effect of the snowmelt. The period from 1981-85 was examined for calibration sequences, and the following were selected:

- 22 May 1984
- 25 May 1985

These peak flow periods were chosen because they are close to or within the period of land use selected for modelling (1984) and were monitored by the WSC at three locations in the basin.

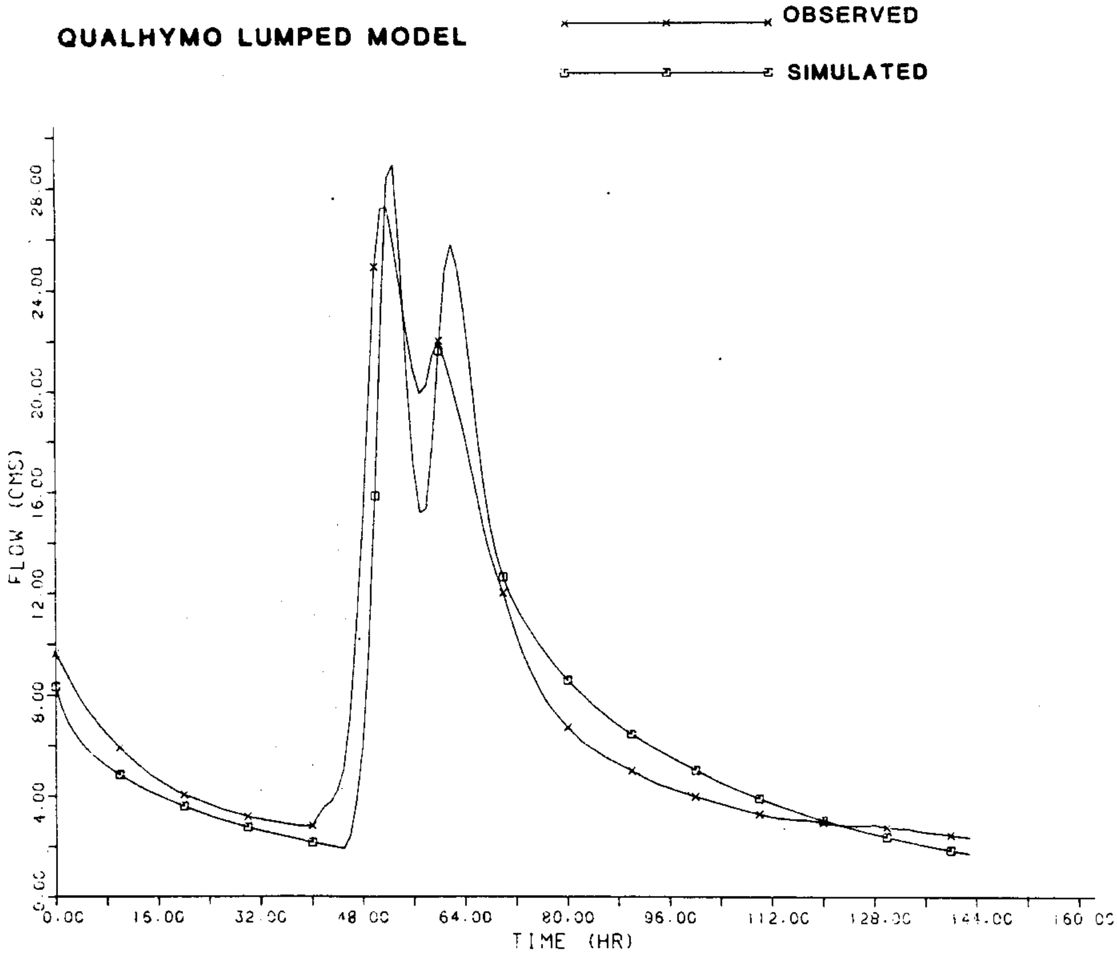
Simulations were initiated about one month before the peak flow periods, employing one hour rainfall increments and beginning with an API condition determined from previous simulations. Following several runs involving adjustments to improve the correspondence between the observed and simulated hydrographs, final values were selected for SMAX, SMIN, and other parameters. These are given in Table 5-4.

Figure 5-2a and 5-2b present plots for Kilbride (May 1984) and Mount Pearl (May 1985), showing reasonably good agreement between the simulated and observed hydrographs. Figure 5-5 shows that the peak flow at Kilbride was also accurately simulated for the latter flow period.

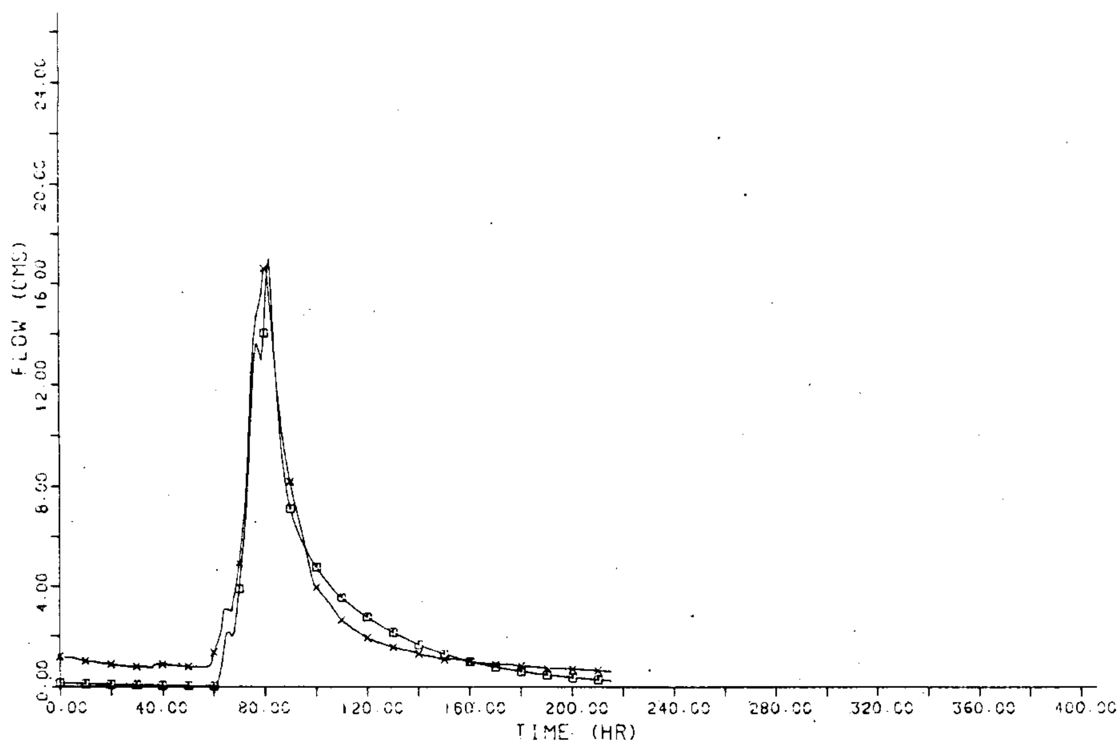
It is appropriate to make two comments at this point:

1. The current version of QUALHYMO provides values of flood peaks at many locations in a watershed, but only gives a full hydrograph at a

QUALHYMO LUMPED MODEL



KILBRIDE MAY 1984 **5-2a**



MT. PEARL MAY 1985 **5-2b**

FIGURE 5-2

single location during each simulation. Hence, in the above cases, it was only possible to show the full hydrograph for one location in the 1984 simulation (Kilbride) and the 1985 simulation (Mount Pearl).

2. No calibration or verification hydrographs were made to compare the simulated and observed flows at Donovans Industrial Park. The WSC advised that data from Donovans was less reliable than other sites on the river (see Chapter 4). Hence, the monitoring station at Mt. Pearl (which is physically close and has a similar drainage area) and the station at Kilbride (which integrates flows from both branches of the river) were selected as the only reliable WSC stations for comparing the monitored and simulated flows.

<u>Snowmelt</u>

Missing streamflow data during some snowmelt-induced high flow periods (in the years around 1984) limited the selection of good snowmelt events for calibration to:

- 10-11 April 1984

The simulation of this case was initiated in the previous fall (in November) and run continuously through the winter to account for snow accumulation/ablation. As noted in Appendix 1.0, QUALHYMO uses the NWS snowmelt model, which includes the following parameters (and their range of values from NWS guidelines):

MFMAX - maximum non-rain melt factor (0.004-0.009)
MFMIN - minimum non-rain melt factor (0.0015-0.0035)
UADJ - mean wind function value during rain or snow periods in mm/millibar (0.017-0.057)
SI - areal water equivalent above which there is always complete areal snow cover (35 mm-130 mm)

It was determined through calibration runs for this event that the best overall results were obtained with the following values:

MFMAX = 0.009
MFMIN = 0.0018
UADJ = 0.057 mm/millibar
SI = 38.0 mm

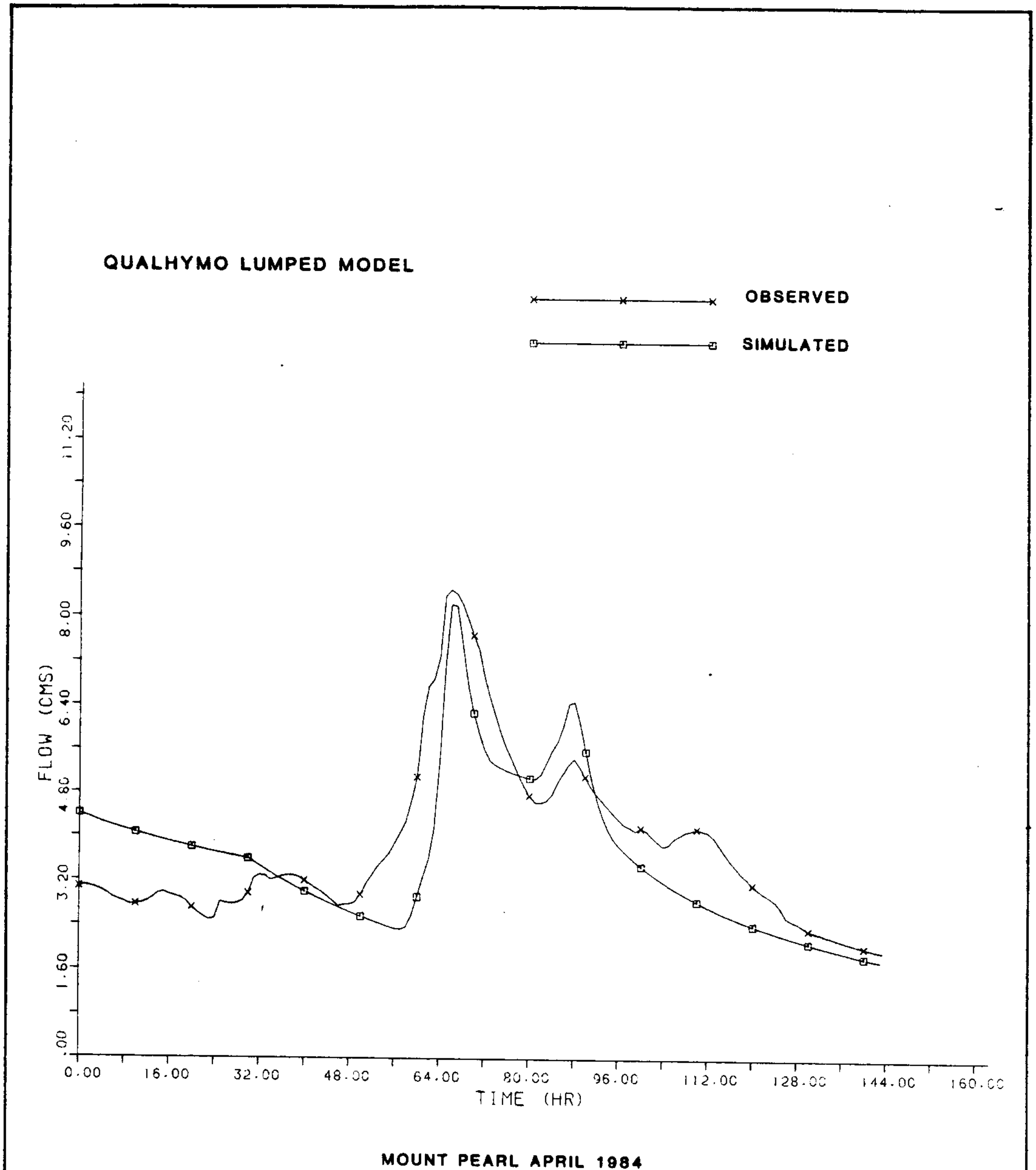
Figure 5-3 plots the calibration results for Mount Pearl. It is also noteworthy that the model also simulated the 7 February annual peak flow as 39.3 m^3/s (as compared to 36.6 m^3/sec recorded). Unfortunately, this occasion is not plotted herein because of missing WSC hourly data, for much of this event.

The calibration of this model could have been further refined. As indicated earlier, however, its main purpose is simply to provide a reasonable indication of the dates when high flows likely occurred in the past before the stream was gauged by the WSC. The results do show this reasonable agreement in time and magnitude of flood events and, hence, no further effort was spent in calibrating this model tool.

5.5.4 Verification of Lumped Modelling

The parameters established through the calibration procedures were employed in the simulation of other high flow sequences to verify the model. These sequences included both rainfall and snowmelt events:

- 26 October 1983
- 4 October 1982
- 26 November 1981
- 27 January 1978



QUALHYMO LUMPED MODEL
OBSERVED
SIMULATED
FLOW (CMS)
.00
1.60
3.20
4.80
6.40
8.00
9.60
11.20
0.00
16.00
32.00
48.00
64.00
80.00
96.00
112.00
128.00
144.00
160.00
TIME (HR)
MOUNT PEARL APRIL 1984

FIGURE 5-3

Plots of these simulations are given in Figure 5-4a to 5-4d, and in Figure 5-5, which compares observed and simulated peaks for all the verification and calibration simulations.

It must be recognized that the input data for the January 1978 case is from St. John's Airport, and not from the CDA station in the Waterford Basin. This and unknown effects of urbanization in the 7 years between the 1978 observed runoff and the 1984 base land use are considered to account for the slightly higher simulated flow estimates (about 15%).

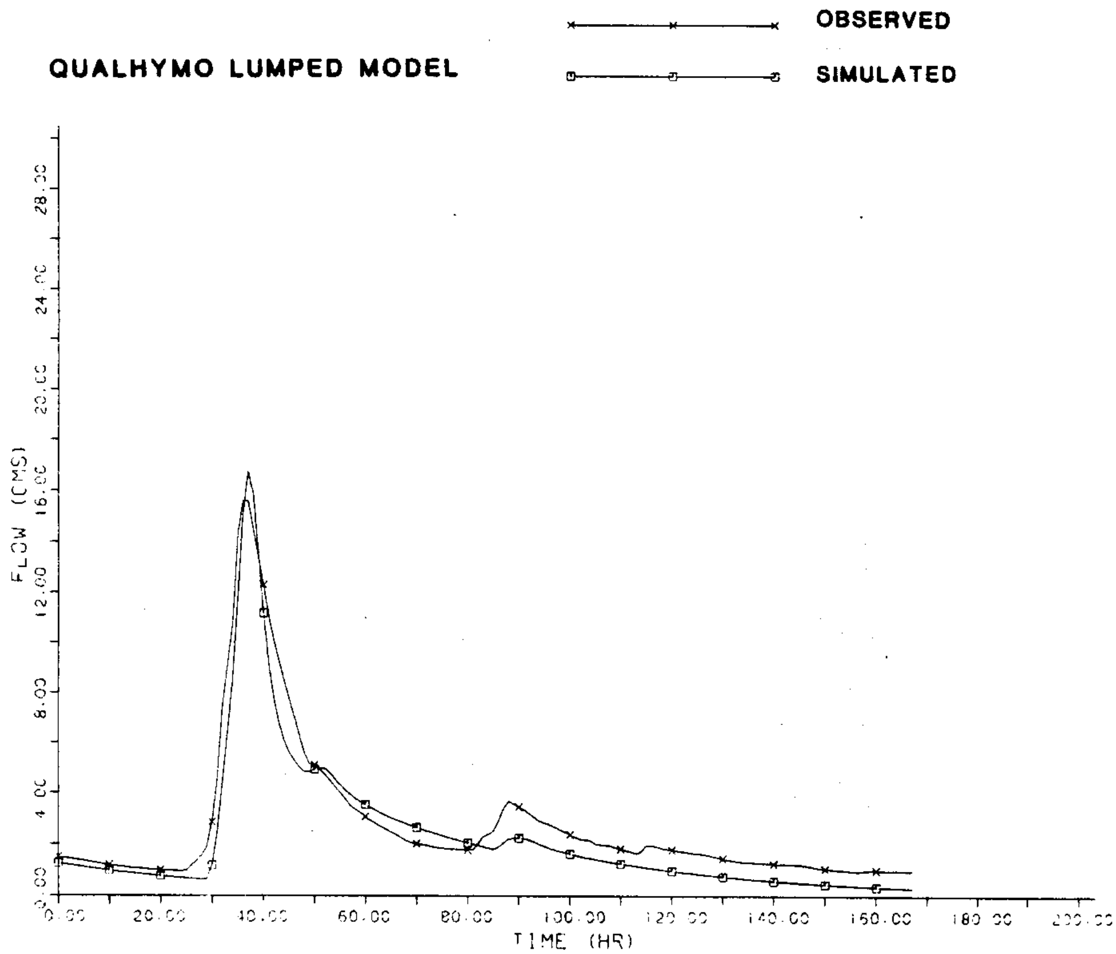
These verification figures show close agreement between observed and simulated discharges for a range of flood peaks from high to low - and provide reasonable confidence in the model's capability to identify high flow periods in years when streamflow was not monitored.

5.5.5 Identification of High Flow Periods (1959-1973)

Having proven that the lumped model identifies the peak flow periods when flows were monitored by the WSC, the next step in this study was to extend the analysis to identify these periods in the past when the dates of high flow periods were not known (i.e., from 1959 to 1973). These periods were determined by continuous simulation using the lumped model.

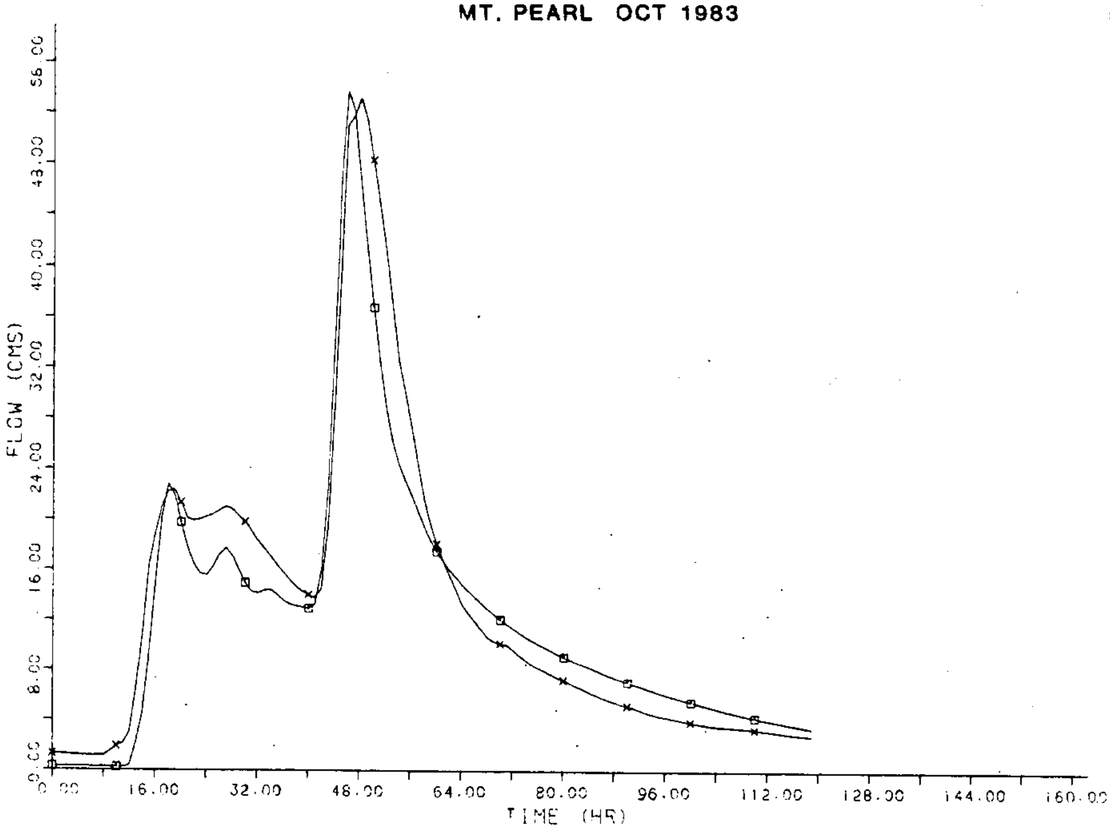
Continuous simulation of hourly streamflow was initiated on January 1, 1959 and carried through to the end of 1973 (15 years). The peak flows in each year were ranked by the model and printed for review and selection of the events for detailed, discretized modelling.

In most years, the model identified one flow period which was clearly the time of maximum discharge in that year (i.e., flood peak more than double the magnitude of the next highest peak). These obvious periods of flood flows were selected for further modelling. Other years had two or more occasions when high flows were similar (e.g., 1970). In these years, all



MT. PEARL OCT 1983

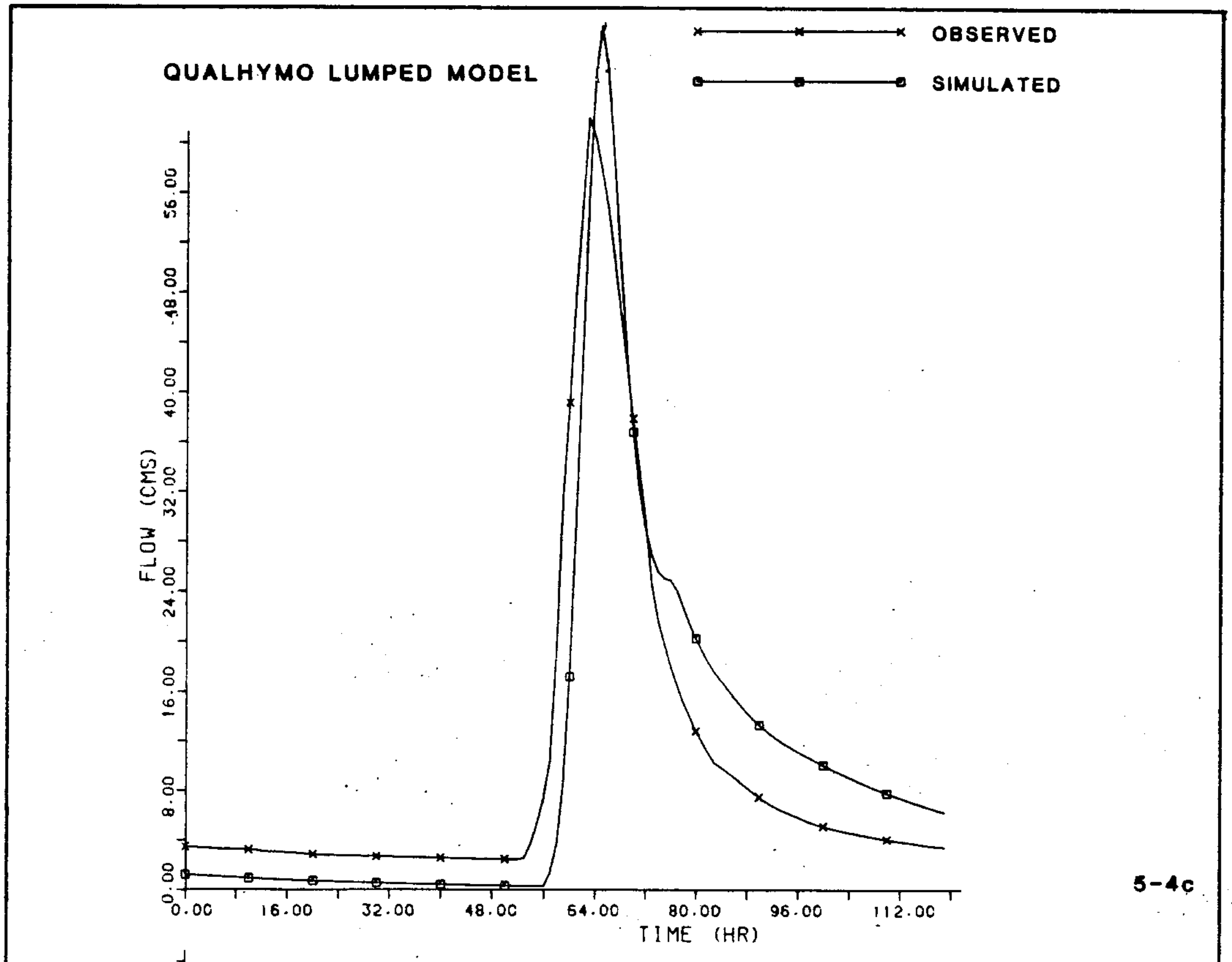
5-4a



5-4b

KILBRIDE OCT 1982

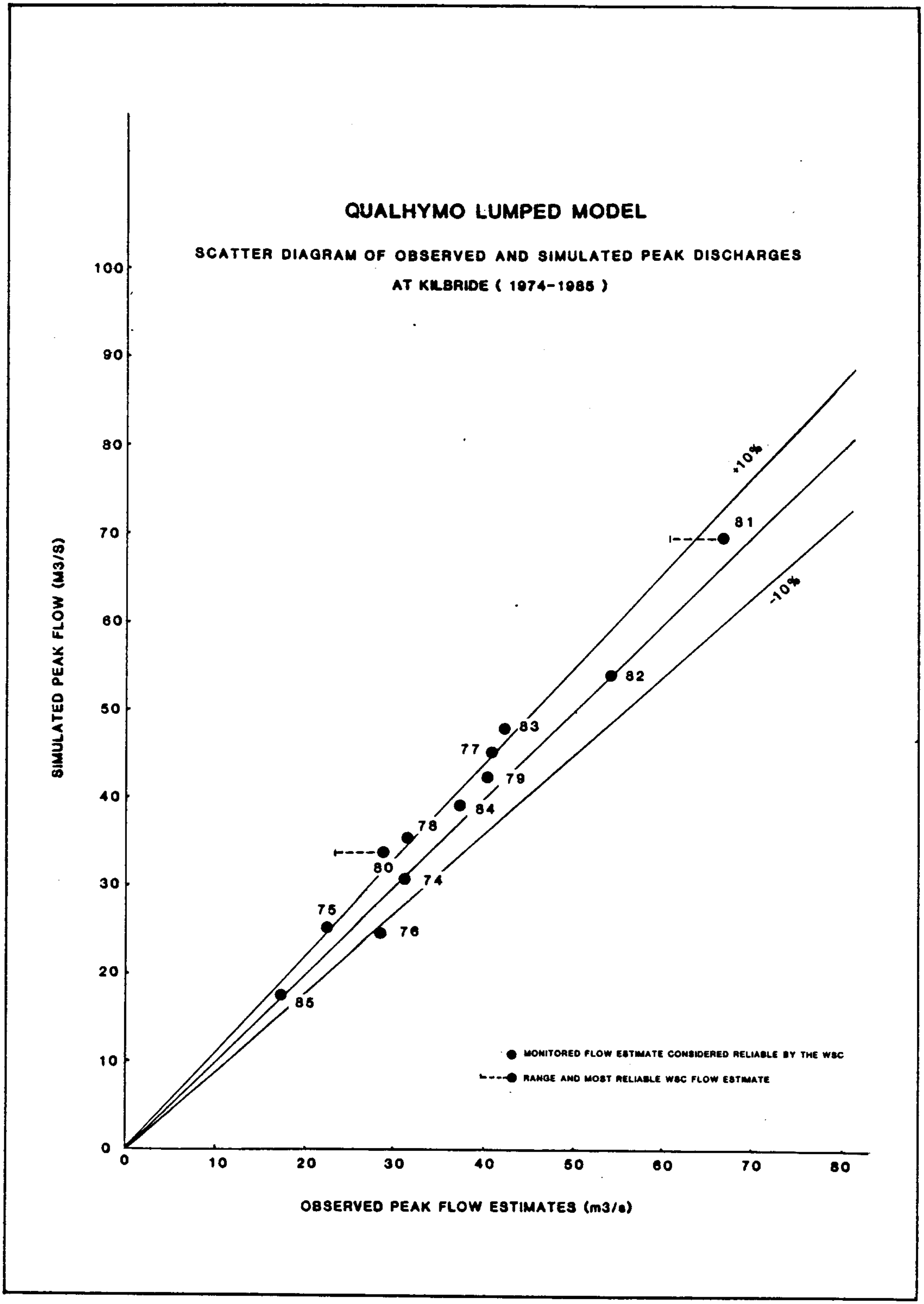
FIGURE 5-4



KILBRIDE NOV 1981

KILBRIDE JAN 1978

FIGURE 5-4



QUALHYMO LUMPED MODEL
SCATTER DIAGRAM OF OBSERVED AND SIMULATED PEAK DISCHARGES
AT KILBRIDE (1974-1985)
SIMULATED PEAK FLOW (M3/S)
OBSERVED PEAK FLOW ESTIMATES (m3/s)
0
10
20
30
40
50
60
70
80
90
100
+10%
-10%
74
75
76
77
78
79
80
81
82
83
84
85
MONITORED FLOW ESTIMATE CONSIDERED RELIABLE BY THE WSC
RANGE AND MOST RELIABLE WSC FLOW ESTIMATE

FIGURE 5-5

similar high flow occasions were retained for detailed modelling to ultimately select the highest flow period.

Table 5-5 summarizes all of the selected high flow occasions from 1959 to 1973, and Table 5-6 summarizes simulated high flow events from 1974 to the present. Noteworthy in Table 5-6 is that the lumped model correctly simulates the time of high flow periods monitored by the WSC. Each of the flood flow periods identified in these tables was simulated by the detailed, discretized model described in the following chapter.

5.6 Discretized Modelling

5.6.1 Initialization

The calibrated parameters from the lumped model were initially employed as input to the discrete model, as they represent reasonable starting estimates. The values are shown in Table 5-4, and SK (the slope of the soil moisture curve) was held constant at 0.055.

Routing reaches and flow summing points were then assembled and are shown schematically in Figure 5-6. As shown in this figure and earlier in Figure 5-1, the subcatchments are much smaller in the discretized model than those used in the lumped model. Consequently, it was decided to use 10 minute time steps for simulating rainfall runoff to ensure accurate results.

5.6.2 Model Calibration

The model calibration followed the same procedure outlined earlier for the lumped model, and employed the same rainfall and snowmelt events:

- 22 May 1984 (rainfall following snowmelt)
- 25 May 1985 (rainfall following snowmelt)

TABLE 5-5

HISTORICAL FLOOD FLOW PERIODS
ESTIMATED BY MODELLING

Year	Date*	Comment
1959	Nov. 11	• coincides with reports of flooding Nov. 10-11 and is clearly the maximum for the year
1960	Oct. 18	• coincides with only report of flooding that year (Oct.17-18)
1961	Mar. 21	• coincides with only report of flooding in 1981 (Mar. 21-22)
1962	Mar. 02 Nov. 20	• two similar flood flows. Nov. 19-20 reports describe local flooding, and there was also another (unknown) flood date in 1962 (presumed March)
1963	Dec. 04 Jan. 01	• two highest flows. Reports of Jan. 1-3 flooding identify ice jams as a factor.
1964	Aug. 05 Nov. 06	• two highest flows that year with flooding reports describing flooding on Aug. 4-5
1965	Nov. 19	• no reports of flooding that year on Waterford. This date is only high flow date at nearby Northeast Pond River, and corresponds to a major rainfall of historical note (AES) in that local area
1966	Dec. 21	• coincides with only report of flooding in 1966 (Dec. 20-22)
1967	Jan. 06	• no reports of flooding that year
1968	Aug. 22	• undated reports describe one flood in 1968, and this date is high flow date on nearby Northeast Pond River
1969	Feb. 15	• no high flow reports, but this date is clearly the highest flow period on nearby Northeast Pond River
1970	Aug. 19	• local flooding from drainage problems also reported in late-February and early March
1971	Feb. 01	• coincides with reports of flooding at several sites
1972	Nov. 11	• coincides with only report of flooding in 1972
1973	Jun. 18	• coincides with Kilbride flood and erosion reports

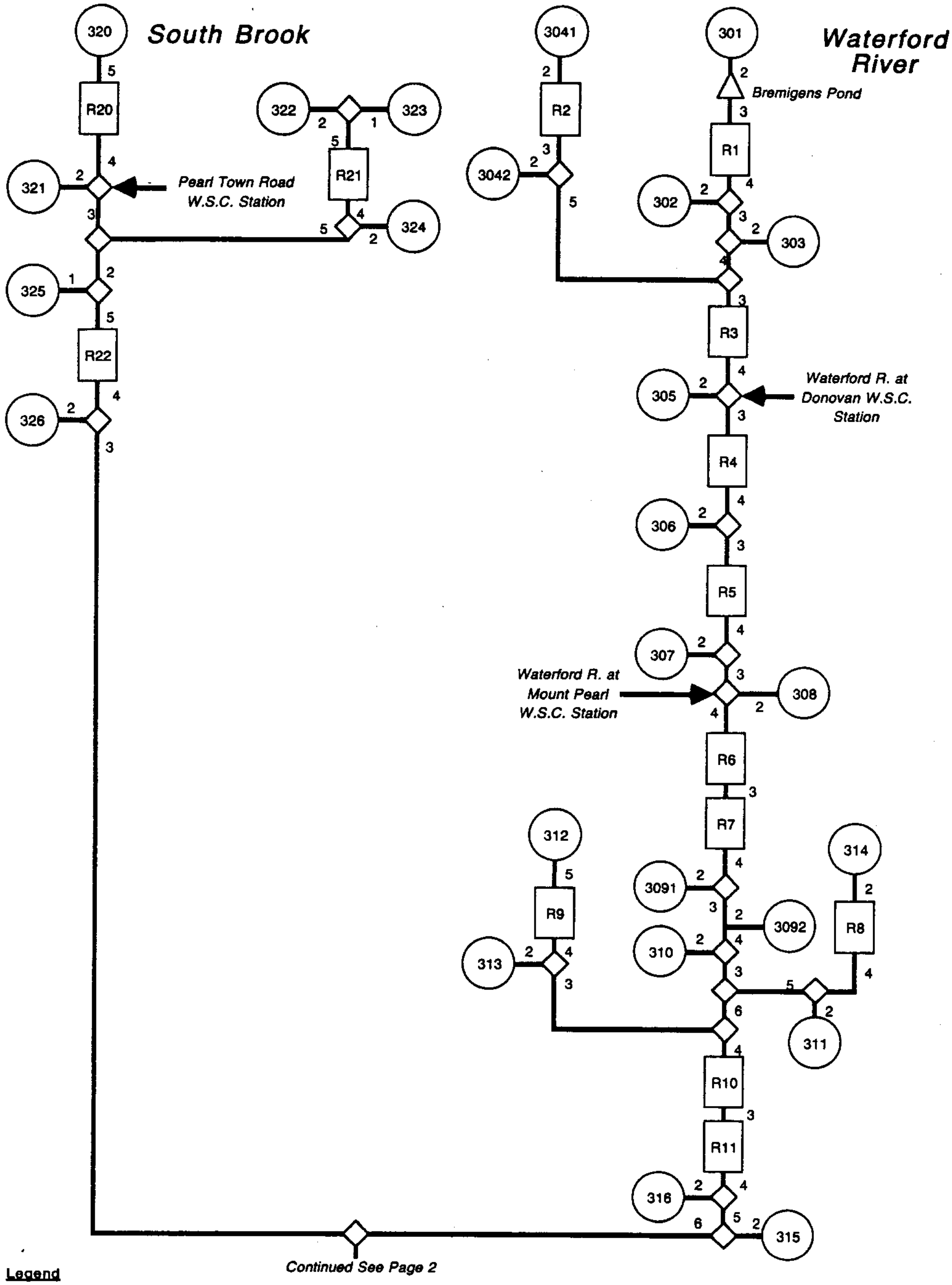
* date(s) of maximum hourly flow at Kilbride estimated by QUALHYMO model

TABLE 5-6

HISTORICAL FLOOD FLOW PERIODS
FROM SIMULATIONS AND MONITORING*

Year	Date**	Comment
1974	Aug. 31 Dec. 11	• coincides with dates of maximum instantaneous and maximum daily flows, respectively
1975	Aug. 23	• coincides with data of maximum flood flow
1976	Jan. 09 Dec. 29	• coincides with date of max. flood flows and a later, projected date of high flows
1977	Dec. 27	• coincides with date of maximum flow and only report of flooding
1978	Jan. 27 Dec. 19	• coincides with dates having highest daily flows and maximum flow (Jan. 27)
1979	Jan. 29	• coincides with maximum flow and flood reports
1980	Oct. 07	• coincides with date of maximum flood flow
1981	Nov. 26	• coincides with extreme flood flow occasion
1982	Oct. 04	• coincides with date of maximum flows but no flooding
1984	Feb. 07	• coincides with date of maximum flood flow
1985	Mar. 25	• coincides with date of maximum daily flow
1986	Apr. 11	• coincides with date of maximum flow (estimated)
1987	Apr. 03	• (not simulated - monitored data provisional)

* Kilbride since 1974 by W.S.C.
** Simulations by QUALHYMO



Legend

101 Hydrologic Unit

R1 Routing Reach

Routing Reservoir

1 QUALHYMO Model Identification Number

Water Survey of Canada Streamflow Station

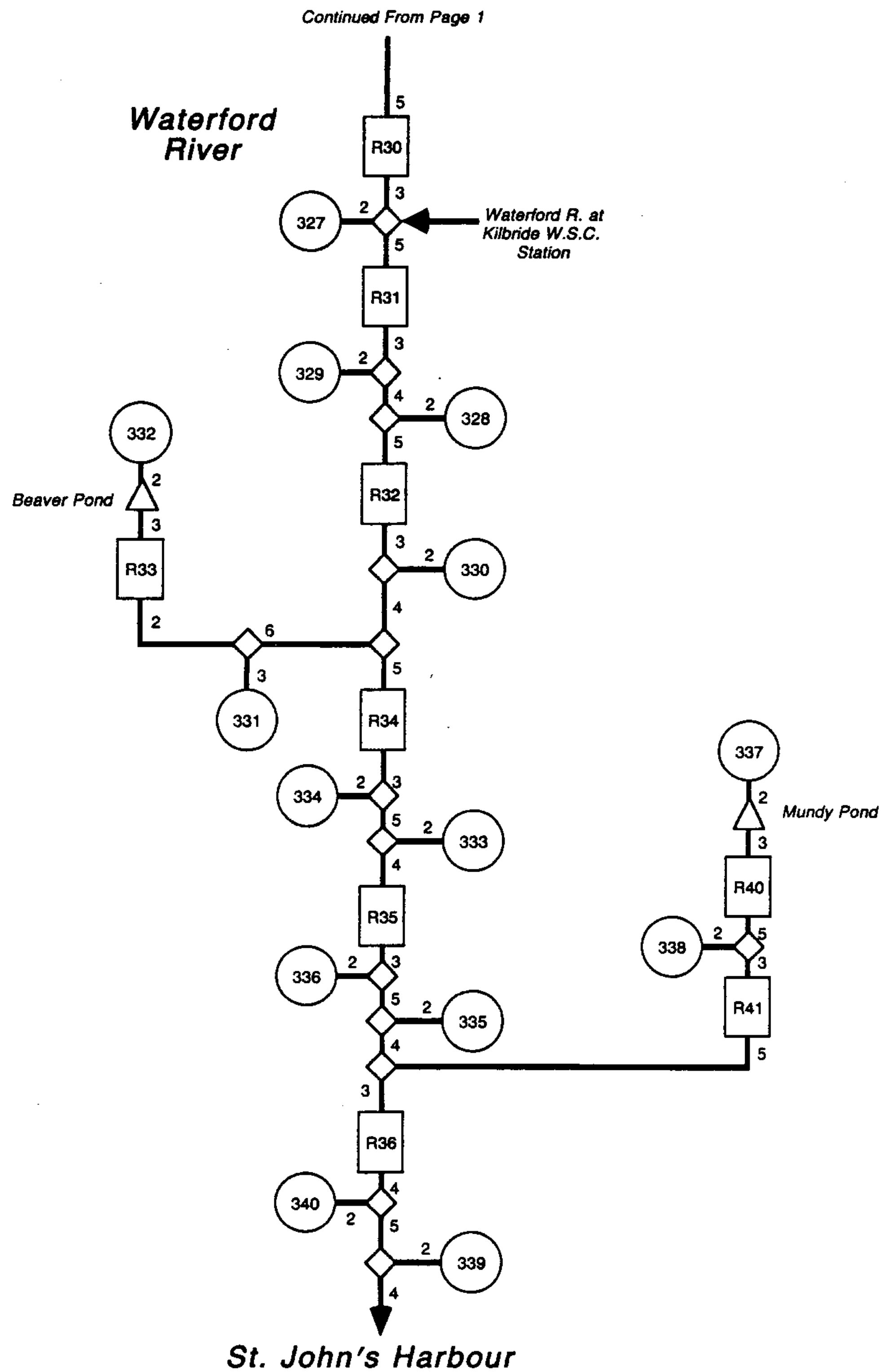
Hydrograph Addition

48160

WATERFORD RIVER BASIN
– QUALHYMO MODEL

Schematic

Page 1 of 2



Legend

101 Hydrologic Unit

R1 Routing Reach

Routing Reservoir

1 QUALHYMO Model Identification Number

Water Survey of Canada Streamflow Station

Hydrograph Addition

48160

WATERFORD RIVER BASIN
– QUALHYMO MODEL

Schematic

- 10 April 1984 (rainfall following snowmelt)

Rainfall

In working with this model, it was determined that some of the parameters had to be fine-tuned to obtain good agreement between the simulated and observed hydrographs. The following adjustments were made:

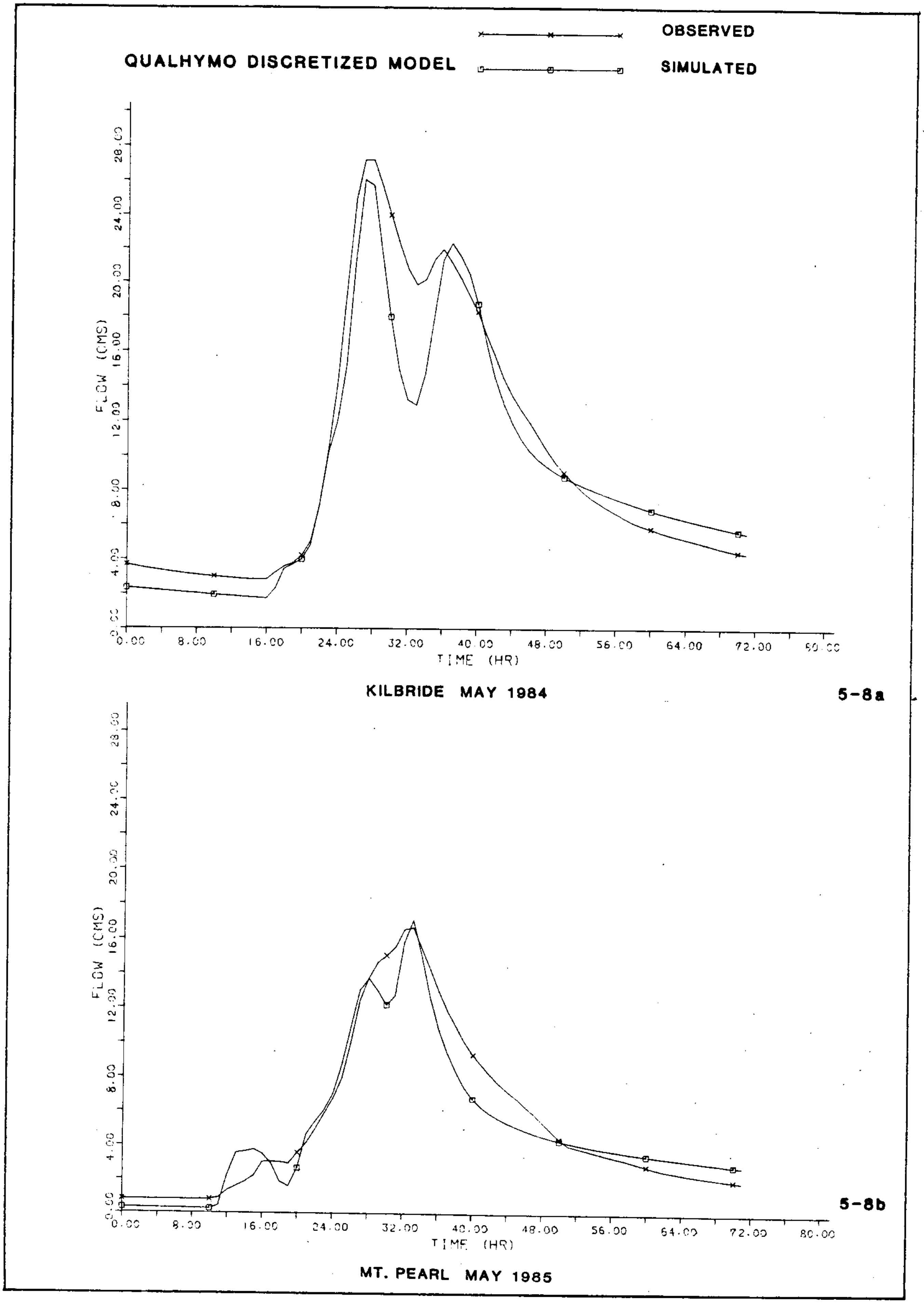
- unadjusted K and Tp values were multiplied by a factor of 3

- initial abstraction was increased from 14 mm for the lumped model to 18 mm for the discretized model.

- recession constant was reduced from 0.000007 mm/sec/mm for the lumped model to 0.000005 mm/sec/mm for the discretized model

Simulations were generally initiated about 3 to 4 days prior to each rainfall event, drawing on antecedent precipitation (API) values from previous model runs to set the initial conditions.

Figures 5-8a to 5-8b plot the observed and simulated hydrographs for the rainfall calibration cases, showing good agreement in terms of peak flow and event timing. Interesting in both cases are troughs in the simulated runoff during the peak flow period. These are caused by sharp reductions in rainfall during the storm, and would be smoothed to conform with the monitored record if rainfall data from several locations in the basin were available for model input (rather than just the single station at St. John's West CDA).



OBSERVED
QUALHYMO DISCRETIZED MODEL
SIMULATED
FLOW (CMS)
TIME (HR)
KILBRIDE MAY 1984
5-8a
FLOW (CMS)
TIME (HR)
5-8b
MT. PEARL MAY 1985

FIGURE 5-8

Snowmelt

As before for the lumped model, the snowmelt (with rainfall) event of 10-11 April 1984 was selected to calibrate the discretized model. Model runs were initiated on December 1, 1983 and carried through in 10 minute time steps to April 20, 1984.

The melt parameters given earlier in Section 5.5.2 were tested and modified along with other parameters. It was concluded that the best results were obtained with the same final values given earlier for the lumped model.

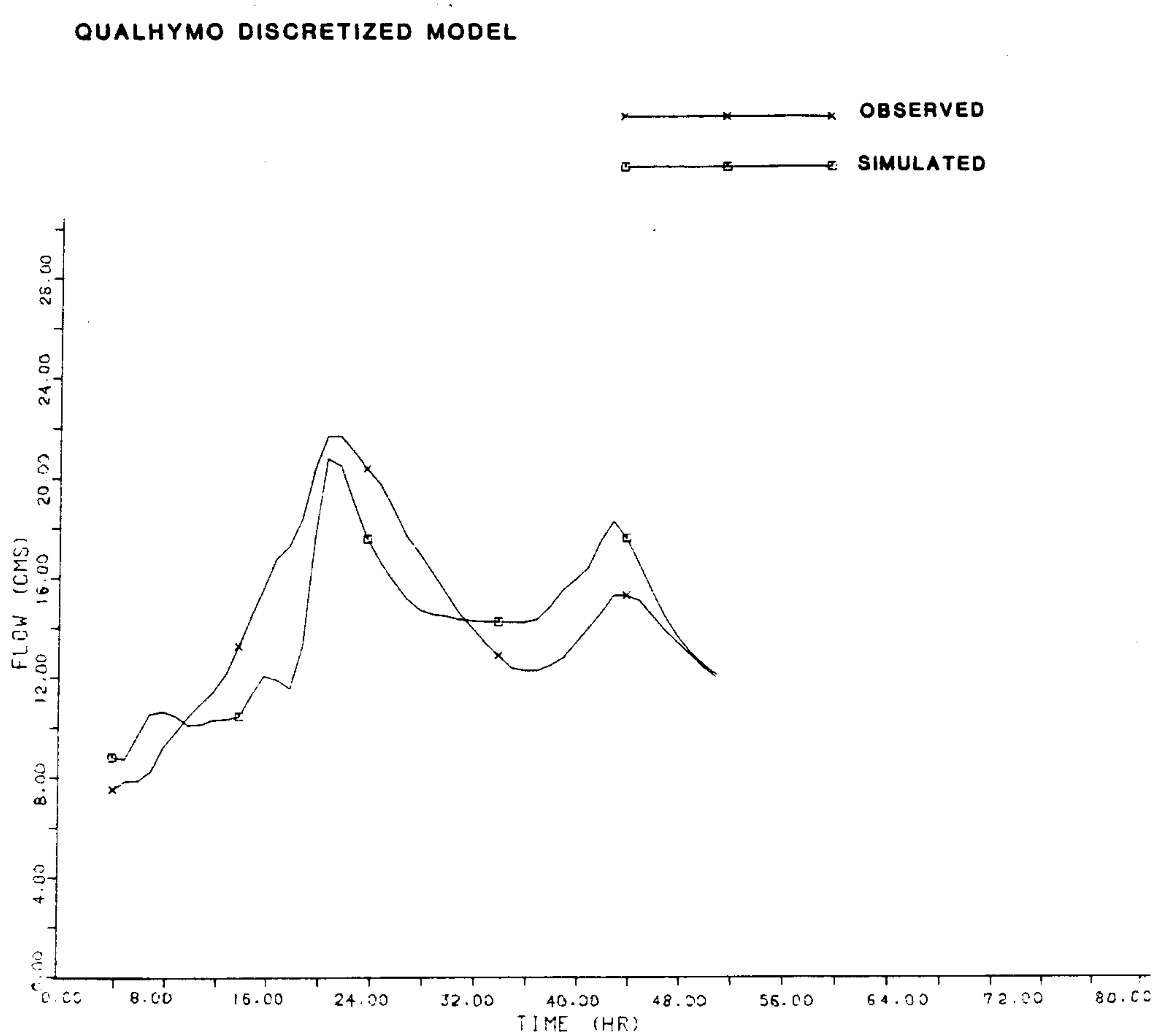
Figure 5-9 compares the modelled and observed hydrographs for this event. As indicated earlier, the annual maximum event occurred on February 7, 1984, when streamflow data is not available for plotting purposes. Water Survey records note that the maximum instantaneous flow at Kilbride was 36.6 m^3/s. The model simulates 32.6 m^3/s or within 12% of the observed.

Table 5-7 lists the final estimates of various model parameters determined from the model calibration.

5.6.3 Model Verification

Using the parameters established through the calibration of the rainfall and snowmelt events identified earlier, other events were simulated to verify the discretized model. Some examples of rainfall events which are plotted to illustrate the model verification are shown in Figures 5-10 for:

- 26 November 1981
- 4 October 1982
- 26 October 1983



KILBRIDE APR 1984

FIGURE 5-9

TABLE 5-7

WATERFORD RIVER HYDROTECHNICAL STUDY

FINAL ESTIMATES OF

SUBCATCHMENT AREAS, IMPERVIOUSNESS AND

COMPLEX NUMBER (CN) MODEL INPUT DATA

Sub-Basin No.	Area (ha)	Imp. Area (ha)	Imp. Area (%)	Perv. Area (ha)	Weighted CN (AMC II)	Smin (mm)	Smax (mm)	K (hrs)	Tp (hrs)
1	193.0	6.3	3.3	186.7	65	58	1385	1.8	1.4
2	268.0	36.4	13.6	231.6	66	57	1316	2.0	1.6
3	228.0	15.1	6.6	212.9	72	50	950	2.1	1.6
4.1	293.6	63.4	21.6	230.2	63	60	1531	2.1	1.7
4.2	46.4	26.5	57.1	19.9	67	56	1248	1.1	0.6
5	106.0	14.2	13.4	91.8	73	49	896	1.4	1.0
6	177.7	18.7	10.5	159.0	70	53	980	1.7	1.3
7	216.8	34.3	15.8	182.5	73	49	896	1.5	1.0
8	133.0	49.3	37.1	83.7	67	56	1248	1.4	1.0
9.1	160.8	23.2	14.4	137.6	65	58	1385	1.0	1.0
9.2	73.4	31.2	42.5	42.2	68	55	1255	1.7	1.0
10	166.2	51.0	30.7	115.2	71	51.5	965	1.9	1.4
11	179.1	11.4	6.4	167.7	71	51.5	965	0.6	0.7
12	298.5	66.5	22.3	232.0	69	52	1134	2.4	1.9
13	110.2	2.0	1.8	108.2	67	56	1265	1.9	1.3
14	160.9	1.5	0.9	159.4	73	49	896	1.1	1.0
15	329.4	128.2	38.9	201.2	67	56	1248	2.0	1.6
16*	84.2	21.0	25.0	63.2	66	57	1316	1.9	1.2
20	547.3	0	0	547.3	67	56	1248	2.8	2.5
21	515.3	42.4	8.2	472.9	74	48	841	3.6	2.9
22	109.0	2.0	1.8	107.0	79	42	603	5.4	2.4
23	221.8	0.8	0.7	221.0	75	47	789	2.0	1.6
24	175.3	30.0	17.1	145.3	72	50	950	1.3	1.1
25	135.5	2.7	6.5	38.6	72	50	950	2.3	1.6
26	306.1	46.3	15.1	259.8	70	53	980	1.3	1.3
27	41.3	2.7	6.5	38.6	69	52	1134	0.4	0.4
28	17.0	6.5	38.0	10.5	67	56	1248	0.9	0.4
29	71.0	0.85	1.2	70.2	67	56	1248	0.5	0.5
30	19.5	63.3	32.0	131.7	67	56	1248	1.1	1.0
31	79.0	3.6	4.5	75.4	68	55	1255	0.4	0.5
32	191.0	2.3	1.2	189.7	78	43	648	1.25	1.1
33	56.0	20.2	36.0	35.8	67	56	1248	0.75	0.55
34	37.0	3.0	8.0	34.0	68	55	1255	0.41	0.37
35	84.0	30.2	36.0	53.8	67	56	1248	1.05	0.76
36	170.0	17.7	10.4	152.3	75	47	749	0.61	0.70
37	255.0	78.9	31.7	176.1	71	51.5	965	1.55	1.32
38	99.8	37.2	37.3	62.6	67	56	1248	1.60	0.96
39	48.0	21.4	44.6	26.6	67	56	1248	0.56	0.41
40	69.0	5.1	7.4	63.9	74	48	841	0.37	0.41

* subbasins 17, 18, 19 were not defined in the Urban Hydrology Study Watershed Modelling-HYMO report (Draft 1985)

An example of snowmelt (with rainfall) verification is given in Figure 5-11 for:

- 27 January 1978

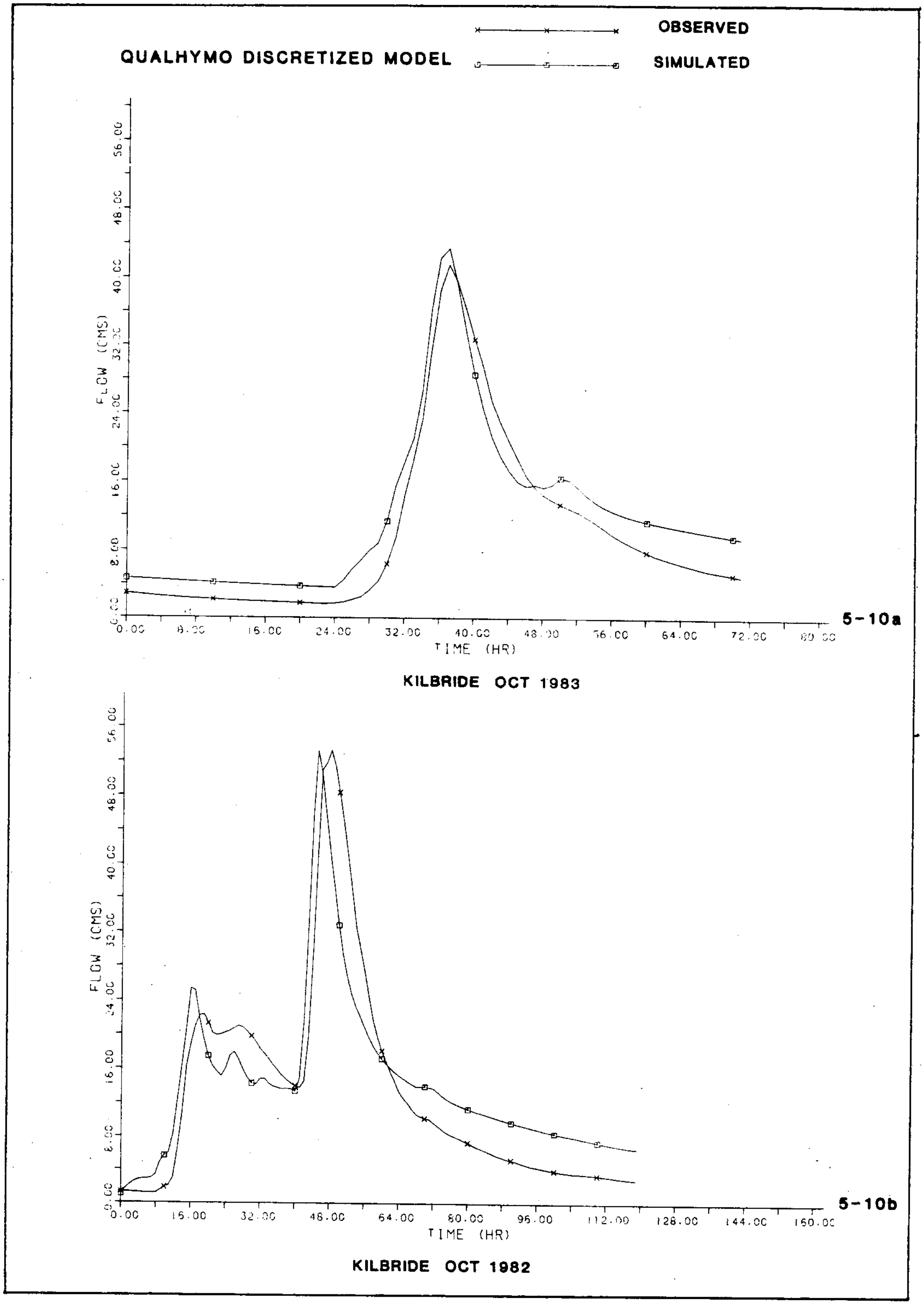
As was noted earlier, the difference between observed and simulated in this latter case is credited to use of rainfall input from St. John's Airport (outside of the basin) and urbanization - which may have affected all of the above results.

Figure 5-12a plots the observed and simulated peak flows at Kilbride for the calibration/ verification runs and for other peak events from 1981 to 1985 (closest years to the 1984 land use information employed by the model. The agreement is excellent. All simulated peaks are within 10% of the observed (i.e., within the projected range of error of the measured flows). Noteworthy too is that the correspondence is good for a wide range of flows - 17 m^3/s to about 60 m^3/s. Figure 5-12b presents similar results for Mount Pearl.

The fit of modelled and simulated results was then tested for the early period of gauged record at Kilbride (1974 to 1980). This was done principally to test the model sensitivity to rainfall input from St. John's Airport rather than St. John's West CDA (used for calibration/verification but not available for years from 1974-80). The results of observed and simulated high flow occasions are shown in Figure 5-13.

Figure 5-13 shows more scatter in results, as would be expected from use of a different rainfall record. Some influence of urbanization may also be a factor, and if there is a trend, it is to somewhat higher simulated peaks than were observed.

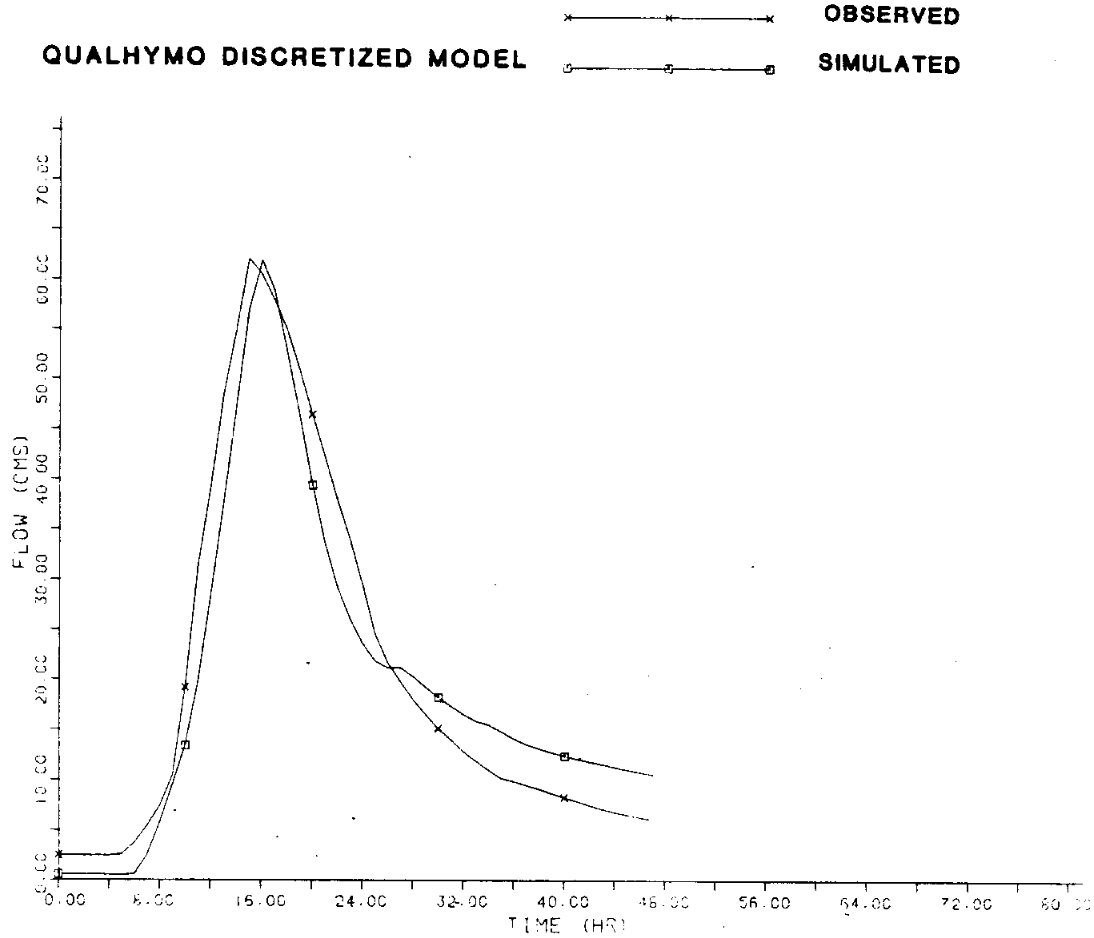
Overall, the results shown in Figure 5-12 and 5-13 support conclusions that:



OBSERVED
QUALHYMO DISCRETIZED MODEL
SIMULATED
FLOW (CMS)
0.00
8.00
16.00
24.00
32.00
40.00
48.00
56.00
TIME (HR)
0.00
8.00
16.00
24.00
32.00
40.00
48.00
56.00
64.00
72.00
80.00
5-10a
KILBRIDE OCT 1983
FLOW (CMS)
0.00
8.00
16.00
24.00
32.00
40.00
48.00
56.00
TIME (HR)
0.00
16.00
32.00
48.00
64.00
80.00
96.00
112.00
128.00
144.00
160.00
5-10b
KILBRIDE OCT 1982

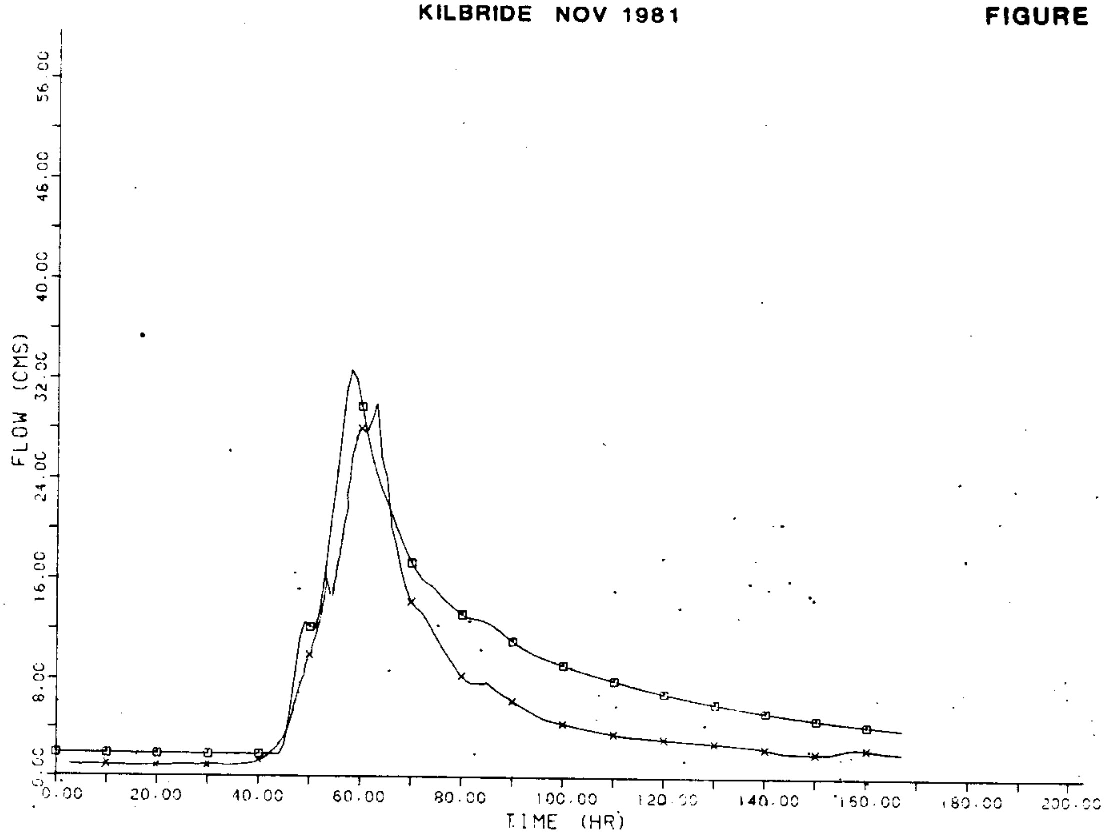
FIGURE 5-10

QUALHYMO DISCRETIZED MODEL



KILBRIDE NOV 1981

FIGURE 5-10c



KILBRIDE JAN 1978

FIGURE 5-11

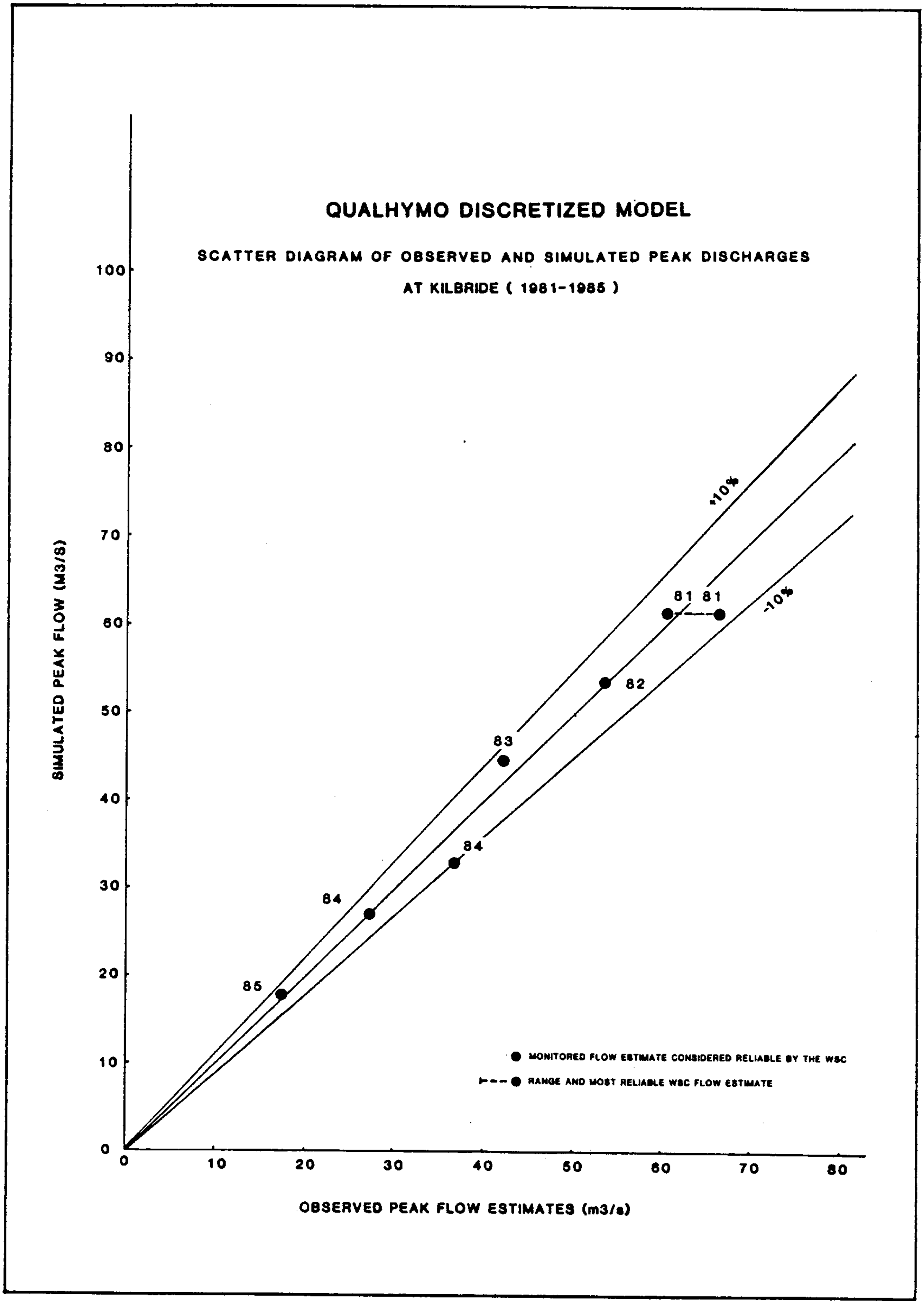

QUALHYMO DISCRETIZED MODEL
SCATTER DIAGRAM OF OBSERVED AND SIMULATED PEAK DISCHARGES
AT KILBRIDE (1981-1985)
SIMULATED PEAK FLOW (M3/S)
OBSERVED PEAK FLOW ESTIMATES (m3/s)
+10%
-10%
81 81
82
83
84
84
85
MONITORED FLOW ESTIMATE CONSIDERED RELIABLE BY THE WSC
RANGE AND MOST RELIABLE WSC FLOW ESTIMATE
0
10
20
30
40
50
60
70
80
90
100

FIGURE 5-12a

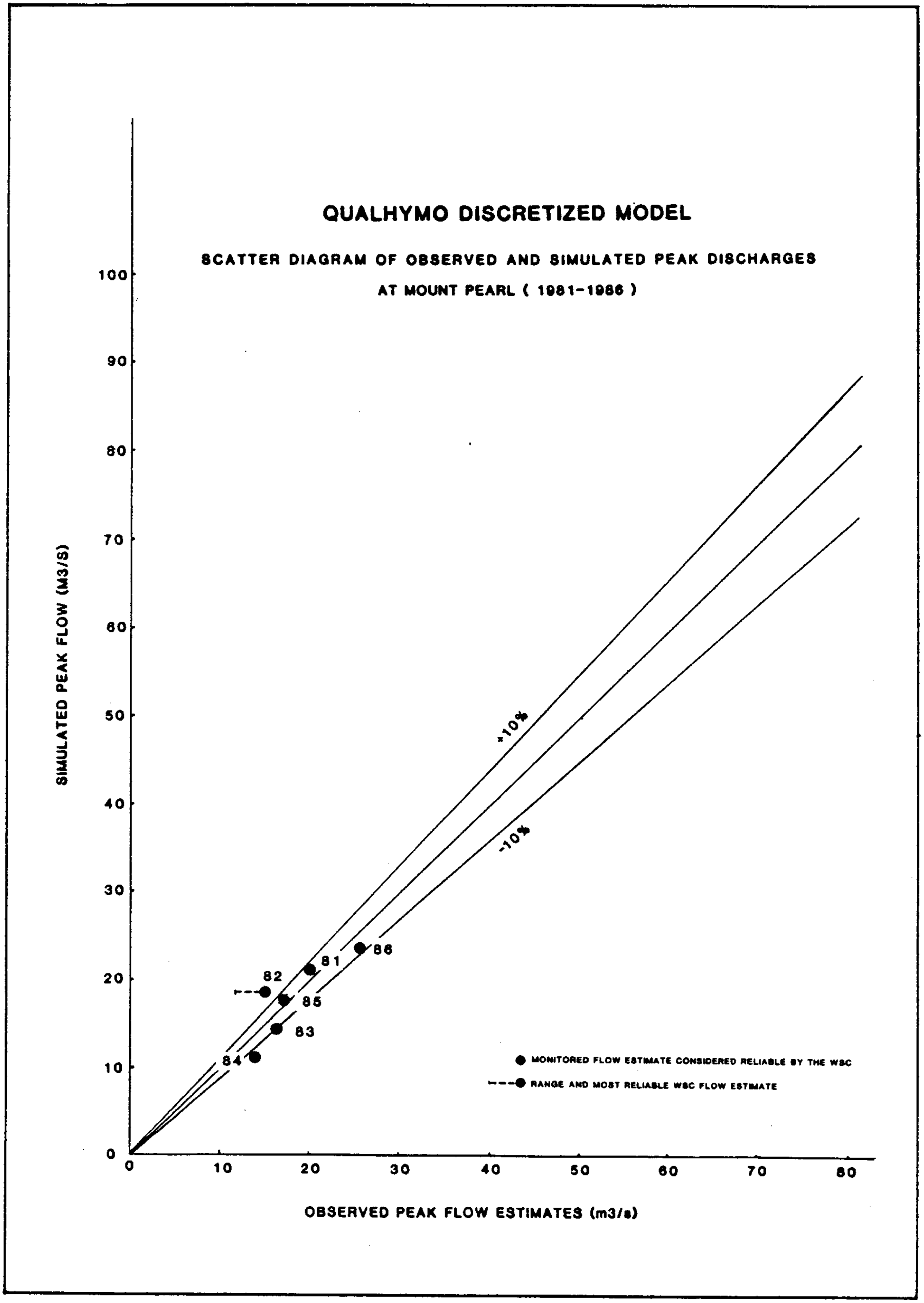

QUALHYMO DISCRETIZED MODEL
SCATTER DIAGRAM OF OBSERVED AND SIMULATED PEAK DISCHARGES
AT MOUNT PEARL (1981-1986)
SIMULATED PEAK FLOW (M3/S)
OBSERVED PEAK FLOW ESTIMATES (m3/s)
0
10
20
30
40
50
60
70
80
90
100
+10%
-10%
81
82
83
84
85
86
MONITORED FLOW ESTIMATE CONSIDERED RELIABLE BY THE WSC
RANGE AND MOST RELIABLE WSC FLOW ESTIMATE

FIGURE 5-12b

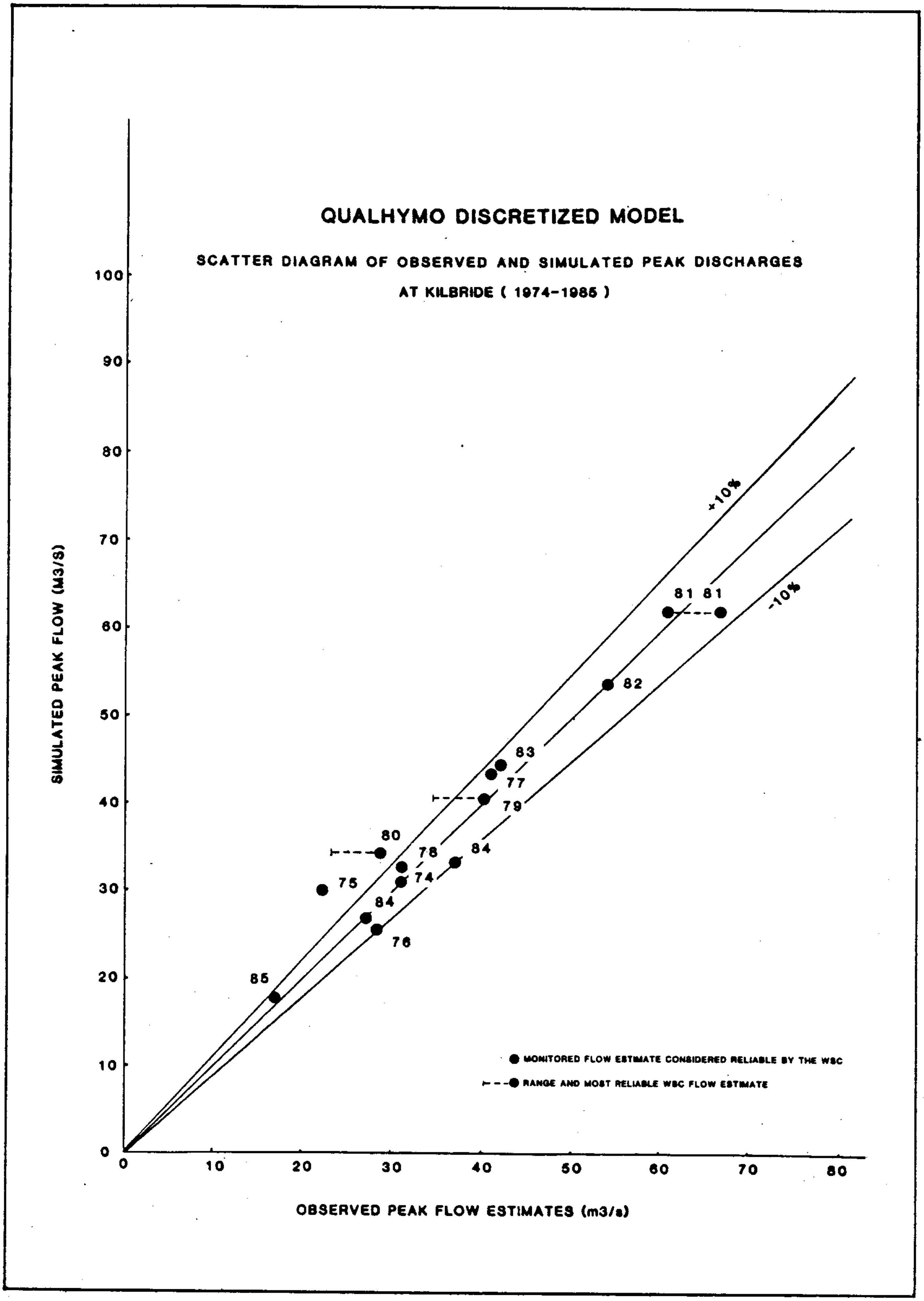

QUALHYMO DISCRETIZED MODEL
SCATTER DIAGRAM OF OBSERVED AND SIMULATED PEAK DISCHARGES
AT KILBRIDE (1974-1985)
SIMULATED PEAK FLOW (M3/S)
OBSERVED PEAK FLOW ESTIMATES (m3/s)
+10%
-10%
81 81
82
83
77
79
80
78
74
84
75
84
76
85
MONITORED FLOW ESTIMATE CONSIDERED RELIABLE BY THE WSC
RANGE AND MOST RELIABLE WSC FLOW ESTIMATE
0
10
20
30
40
50
60
70
80
90
100

FIGURE 5-13

- simulations of recent years having land use similar to that contained in the QUALHYMO model (1984) can be undertaken with great accuracy

- St. John's Airport data can be used with reasonable confidence to simulate precipitation runoff in the Waterford Basin

- the streamflow record on the Waterford River can be extended back to 1959 with QUALHYMO to derive useful data for flood flow frequency analysis based on present land use conditions

5.6.4 Simulation of High Flow Periods (1959-1986)

This stage of modelling was relatively straightforward. The calibrated/verified model parameters were employed without adjustment. Precipitation data from St. John's Airport (used for lumped runs described in Section 5.5.5) were used for all the QUALHYMO simulation, and API values were taken from earlier runs to initialize the model.

Each possible high flow period listed in Tables 5-5 and 5-6 was simulated and the peak flow was determined for each year. These values are listed in Table 5-8 for the Water Survey site at Kilbride.

5.7 Frequency Analysis of Flood Flows

5.7.1 Analysis of Modelled Data

The results provided in Table 5-8 include several years when more than one event was simulated (e.g., 1961). This was done because earlier modelling (described in Section 5.5.5 using the lumped model) identified that some years had more than one flow period which could be the annual peak. Simulation of all these occasions with the calibrated and verified discretized model clearly separates the secondary peaks from the annual peaks. The annual peaks are flagged by an asterisk in the table for those years when more than one event was simulated.

TABLE 5.8

SUMMARY OF SIMULATED PEAK FLOWS
WATERFORD RIVER AT KILBRIDE (1959-86)

Date	Observed Peak Flow at Kilbride (m^3/s)	Simulated Peak Flow at Kilbride Using Discretized Model (m^3/s)
11 Nov. 1959	-	71.04
18 Oct. 1960	-	55.76
26 Mar. 1961	-	29.31(*)
09 May 1961	-	25.58
02 Mar. 1962	-	43.86(*)
20 Nov. 1962	-	35.91
01 Jan. 1963	-	24.24
04 Dec. 1963	-	29.31(*)
05 Aug. 1964	-	48.42
06 Nov. 1964	-	66.44(*)
19 Nov. 1965	-	31.81
21 Dec. 1966	-	61.40
06 Jan. 1967	-	47.59
22 Aug. 1968	-	28.31
15 Feb. 1969	-	34.43
19 Aug. 1970	-	57.23
01 Feb. 1971	-	87.99
11 Nov. 1972	-	54.77
18 Jun. 1973	-	33.83(*)
12 Aug. 1973	-	28.72
11 Dec. 1974	-	28.99
31 Aug. 1974***	30.9	31.27(*)
23 Aug. 1975	21.8	30.09
09 Jan. 1976	28.3	24.40(*)
28 Dec. 1976	-	23.46
27 Dec. 1977	40.2	41.73
27 Jan. 1978	30.9	32.00
19 Dec. 1978	-	36.56(*)
29 Jan. 1979	34.50	38.00
07 Oct. 1980	22.7 - 28.2**	32.43
26 Nov. 1981	62.1	62.19
04 Oct. 1982	53.4	54.66
26 Oct. 1983	41.7	42.63
07 Feb. 1984	36.6	37.38
25 May 1985	n/a	53.14
11 Apr. 1986	62.7(**)	71.49

* Annual Peak Flow event
** Estimated peak flow or range
n/a Peak flow not recorded
*** Simulated peaks from this and subsequent years coincide in time with monitored flows at Kilbride

Values in Table 5.8 illustrate only the results at Kilbride. Similar results were also derived at 18 flow points within the watershed and along the river course. Those along the river are of particular value for establishing flow points for use in backwater modelling, whereas other points in the basin may be used for determining discharges for smaller tributary areas.

All of the annual flood flows at each of the 18 flow points were subject to frequency analyses to determine the 1:20 year and 1:100 year flood flows. The Consolidated Frequency Analysis Package (Inland Waters Directorate, Environment Canada; 1985) was employed to develop probability distributions, which included:

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

Appendix 2.0 presents the tabular and graphical output of each analysis.

The Generalized Extreme Value distribution (GEV) initially provides several relevant statistics on the distribution of the annual maximum flood. These indicate that the distribution has a lower bound and a positive skew, giving the upward curving shape without an upper bound. The coefficients of skew and kurtosis indicate that a three-parameter distribution is appropriate (e.g., 3PLN) and that a two-parameter distribution (e.g., Gumbel I) is not appropriate.

Fitting of the Three-Parameter Log Normal distribution to the transformed data provides coefficients of skew (-0.189) and kurtosis (2.556) for comparison with the theoretical values of 0.0 and 3.0 respectively. Both statistics are reasonably close to their anticipated values. The coefficient of skew (-0.189) indicates a more non-symetric distribution than is observed in most Atlantic provinces and Newfoundland at large. However, its value is very close to those determined for stations which are on or near the Avalon Peninsula and which were evaluated in the Regional Flood Frequency Analysis for the Island of Newfoundland (e.g., Northwest Brook, CS = -0.197), (Reference: Canada-Newfoundland Flood Damage Reduction Program, 1984).

The moment coefficient of kurtosis (2.556) is also lower than for many Newfoundland rivers, indicating a relatively uniform gradation in the peak flow sequence. However, as above, this lower than average value is also seen in other rivers in or near the Avalon Peninsula.

The Log Pearson III distribution (LP3) for the flood series employs three parameters to define the shape of the distribution. Unfortunately, it is difficult to assess how closely this distribution matches the data because theoretical values for coefficients of skew and kurtosis are not defined. This assessment must be done visually and by employing statistical tests for goodness of fit (e.g., Kolmogorov and chi-square tests). It is evident from figures in Appendix 2.0 that this distribution almost fits the data as well as does the 3PLN distribution.

Last, the Wakeby distribution employs five parameters to allow the lower and higher flows to be modelled separately. The statistics indicate that the distribution is a valid one for this data, but flag that this distribution gives an upper bound at 126 m^3/s. This is not large by comparison to the observed and estimated peak flows and appears to be affecting the shape of the frequency curve in the range of our interest (i.e., 1:20 year to 1:100 year).

Table 5.9 summarizes the frequency analysis results for the above distributions at Mount Pearl and Kilbride. Other than the Wakeby distribution, which appears to underestimate peak flows because of its upper bound, there is very little difference between the GEV, 3PLN and LP3 distributions. Of these three, the 3PLN distribution provides a slightly better fit to the

TABLE 5.9

FLOOD FREQUENCY ANALYSIS
WATERFORD RIVER

1:100 YEAR RETURN PERIOD (m^3/s)

Location	Generalized Extreme Value	3PLN Distrib.	Log Pearson Type III	Wakeby Distrib.
Mount Pearl	36.70	36.20	37.20	30.40
Kilbride	118.00	118.00	122.00	94.90

1:20 YEAR RETURN PERIOD (m^3/s)

Location	Generalized Extreme Value	3PLN Distrib.	Log Pearson Type III	Wakeby Distrib.
Mount Pearl	25.70	26.20	26.10	25.40
Kilbride	81.30	83.30	83.10	79.40

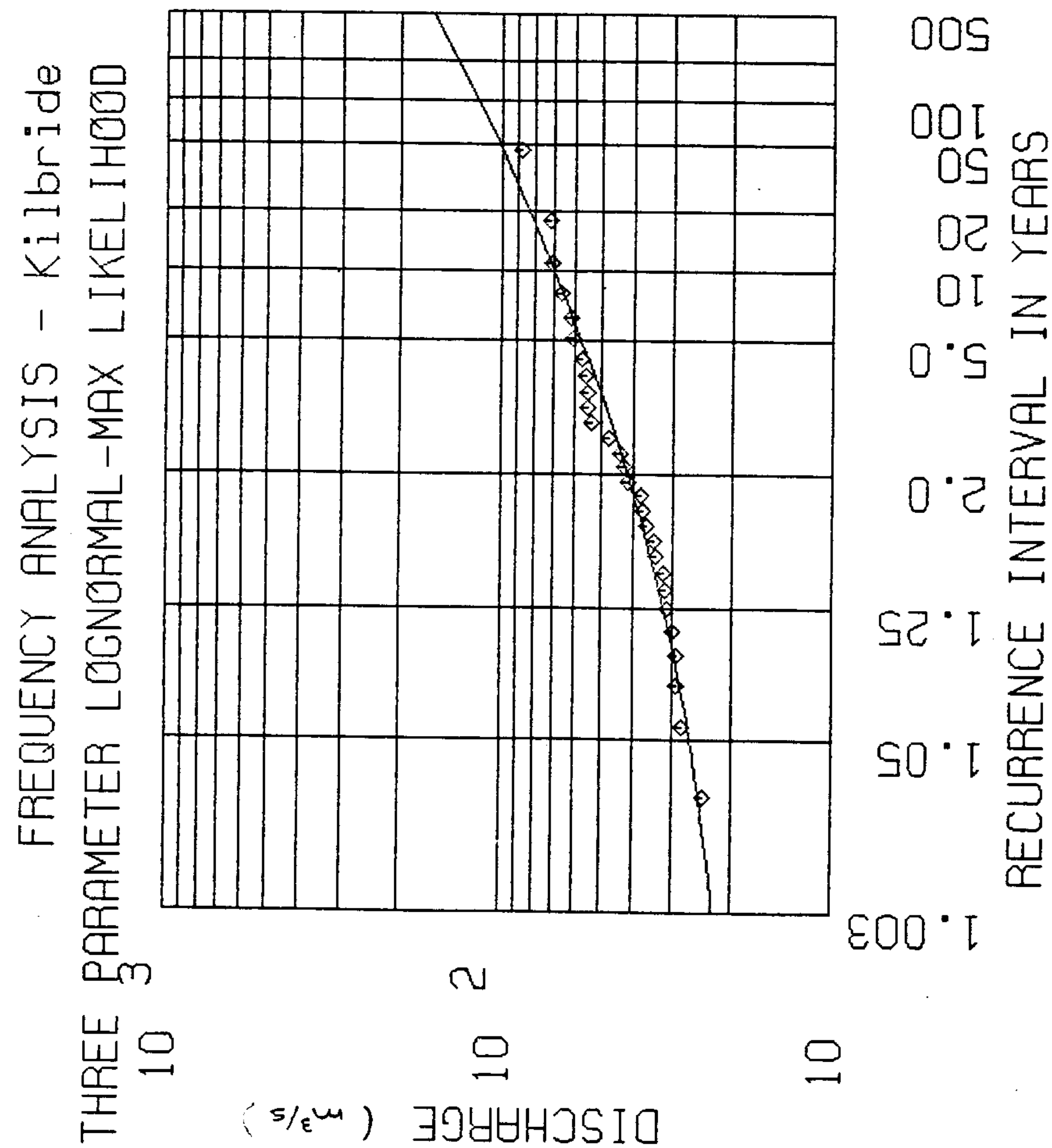
data (both graphically and statistically), and is the distribution which is best suited to match the regional statistics of other nearby rivers in the Avalon Peninsula. This latter consistency with the Regional Flood Frequency Analysis as well as reasonable fit to the data make the Three Parameter Log Normal distribution (3PLN) the distribution of choice for determining flood flow frequencies from the modelled streamflow data in the Waterford River Basin. The fit of this distribution to data at Kilbride is shown in Figure 5-14.

5.7.2 Comparison with Other Data

Two tests were conducted to determine if the selected approach using the 28 years of modelled peak flows (with 1984 land use conditions prevailing for all years) gives reasonable results.

The first test applied the 3PLN, GEV and LP3 methods to compute flood frequencies using the observed flow data at Kilbride. The three methods give similar results (Appendix 3.0) and the 3PLN distribution gives a 1:100 year flow of 104 m^3/s and a 1:20 year flow of 70.6 m^3/s. Both of these return period flood flows are within the range of the above-described results using 28 years of data but are lower. This is expected because of missing high flow periods in the short record of the monitored data (e.g., 1959), and perhaps because of the effect of urbanization (e.g. decreases in pervious areas which hold moisture in the soil, more rapid runoff through storm sewers and channels, etc.). These latter effects would be expected to increase flood peaks above those monitored in the years prior to 1984.

The second test involved use of the regional flood frequency analysis prepared by the Canada-Newfoundland Flood Damage Reduction Program (1984). The report and users guide from that study provide a procedure for estimating peak flows based on physiographic and hydrometeorological characteristics of a watershed. Application of the approach to the Waterford basin at Kilbride yields a 1:100 year flood peak estimate of approximately 103 m^3/s, and a 1:20 year flood flow of 76 m^3/s. As expected, these values are again lower



FREQUENCY ANALYSIS - Kilbride
THREE PARAMETER LOGNORMAL-MAX LIKELIHOOD
10^3
10^2
10
DISCHARGE (m³/s)
1.003
1.05
1.25
2.0
5.0
10
20
50
100
500
RECURRENCE INTERVAL IN YEARS

FIGURE 5-14

than those developed by frequency analysis using 28 years of modelled data. Again, this is expected because the regional approach is based on streamflow records from largely undeveloped watersheds. Significant urban development within the Waterford basin with elimination of natural storage depressions, removal of vegetation and soils through lot grading, etc. is assumed to be the cause of the slightly higher flow estimates developed in Section 5.7.1.

Overall, frequency analyses of the 28 years of modelled flood flows give values which are slightly higher than are projected by observed data and the regional approach. This result is anticipated and logical in view of the nature of watershed development, and it confirms that the 28 year frequency analysis presents reliable and conservative results.

All of the above-described estimates of flood flows are based on streamflow derived from historical rainfall and snowmelt conditions.

5.8 Design Rainfall Simulation

Extreme rainfall events (i.e. the 1:20 and 1:100 year storms) will also cause high flows in the river - possibly higher than those discussed above. Development of runoff from these rainfall storms is described below.

5.8.1 Design Rainfall Distributions - St. John's

The Canadian Climate Centre (CCC) of Environment Canada's Atmospheric Environment Service has done considerable research into design rainfall distributions (e.g. Hogg, 1980, 1982; Watt et al., 1986). Several discussions were held with W.D. Hogg of the CCC to determine the most appropriate distributions for the Waterford River Basin - including short-term, convective shower events (thunder-storms) and longer duration, synoptic scale cyclonic events.

It was concluded that the most appropriate distribution for short duration storms be based on the recent work by Watt et al (1986). This approach employs a two parameter model giving a linear rise to the peak intensity of the storm followed by exponential decay in the rainfall. The distribution of rainfall for long duration storms (i.e. 12-hour events) is based on work by Hogg (1980), which initially examined 1-hr and 12-hr rainfall characteristics, and which subsequently examined regional differences in these characteristics (Hogg, 1982). As part of the latter work, Hogg prepared data sheets representing the results of analysis of 12 hour storms at selected locations across Canada (1982). St. John's was one of the selected stations.

Table 5-10 summarizes the distribution of total design rainfall determined for the St. John's area for 1-hr and 12-hr storm events. It is considered appropriate (Hogg, p.c. 1987) to employ the 1-hour distribution for design storm duration up to approximately 4 hours, and the 12 hour distribution for longer durations.

Previous urban hydrology studies by Environment Canada and Newfoundland Environment in the Waterford River Basin conclude that the longer duration storms (i.e., 12 hour durations) produced the maximum flood flow response. Precipitation within the peak hour(s) of such storms may also be distributed to give a peak within the peak hour (Hogg p.c., 1987). This slightly modifies the 12-hour distribution, as shown in Figure 5-15, to yield a distribution given in Table 5-11 (100-year, 12-hour storm).

Initial conditions in the watershed must reasonably reflect average conditions which would likely be present just prior to the advent of such storms. In the watershed modelling report of the previous Urban Hydrology Study, the authors examined antecedent moisture conditions (AMC) before several annual peak flow events and concluded that a very wet condition was a reasonable choice for peak flow estimates (AMC III). They also selected a mean of base flow observed in the river before such storms.

Herein, we also examined conditions prior to high flow events, drawing on sixty-three major storms from 1959 to 1986, but focussing on conditions just

TABLE 5-10

WATERFORD RIVER BASIN

DESIGN RAINFALL DISTRIBUTION

1-Hour Storm (1)

Time (mins)	Percent of Total Rainfall
00-05	2.4
05-10	7.3
10-15	12.2
15-20	17.1
20-25	21.9
25-30	22.3*
30-35	10.0
35-40	4.1
40-45	1.6
45-50	0.7
50-55	0.3
55-60	0.1
1 hour	100 %

* peak at 27th minute
  25-27 min 10.1%
  27-30 min 12.2%

12-Hour Storm (2)

Time (hr)	Percent of Total Rainfall
00-01	1.0
01-02	1.0
02-03	5.0
03-04	11.0
04-05	20.0
05-06	25.0
06-07	18.0
07-08	10.0
08-09	5.0
09-10	2.0
10-11	1.0
11-12	1.0
12 hour	100%

RAINFALL TOTALS(3)

1 hr	20 Year	26.50 mm
1 hr	100 Year	32.69 mm
12 hr	20 Year	80.00 mm*
12 hr	100 Year	97.94 mm*

---

(1) ref. Watt et al., 1986

(2) ref. Hogg, 1982b

(3) ref. AES, 1987

* rainfall totals employed in Table 5-11 and Table 5-12

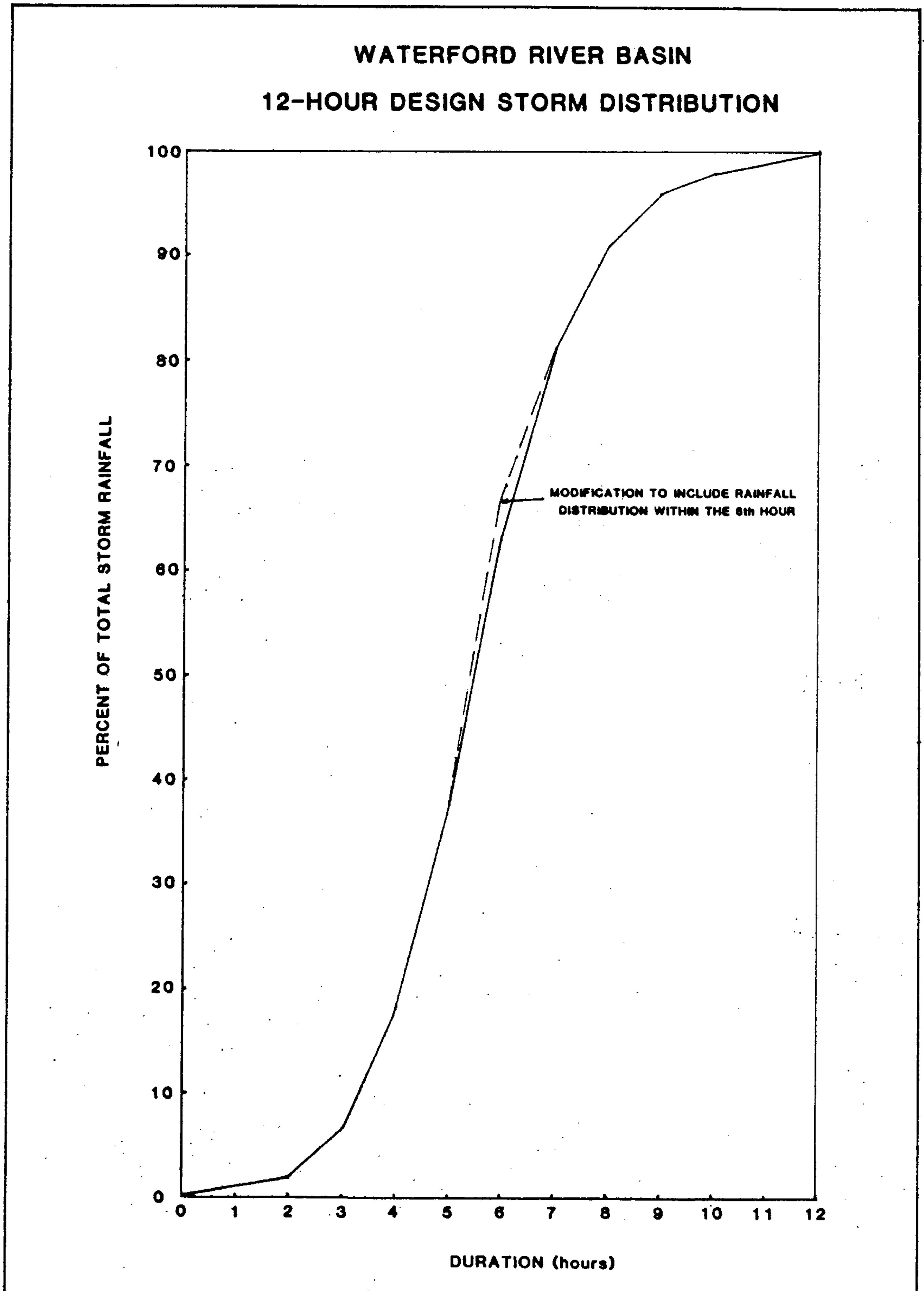

WATERFORD RIVER BASIN
12-HOUR DESIGN STORM DISTRIBUTION
PERCENT OF TOTAL STORM RAINFALL
100
90
80
70
60
50
40
30
20
10
0
0
1
2
3
4
5
6
7
8
9
10
11
12
DURATION (hours)
MODIFICATION TO INCLUDE RAINFALL DISTRIBUTION WITHIN THE 6th HOUR

FIGURE 5-15

TABLE 5-11

DISTRIBUTION OF 100-YR, 12-HR

DESIGN STORM RAINFALL (97.94 mm)

WATERFORD RIVER BASIN

Time (mins)	Rainfall/Time Interval (mm)	Time (mins)	Rainfall/Time Interval (mm)
000-005	0.0816	355-360	1.65
005-010	0.0816	360-365	1.55
-	-	365-370	1.40
055-060	0.0816	370-375	1.35
060-065	0.0816	375-380	1.30
-	-	380-385	1.20
110-115	0.0816	385-390	1.15
115-120	0.0820	390-395	1.10
120-125	0.4079	395-400	1.05
125-130	0.4081	400-405	1.00
130-135	0.4081	405-410	0.95
-	-	410-415	0.90
175-180	0.4081	415-420	0.85
180-185	0.8976	420-425	0.8162
185-190	0.8978	425-530	0.8162
190-195	0.8978	-	-
-	-	470-475	0.8162
230-235	0.8978	475-480	0.8158
235-240	0.8978	480-485	0.4079
240-245	1.6323	485-490	0.4081
245-250	1.6323	490-495	0.4081
-	-	-	-
290-295	1.6323	535-540	0.4081
295-300	1.6327	540-545	0.1636
300-305	1.8000	545-550	0.1632
305-310	2.13	-	-
310-315	2.42	590-595	0.1632
315-320	2.72	595-600	0.1632
320-325	3.00	600-605	0.0820
325-330	3.30	605-610	0.0816
330-335	2.942	-	-
335-340	2.40	655-660	0.0816
340-345	2.20	660-665	0.0816
345-350	1.95	-	-
350-355	1.80	715-720	0.0816

TABLE 5-12

DISTRIBUTION OF 20-YR, 12-HR

DESIGN STORM RAINFALL (80.00 mm)

Time (mins)	Rainfall/Time Intervall (mm)	Time (mins)	Rainfall/Time Interval (mm)
000-005	0.066	360-365	1.266
005-010	0.066	365-370	1.145
-	-	370-375	1.103
055-060	0.066	375-380	1.062
060-065	0.066	380-385	0.980
-	-	385-390	0.939
110-115	0.074	390-395	0.899
115-120	0.074	395-400	0.858
120-125	0.333	400-405	0.817
125-130	0.333	405-410	0.776
130-135	0.333	410-415	0.735
-	-	415-420	0.694
175-180	0.337	420-425	0.674
180-185	0.733	425-430	0.666
185-190	0.733	-	-
190-195	0.733	470-475	0.666
-	-	475-480	0.666
230-235	0.733	480-485	0.337
235-240	0.737	485-490	0.333
240-245	1.333	490-495	0.333
245-250	1.333	-	-
-	-	535-540	0.333
290-295	1.333	540-545	0.137
295-300	1.337	545-550	0.133
300-305	1.470	550-555	0.133
305-310	1.740	-	-
310-315	1.977	590-595	0.133
315-320	2.222	595-600	0.133
320-325	2.450	600-605	0.074
325-330	2.696	605-610	0.066
330-335	2.403	-	-
335-340	1.960	655-660	0.066
340-345	1.797	660-665	0.066
345-350	1.593	-	-
350-355	1.470	715-720	0.066
355-360	1.348		

prior to 28 annual events. It was determined that average conditions in the watershed before the annual peak flow events were wet, but not extremely wet. The API value averaged 50 mm, which lies between the AMC II (35.8 mm) and AMC III (61.6 mm) condition. In view of this finding, it was decided to determine runoff employing the average API value of 50 mm, but also evaluate the sensitivity of flow projections using 61.6 mm (the AMC III condition).

5.8.2 Design Rainfall Simulations

Table 5-13 summarizes the results of the design rainfall simulations. The first two columns show the results of this current study and the third column gives the results of earlier HYMO modelling.

It is initially apparent that the current study gives higher runoff than the earlier study. This may be due to differences in the models (QUALHYMO is better suited for this work), the distribution of the design rainfall or differences in imperviousness between land use employed in this study (i.e., 1984 land use vs. 1981 use employed earlier).

The results of the present QUALHYMO study show that streamflow is somewhat sensitive to the antecedent moisture condition of the watershed. Flood flows with an API of 61.6 mm (AMC III) are about 12% higher than those with the average, pre-storm API condition (50 mm). However, this 12% increase in flow results from a 23% increase in the API.

5.9 Selection of Design Flows

Table 5-14 summarizes the flood flow estimates derived from design storm rainfall and the frequency analyses of simulated streamflows (discussed in Section 5.7.1).

The flood flows using streamflow frequency analyses are about 20% higher than the design rainfall runoff. This result shows that snowmelt, snowmelt with rainfall, and rainfall all join to play a role in defining flood peaks

TABLE 5-13

SUMMARY OF PEAK FLOWS FOR DESIGN RAINFALL STORMS*

1:100 YEAR STORM

WSC Gauge Location	Present QYALHYMO Study (m^3/s)		Previous HYMO Model Study (m^3/s)
	API = 50	API = 61.6	AMC III
Donovans	19.14	21.4	14.7
Mount Pearl	30.36	33.8	25.7
Kilbride	95.07	105.4	81.5

1:20 YEAR STORM

Donovans	14.1	-	11.6
Mount Pearl	21.8	-	19.9
Kilbride	67.9	-	64.6

* Values include baseflow of 0.36 m^3/s, 0.67 m^3/s and 2.57 m^3/s at Donovans, Mount Pearl and Kilbride, respectively

\- not simulated

TABLE 5-14

SUMMARY OF FLOOD FLOW ESTIMATES
DESIGN STORMS AND FREQUENCY ANALYSES

1:100 YEAR RETURN PERIOD (m^3/s)

Location	Design Rainfall	3 PLN* Distrib.
Mount Pearl	30.36	36.20
Kilbride	95.07	118.00

1:20 YEAR RETURN PERIOD (m^3/s)

Location	Design Rainfall	3 PLN* Distrib.
Mount Pearl	21.8	26.20
Kilbride	67.9	83.30

* selected distribution - the Three Parameter Log Normal Distribution

on Waterford River Basin. The design rainfall events provide reasonably good estimates of flood flows but these estimates are slightly lower than projections using a long sequence (28 years) of river flows.

It was noted earlier that the Three-Parameter Log Normal distribution provides the best fit to the data, matches regional statistics, and is also a distribution which is consistent with that selected by the Canada-Newfoundland Flood Damage Reduction Program for general flood studies in Newfoundland. It also provides flood peaks which are higher than design rainfall events and, hence, has been selected for determining backwater conditions along the Waterford River.

Figure 5-14 gave the frequency analysis of design flood flows at one location on the river - Kilbride. Table 5-15 summarizes the 1:20 year and 1:100 year design flood flows derived from the Three Parameter Log Normal Distribution at 18 other points along the river. These values are used in the following section to derive 1:20 year and 1:100 year flow profiles.

TABLE 5-15

SUMMARY OF 1:20 AND 1:100 YEAR DESIGN FLOWS
AT VARIOUS POINTS ALONG THE WATERFORD RIVER

Hydrologic Model Flow Points	Design Peak Flows (m^3/s) 1:20 yr	1:100 yr
100 (west Tributary at Donovans)	6.13	9.13
102 (south Tributarty at Donovans)	5.53	6.43
103 (Combined Trib's at Donovans)	15.1	15.6
104 (Donovan Flow Gauge)	16.9	22.8
107 (Mt Pearl Flow Gauge)	26.2	36.2
109	29.2	40.3
110	32.2	44.2
112	38.3	52.4
114	44.9	61.4
116 (U/S of Waterford R. & South Brook Confluence)	52.2	71.1
123 (D/S of Waterford R. & South Brook Confluence)	82.7	117.0
124 (Kilbride Flow Station)	83.3	118.0
	85.0	121.0
129	93.1	133.0
131	94.5	135.0
133	97.4	139.0
137 (Outlet of Waterford River)	102.0	145.0

* Cross section numbering in metres from the river mouth

6.0 HYDRAULIC ANALYSIS

6.1 General

The purpose of the hydraulic investigation is to derive the 1:20 and 1:100 year open water surface profiles along the study reaches using the results of the hydrologic information provided in Chapter 5.0.

To carry out the investigations, the HEC-2 (HEC, 1982) computer model was used. This model was selected for this study because it represents the state-of-the-art for the computation of water surface profiles for steady state conditions in open channels. It has been successfully used in similar applications in the U.S. and Canada, is well-documented, is parameter efficient for calibration, and is flexible in use. The HEC-2 Model was also selected since it has been applied to three (3) short reaches along the Waterford River in a previous study (Flood Study of the Urban Hydrology Study of the Waterford River Basin, 1986).

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

Full details of the HEC-2 model and its underlying theory are given in the user's manual (HEC, 1982). The release used for this study was issued November 1976 updated May 1984.

6.2 Previous Hydraulic Analysis

As part of the Urban Hydrology Study of the Waterford River Basin (1986), a hydraulic investigation of three short reaches along the Waterford River was undertaken jointly by Environment Canada and Newfoundland Department of Environment. The three reaches studied were:

(i) Kilbride: This reach extends from the Water Survey of Canada hydrometric gauge (02ZM008) just below the bridge at Kilbride to the ruins of an old control structure for the Bowring Park pond. The length of the reach is about 335 metres and includes 20 HEC-2 cross sections.

(ii) Mount Pearl: This reach extends from a foot bridge (Steady Waters Bridge) located at the end of Forest Avenue to upstream of Winston Ave., downstream of the Water Survey of Canada hydrometric gauge (02ZM010). The length of the reach is about 1130 metres and includes 14 HEC-2 cross sections.

(iii) Donovans: This reach extends from the Water Survey of Canada hydrometric gauge site (02ZM011) on the abandoned bridge abutment to downstream of the merging of the two tributaries above the Newfoundland Hardwoods Limited buildings. Two bridges are located within this reach. The length of the reach is about 600 metres and includes 22 HEC-2 cross-sections.

The HEC-2 model for each reach was calibrated/validated based on events monitored during the 1981-1983 period. Each model was subsequently used to compute the 1:20 and 1:100 year water surface profiles for floodplain mapping purposes. Further details on the model set-up, calibration/validation and computation of design water levels are provided in the earlier Flood Study report (1986).

Five of these sections were re-surveyed as part of this study. No significant changes were found in the channel geometry since 1981-83, and the above 56 sections were included in the subsequent development of the HEC-2 model.

6.3 Set-Up of the HEC-2 Model

The HEC-2 model for the Waterford River was set-up using river cross-sectional data obtained from field surveys (undertaken during the summer and fall of 1987) and topographic data from other sources. The surveys included measurements of representative channel cross sections and structures across the watercourse, and photographs at each section.

A total of 133 representative river valley and structure cross-sections were surveyed along the Waterford River from the CN Dockyard to the Clyde Corner culverts in the Donovan area. The total distance surveyed along the river was about 13.7 km (excluding most of about 2.1 km previously surveyed for the Kilbride, Mount Pearl and Donovan reaches). The surveyed cross sections represent an average of about 10 cross-sections per km.

All the details of some of the structures were not surveyed since construction drawings were available. However, the major dimensions were surveyed at these structures and the top of road and/or low chord elevations were tied into geodetic datum. The structures which had drawings available were:

- Symes Bridge
- Waterford Lane Bridge
- Kilbride Bridge
- Brookfield Bridge
- Outerbridge Street/Park Avenue Bridge
- Commonwealth Avenue Bridge
- Newfoundland Fibreply Bridge
- Newfoundland Hardwoods Bridge
- TransCanada Highway Bridge at Donovans

The cross-sections were surveyed above and below the waterline and in most cases beyond the top of bank. Each cross-section was referenced to geodetic datum.

The field cross section information at most stations was supplemented with 1:2500 scale topographic mapping (with 0.5m contour intervals). The supplementary data was used to extend the measured valley sections to elevations near the outer limits of the flood plain mapping sheets. This measure proved largely unnecessary, however, as flood water levels (determined later in the study) do not reach the mapping limits. Several sections were coded entirely from the 1:2500 scale mapping. These are the western-most sections on the western tributary at the Trans-Canada Highway. Another was coded from bathymetric survey data obtained by others for CNR developments at the harbour mouth. Ten other sections were coded using channel cross sections from the field surveys and overbank data from the 1:2500 scale mapping.

The structures across the watercourse were coded using the guidelines outlined in the HEC-2 Users Manual. For example, the skewness of the structures was taken into account in the coding where the structure is not perpendicular to the flow.

A total of 209 cross sections were used to model the Waterford River. Ninety one are bridge-related sections, four are sections from the mapping near the Trans-Canada Highway, one is within St. John's Harbour, and 113 are river sections between the Harbour and the upstream study limit. Locations of some of these sections are shown on the flood profile mapping examples given at the end of Chapter 7 (Figures 7-1 to 7-7).

The cross section numbers generally represent the total distance in metres from the upstream face of the CN Dockyard/synchrolift. However, the HEC-2 model begins about 50 metres downstream of that structure to estimate the hydraulic effect of the Dockyard with its many piers.

All the cross-sections were coded adopting the convention of looking downstream from left to right bank, and all the sections are referenced to Geodetic Survey of Canada (GSCD) datum.

Plots of all cross-sections and bridge/culvert structures are presented in the Technical Appendix. The Manning's roughness coefficients for the left and right overbanks and channel are shown in the above-noted plots and are listed for each section in Appendix 4.0. Initially the roughness coefficients were estimated from photographs taken of each section during the field work and from available mapping. The values were estimated by the use of the Cowan equation employed in the previous flood study (for the Kilbride, Mount Pearl and Donovan reaches). Subsequently some of the values were adjusted during the calibration of the model.

6.4 Calibration of the HEC-2 Model

6.4.1 General

In order to obtain confidence in the computed water surface profiles, the HEC-2 model was calibrated and validated with observed water levels and flow data. For the Waterford River, this information was obtained from:

(i) Selected events which were monitored for the Urban Hydrology Study (UHS) during the 1981-1983 period for the Kilbride, Mount Pearl and Donovan reaches. Tabulation of observed water levels and flows are presented in the Flood Study report of the UHS (1986).

(ii) Water level monitoring program undertaken for this study during the 26 March - 13 November 1987 period. This monitoring program is described in Section 6.4.2.

(iii) Supplementary hydrology data (hydrographs) from the hydrology study (Chapter 5).

6.4.2 Water Level Monitoring Program

Five crest gauges were initially installed along the Waterford River to obtain water levels suitable for calibration purposes. The crest gauges were installed on the upstream side of the following bridge abutments:

- Symes Bridge (cross-section 1575)
- Brookfield Road Bridge (cross-section 5425)
- Birch Avenue Tracks (cross-section 2010)
- Newfoundland Fibreply Limited Bridge (cross-section 3007.1)
- Newfoundland Hardwoods Limited Bridge (cross-section 3014.8)

The observed levels from this program are provided in the Technical Appendix with other measurement data.

The crest gauges were referenced to geodetic datum corresponding to the bottom of each gauge. Unfortunately, the gauge at Birch Ave. was repeatedly damaged by vandals and it proved necessary to manually measure the water level at this location. A reference point was marked at the top of the abutment, and the field crew measured from this point to the water surface. This approach was eventually abandoned as it proved impossible for the field crew to time their arrival at the time of the peak water level.

A major high flow event did not occur during the 26 March - 13 November 1987 monitoring period. From hourly flow data obtained from Water Survey of Canada, the peak flow of 31.1 m^3/s occurred at the Kilbride hydrometric station on 3 April 1987. There were other smaller flow events but the peaks were generally less than 20 m^3/s at the Kilbride station.

6.4.3 Model Calibration

The event which occurred on 3 April 1987 was selected to calibrate the model, because it represents the most recent hydraulic condition in the river and the observed water levels are for the full length of the river.

The above noted event ws generated primarily by snowmelt, and was almost identical to the smowmelt event that occurred on 27 January 1978. The observed and simulated peak flows for both events are presented below:

Hydrometric Station	Observed (Simulated) Peak Flow (m^3/s)		
	27 January 1978		3 April 1987
Mount Pearl	-	(10.3)*	10.3
Kilbride	30.9	(32.0)**	31.1

* Simulated with the QUALHYMO Model. The Mount Pearl Station was not active in 1978.

** Simulated with the QUALHYMO Model.

Of importance in this table is that the observed flows at Kilbride are practically identical, and that the simulations (which give peak flows at a variety of locations) are also similar to the observed flows. Hence for practical purposes to fill the need for flows at many unmonitored locations, the January 1978 event (simulated with QUALHYMO) was used to distribute the flows to unmonitored locations in the HEC-2 Model.

Several computer runs were made to calibrate the HEC-2model. After each run adjustments were made to the Manning's roughness values until good agreement was obtained between the simulated and observed water levels. The results are presented in Table 6-1. Also presented in Table 6-1 are the observed and simulated water levels at the Kilbride hydrometric station, obtained from Water Survey of Canada.

The Manning's roughness values used in the calibration are presented in the cross-section plots in the Technical Appendix and are listed in Appendix 4.0. The values are given for the left and right overbanks and channel.

TABLE 6-1

CALIBRATION OF WATERFORD RIVER

HEC-2 MODEL - 3 APRIL 1987 EVENT

Location	Cross Section Number	Peak Flow (m^3/s)	Water Surface Elevation (m) Observed	Computed
Symes Bridge	1575	37.10	12.52	12.52
Kilbride Gauge	1001*	31.10	32.17	32.11
Brookfield Bridge	5425	19.10	56.11	56.09
Birch Ave. Tracks	2010*	12.15	No data**	103.38
Newfoundland Fibrply Bridge	3007.1*	7.12	134.31	134.33
Newfoundland Hardwoods Bridge	3015*	5.18	135.04	135.05

* Cross section numbering used in previous Waterford River Flood Study

** Peak water level not measured due to damaged crest gauge. However in October 1983, the flow at cross-section 2010 was about 15.2 m^3/s and the observed water level 103.54 m. The simulated water level in April 1987 appears in close agreement for a flow of 12.15 m^3/s.

6.5 Validation of the HEC-2 Model

After the HEC-2 model was calibrated, it was validated to ensure it was predicting reasonably accurate results. The observed water levels from the previous flood study (1986) were used to validate the model. Two events were selected for validation of the Kilbride and Mount Pearl reaches, and one event was used for the verification of the Donovan reach.

6.5.1 Kilbride Reach

This reach corresponds to cross sections 1001 to 1008 in the previous flood study (1986). The only change made to the original HEC-2 model was in the area of cross section 1007 where the cross section was slightly altered due to the construction of the Malloys Lane Arterial Road over the river. However, the bridge is high and wide and does not obstruct the flow of water underneath.

The two events selected for validation were the 26 November 1981 and the 26 October 1983 events. The HEC-2 models were run from St. John's Harbour to upstream of cross section 1008. The flows were distributed to appropriate HEC-2 cross section using the flows simulated with the QUALHYMO Model as part of the hydrologic analysis.

The starting water levels were obtained from the November 1981 and October 1983 monthly tide levels observed at St. John's Harbour. However, results are insensitive to the starting water level since critical flow conditions occur a short distance upstream of the CN Dockyard.

The results of the validation for the 26 November 1981 and 26 October 1983 events are presented in Table 6-2 and Table 6-3, respectively.

TABLE 6-2

VERIFICATION OF WATERFORD RIVER HEC-2 MODEL
FOR KILBRIDE REACH

26 November 1981 Event

Cross Section Number***	Flow (m^3/s)	Water Surface Elevation (m)	
		Observed	Computed
1001	47.2	32.40	32.42
1001.5	47.2		32.51
1002.1	47.2		32.42
1002.9	47.2		32.45
1003	47.2	32.53	32.61
1004	47.2	32.49	32.61
1005	47.2	32.84**	32.64
1005.7	47.2		32.87
1005.8	47.2		33.12
1005.9	47.2		33.39
1006	47.2	33.30	33.40
1006.1	47.2		33.44
1006.2	47.2		33.39
1006.3	47.2		33.39
1006.4	47.2		33.39
1006.5	47.2	33.27	33.39
1006.7	47.2		33.40
1006.8	47.2		33.40
4008*	47.2	33.79	33.84
1008	47.2	33.98	34.02

* Approximate location as cross section 1007 in previous study (1986)

** Previous study identified possible datum error at this section

*** Cross section numbering used in previous study (see Table 7-2)

TABLE 6-3

VERIFICATION OF WATERFORD HEC-2 RIVER MODEL
FOR KILBRIDE REACH

26 October 1983 Event

Cross Section Number****	Flow (m^3/s)	Water Surface Elevation (m)	
		Observed	Computed
1001	39.30	32.47***	32.35
1001.5	39.30	32.35	32.41
1002.1	39.30		32.35
1002.9	39.30		32.37
1003	39.30	32.42	32.49
1004	39.30		32.49
1005	39.30	32.52**	32.51
1005.7	39.30	32.94	32.77
1005.8	39.30		33.05
1005.9	39.30		33.26
1006	39.30	33.36	33.26
1006.1	39.30		33.32
1006.2	39.30		33.28
1006.3	39.30		33.28
1006.4	39.30		33.28
1006.5	39.30	33.22	33.28
1006.7	39.30		33.29
1006.8	39.30		33.29
4008*	39.30	33.69	33.65
1008	39.30	33.80	33.82

* Approximate location as cross section 1007 in previous study (1986

** Previous study identified possible datum error at this section

*** Water level appears high for observed flow. In 26 November 1981, the water level was lower (32.40 m) for a higher flow (47.2 m^3/s)

****Cross section numbering used in previous study (see Table 7-2)

Generally there is fairly good agreement between the observed and computed water surface elevations. The verification simulation in Table 6-2 gives computed results which are slightly higher than the observed levels, and that of Table 6-3 gives values which are slightly lower. There appears to be some discrepancy at cross section 1005 but, as indicated in the previous flood study, a datum error is suspected at this section. Another possible reason for the discrepancy may be due to local debris blockage. Some inconsistency in water level measurements may also occur for this reach since water levels were measured from downstream to upstream rather than in the direction of the flow, and recent channel modifications slightly affect the upper and lower ends of the reach.

The selected Manning's roughness coefficients for this reach range from 0.03 to 0.04 for the overbanks and 0.02 to 0.03 for the channel.

6.5.2 Mount Pearl Reach

This reach corresponds to cross section 2001 to 2013 in the previous study. The events selected for verification for this reach were the 26 October 1983 and 20 June 1982 events.

Initially, attempts were made to validate the HEC-2 model using the calibrated model and the representative flows presented in the previous study (12.0 m^3/s for the 26 October 1983 event and 8.67 m^3/s for the 20 June 1982 event). However, using these flows and realistic roughness values did not produce good agreement with the measured water levels.

Subsequently, the hydrometric record of the 26 October 1983 event was subject to close investigation. From hourly flow data provided by Water Survey of Canada at the Mount Pearl gauge, it was found that the staked profile was measured during the recession limb of the hydrograph. At time 16:16 when the measurements were started at cross section 2013 (the upper section), the flow at the Mount Pearl gauge was about 13.3 m^3/s. Since the water levels were measured during the recession, the flows along the staked reach were in fact higher than at the upstream gauge at Mount Pearl. By taking into account the travel time between the gauge and section 2013, it was estimated that the flow at section 2013 was about 13.8 m^3/s. Next the local inflow from the QUALHYMO model simulation was added to the above flow to obtain representative flows for the study reach. The flow at cross section 2001 was estimated to be 15.2 m^3/s. Above cross section 2011, the flow was reduced to 14.3 m^3/s because a small tributary empties into the main river downstream of this section.

The HEC-2 model was subsequently run with the revised flows starting downstream of cross section 2001, and the results for the 26 October 1983 event are presented in Table 6-4. Except for cross sections 2008 and 2009,

TABLE 6-4

VERIFICATION OF WATERFORD RIVER HEC-2 MODEL
FOR MOUNT PEARL REACH

26 October 1983 Event

Cross Section Number****	Flow (m^3/s)	Water Surface Elevation (m) Observed	Computed
2001	15.2	102.18	102.20
2002	15.2		102.25
2002.5	15.2		102.28
2003	15.2	102.30	102.29
2004	15.2	102.34	102.32
2005	15.2		102.40
2006	15.2	102.53	102.49
2007	15.2	102.68	102.64
2008***	15.2	103.00*	102.73
2009***	15.2	103.63	103.01
2010	15.2	103.54**	103.44
2011	14.3	103.75	103.67
2012	14.3	104.07	104.24
2013	14.3	104.39	104.41

* Measurement error suspected

** Measurement reported not to be at this section (Flood Study, 1986)

*** Manning's roughness values used were: 0.12 for left bank, 0.09 for right bank and 0.08 for channel only at these sections

**** Cross section numbering used in previous study (see Table 7-2) there is generally good agreement between the observed and simulated water levels. As indicated in the previous flood study (1986), a measurement error is suspected at section 2008, and the water level measurement reported at 2010 is from another section.

To improve the results at sections 2008 and 2009, additional cross sections were added downstream. However the additional sections did not improve the results and they were subsequently discarded.

The Manning's roughness coefficients for this reach ranged typically from 0.035 to 0.09 for the overbanks and 0.025 to 0.08 for the channel. Only at cross sections 2008 and 2009 were the coefficients for the left overbank increased to 0.12.

Similar considerations as described for the 26 October 1983 event were undertaken to revise (increase) the flow for the 20 June 1982 event. The flows were estimated to be 10.2 m^3/s at cross section 2001 and reduced to 9.3 m^3/s upstream of cross section 2011.

The results are summarized in Table 6-5. As can be seen, there is fairly good agreement for the whole profile except at cross section 2009.

6.5.3 Donovans Reach

This reach corresponds to cross sections 3001 to 3017 in the previous study. Two changes were made to this part of the previous model. One was near cross section 3001 (site of the abandoned Donovan hydrometric gauge) where a section was added to take into account the existing abutments from the demolished bridge. The other was at the Newfoundland Hardwoods Limited bridge (cross section 3014.8) to enable computer modelling of the stoplog structure upstream (rather than manual calculation).

TABLE 6-5

VERIFICATION OF WATERFORD RIVER HEC-2 MODEL
FOR MOUNT PEARL REACH

20 June 1982 Event

Cross Section Number***	Flow (m^3/s)	Water Surface Elevation (m)	
		Observed	Computed
2001	10.2	101.91	102.01
2002	10.2	101.96	102.05
2002.5	10.2		102.08
2003	10.2	102.11	102.09
2004	10.2	102.11*	102.12
2005	10.2		102.22
2006	10.2	102.33	102.32
2007	10.2	102.57	102.48
2008**	10.2		102.57
2009**	10.2	103.10	102.88
2010	10.2		103.33
2011	9.3	103.56	103.56
2012	9.3	103.98	104.07
2013	9.3	104.21	104.32

* 15m downstream of section

** Manning roughness values same as presented in Table 6-4

*** Cross section numbering used in previous study (see Table 7-2)

Initially, the 4 October 1982 and the 26 October 1983 high flow events were selected for verification purposes. However, these two events have similar flows and the 26 October 1983 event was ultimately selected (since it has more water level measurements than the 1982 event).

Attempts were initially made to verify the HEC-2 model using the calibrated model and the flow estimate of 6.72 m^3/s reported in the previous study. However, the computed and measured water levels were not in good agreement and it was again suspected that the streamflow during profiling might not be truly representative of actual conditions.

Hourly flow data for the above event was not available at the Donovan station to complete this analysis. However, the simulated flow from the QUALHYMO model for this event was slightly higher than the observed and was entered into the model. A flow of 9.0 m^3/s was used from cross section 3001 to 3010 and then it was reduced to 7.7 m^3/s to account for reduced drainage area above section 3010.

The results of the verification for the Donovan reach are presented in Table 6-6. As can be seen there is fairly good agreement between the observed and computed water levels except above the Newfoundland Hardwoods Limited bridge (sections 3015 to 3017). It is suspected that the accuracy of water level monitoring in this area is in the range of plus or minus 0.15 m, perhaps because of debris blockage at the bridge.

The Manning's roughness coefficients ranged from 0.08 to 0.09 for the overbanks and 0.025 to 0.085 for the channel.

6.5.4 Calibration/Verification Conclusions

From the results presented in Tables 6-1 to 6-6, it can be seen that the observed and computed water levels are in reasonably close agreement. Consequently, it is concluded that the model is capable of accurately predicting open water surface profiles for flood flow events.

TABLE 6-6

VERIFICATION OF WATERFORD RIVER HEC-2 MODEL
FOR DONOVAN REACH

26 October 1983

Cross Section Number**	Flow (m^3/s)	Water Surface Elevation (m)	
		Observed	Computed
3001	9.0	133.67	133.67
3001.9	9.0		133.61
3002	9.0		133.76
3003	9.0	133.77*	133.79
3004	9.0	134.03*	134.28
3005	9.0		134.41
3006	9.0	134.36	134.40
3007.1	9.0		134.40
3007.9	9.0		134.40
3008	9.0	134.46	134.39
3009	9.0		134.45
3010	9.0	134.49*	134.44
3011	7.7	134.53*	134.53
3012	7.7	134.64	134.57
3013	7.7	134.64	134.62
3014.1	7.7		134.61
3014.2	7.7		134.51
3014.8	7.7		135.07
3014.9	7.7		135.15
3015	7.7	135.39* (135.12)	135.06
3016	7.7	135.40	135.27
3017	7.7	135.48* (135.47)	135.28

* Water levels at these sections were measured as follows:
Section 3003, 2m upstream; section 3004, 21m downstream; section 3010, 3m upstream; section 3011, 14m downstream; section 3015, 4m upstream; section 3017, 2m upstream

( ) interpolated level at actual section

** Cross section numbering used in previous study (see Table 7-2)

6.6 Sensitivity Testing for the HEC-2 Model

6.6.1 General

In order to test the sensitivity of the computed water levels to variations in parameters within the model, a number of computer runs were made by varying one parameter while holding the others at their previously determined values. The parameters changed in the modelling were: channel invert by ± 0.15 m; discharge by ± 30%; starting water level by ± 0.15m; Manning's 'n' values by ± 10%; and expansion and contraction coefficients by ± 10%. The above variations in parameters were used to be consistent with those used in the previous flood study (1986).

As indicated in Section 6.3, the HEC-2 model for the Waterford River consists of 210 cross sections. In order to reduce computer costs, it was decided to conduct sensitivity testing on a representative part of the Waterford River. The reach selected extends from cross section 6450 below Dunn's Avenue Bridge to downstream of Confederation Avenue bridge (cross section 9415) and includes the Mount Pearl reach modelled in the previous study. This reach was selected because part of the river is steep and part of it has moderate slopes. The Mount Pearl reach is relatively flat and the reach downstream is relatively steep.

The 1:100 year flow was used for sensitivity testing and the results are presented in Table 6-7.

6.6.2 Sensitivity to Changes in Channel Invert

The channel invert between cross section 6450 and 9415 was changed by ± 0.15 metre. Table 6-7 shows that the change in the water surface profile is more pronounced for the steeper reach than the flatter one. For the steeper reach (cross sections 6955 to 7193) the change in water level is between 0.07 to 0.11 metres whereas for the flatter reach (cross section 2001 to 2013) the change is only 0.0 to 0.02 metres.

TABLE 6-7

HEC-2 SENSITIVITY ANALYSIS FOR WATERFORD RIVER

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

	Slope	Moderate			steep					moderate		
Parameters	Parameter Variation	6955	6970	7193	7363	7697	2001*	2004*	2008*	2010*	2013*	9415
Bottom of Channel	+0.15m	+0.08	+0.08	+0.09	+0.01	+0.03	+0.02	+0.02	+0.01	0.0	0.0	+0.02
	-0.15m	-0.07	-0.07	-0.11	-0.02	+0.01	-0.01	-0.01	-0.01	-0.01	0.0	0.0
Discharge	+30%	+0.19	+0.45	+0.32	+0.11	+0.12	+0.14	+0.17	+0.18	+0.13	+0.07	+0.11
	-30%	-0.24	-0.21	-0.28	-0.23	-0.10	-0.20	-0.24	-0.22	-0.13	-0.10	-0.13
Starting Water Level**	+0.15m	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	-0.15m	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Manning's "n"	+10%	+0.05	+0.08	-0.02	+0.11	0.0	+0.06	+0.05	+0.06	+0.04	+0.01	+0.06
	-10%	-0.06	-0.08	-0.02	-0.13	0.0	-0.07	-0.07	-0.06	-0.05	-0.02	0.0
Expansion and Contraction Coefficients	+10%	0.0	+0.02	-0.02	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	-10%	-0.01	-0.03	-0.02	-0.01	0.0	0.0	0.0	0.0	0.0	0.0	0.0

* cross section numbering used in previous study (see Table 7-2)
** critical depth occurs upstream of cross section 6450

6.6.3 Sensitivity to Peak Discharge

Variations of ± 30% in peak discharge altered the water levels from 0.07 to 0.45 m. The change in water level is affected primarily by the configuration at each cross section. Typically cross sections with steep sides will produce greater change in water levels whereas cross sections with flatter sides will produce smaller change in water levels.

For the Waterford River, the sensitivity analysis indicates that the water surface profile is generally not very sensitive to large variations in discharges.

6.6.4 Sensitivity to Starting Water Level

The Waterford River has some fairly steep reaches where critical flow conditions occur. Therefore starting water level conditions have only a localized effect on water levels. In the sensitivity testing undertaken, the 1:100 year water level at cross section 6450 was varied by ± 0.15 metre. However, critical depth occurred shortly upstream and therefore the water levels upstream were not affected by the starting water levels.

Noteworthy is that critical depth occurs at about 45 locations from the confluence at Donovan to the outlet, and hence, it is felt that water levels are generally insensitive to starting elevations throughout the River. Although subcritical flow may occur at these locations, the use of critical depth values appropriately covers the effect of debris within the streamflow and other naturally occurring small perturbations in water levels.

6.6.5 Sensitivity to Roughness Coefficient

As can be seen from Table 6-7, variation in the Manning's 'n' values by ± 10% results in water levels change of less than 0.13 metres. Therefore, the water surface profile is not significantly affected by the modeller's judgement in selecting the Manning's 'n'.

6.6.6 Sensitivity to Expansion and Contraction Coefficients

Variation in the expansion and contraction coefficients by ± 10% has localized effect on the water levels by generally less than 0.03 metres. The results indicate that the model is not sensitive to errors in estimating the expansion and contraction coefficients.

Overall, the modelled results are insenstive to all but relatively large changes in streamflow at a few, narrow cross sections. In a river-wide sense, however, the model is not very sensitive to variations in all parameter values.

7.0 FLOOD LEVEL PROFILES

As indicated previously, the purpose of this study is to derive open water surface profiles for the 1:20 and 1:100 year return period flood flows.

In Chapter 5.0, 28 years of annual maximum peak flows were generated by the QUALHYMO model at 18 points along the main branch of the Waterford River. At each point, flood frequency analyses were conducted using the CFA88 program and the Three Parameter Lognormal distribution was employed to provide the best estimate of the 1:20 and 1:100 year flows.

A summary of these flows for 1:20 and 1:100 year return period floods is presented in Table 7-1. Also presented are the 18 hydrologic model flow points and the corresponding HEC-2 cross sections. Noteworthy is that the HEC-2 cross-section numbering for each flow point is in metres from the river mouth to provide a consistent way to identify cross section locations. Table 7-2 provides a listing of this numbering system and that used in the previous flood study (1986), to assist those who are examining both studies.

7.1 Starting Water Levels

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

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

TABLE 7-1

SUMMARY OF 1:20 AND 1:100 YEAR DESIGN FLOWS
AT VARIOUS POINTS ALONG THE WATERFORD RIVER

Hydrologic Model Flow Points	HEC-2 * Cross-section	Design Peak Flows (m^3/s) 1:20 yr	1:100 yr
100 (west Tributary at Donovans)	12355	6.13	9.13
102 (south Tributarty at Donovans)	12312	5.53	6.43
103 (Combined Trib's at Donovans)	12302	15.1	15.6
104 (Donovan Flow Gauge)	11758	16.9	22.8
107 (Mt Pearl Flow Gauge)	9870	26.2	36.2
109	7947	29.2	40.3
110	7070	32.2	44.2
112	6970	38.3	52.4
114	6450	44.9	61.4
116 (U/S of Waterford R. & South Brook Confluence)	4070	52.2	71.1
123 (D/S of Waterford R. & South Brook Confluence)	4008	82.7	117.0
124 (Kilbride Flow Station)	3694	83.3	118.0
128	3008	85.0	121.0
129	2206	93.1	133.0
131	1575	94.5	135.0
133	745	97.4	139.0
137 (Outlet of Waterford River)	0	102.0	145.0

* Cross section numbering in metres from the river mouth

TABLE 7-2

REVISED CROSS-SECTION NUMBERING

FOR EARLIER BACKWATER STUDY

Kilbridge Reach

Previous Section No.*	Revised Number**
1,001.0	3,694
1,001.5	3,698
1,002.1	3,702
1,002.9	3,709
1,003.0	3,711
1,004.0	3,722
1,005.0	3,764
1,005.7	3,837
1,005.8	3,883
1,005.9-6.0	3,917
1,006.1	3,933
1,006.2	3,945
1,006.3-.4-.5	3,957
1,006.7-.8	3,960
1,007.0	3,997
1,008.0	4,028

Mount Pearl Reach

Previous Section No.	Revised Number
2,001	7,947
2,002	7,963
2,002.5	8,022
2,003	8,120
2,004	8,232
2,005	8,357
2,006	8,508
2,007	8,606
2,008	8,629
2,009	8,776
2,010	8,842
2,011	8,896
2,012	8,988
2,013	9,048

Donovans Reach

3,001	11,758
3,002	11,767
3,003	11,774
3,004	11,873
3,005	11,918
3,006	11,924
3,007.1	11,930
3,007.9	11,933
3,008	11,938

Donovans Reach (continued)

3,009	11,942
3,010	12,045
3,011	12,124
3,012	12,200
3,013	12,227
3,014	12,232
3,015	12,235
3,016	12,244
3,017	12,302

* previous Waterford River Basin Flood Study (1986)

** current study employing section numbering in metres from river mouth

The instantaneous water levels (no wave effect) in the harbour also provide a second data set for evaluating the 1:20 year and 1:100 year flood elevations near the river mouth. MEDS records indicate that the highest annual levels occur in the winter months when streamflows are low. Frequency analyses of these levels (in the Technical Appendix) provide the following results:

Water Level at Waterford River Mouth	Instantaneous Level (GSCD)
1:20 year	1.54 m
1:100 year	1.69 m

Hence, to show both possibilities of flooding (from high flows on the Waterford River and high levels at the river mouth) the highest of these two must be plotted near the mouth.

7.2 Flood Level Delineation

The complete calibrated/verified HEC-2 model was run with a starting level of 0.62 m and the 1:20 and 1:100 year peak flows. This provided water level data at all cross sections in the model, and a summary printout of the results for both flood flows is presented in Appendix 5.0.

Typical cross section locations in the model are plotted in Figures 7-1 to 7-7, which are photo-reductions of portions of the floodplain mapping. Each section is marked as a line on the map, the circle connecting the line gives the 1:20 and 1:100 year flood water level. The dashed and solid lines connecting the cross sections delineate the 1:100 year and 1:20 year flood hazard areas, respectively. There are many locations along the river where the 1:100 year flood levels are only slightly higher than the 1:20 year levels. In these locations, only the 1:20 year flood line has been plotted.

Complete verions of the flood risk maps (showing flood levels from the harbour to Donovans Industrial Park) may be obtained from the Department of Environment and Lands, St. John's.

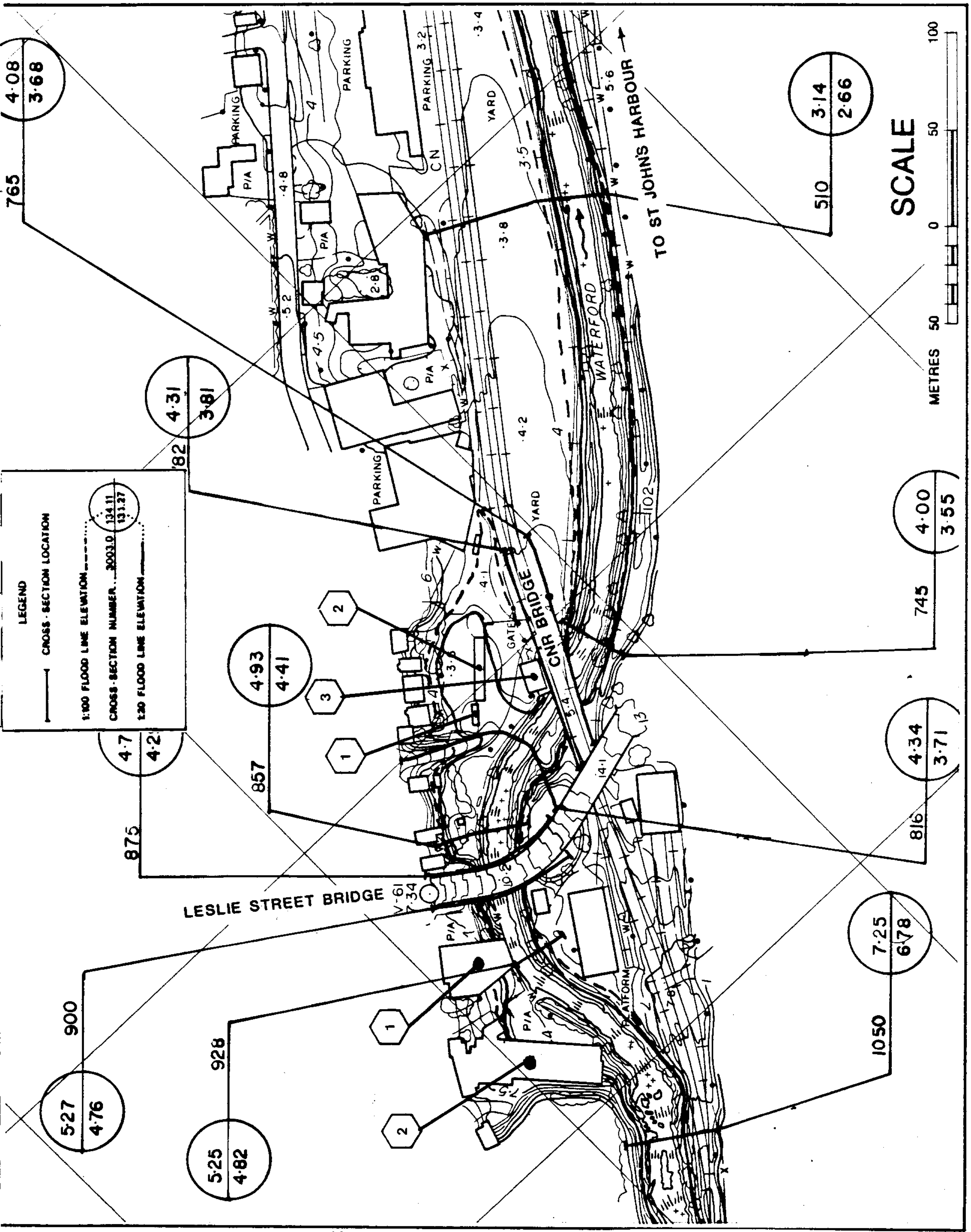
The mapping delineates a number of individual structures and groups of buildings which are within the floodplain. However, the number is fewer than would be anticipated from review of historical reports - most likely the result of problems and damages incurred during previous floods. It is noted, for example, that bridges which formerly obstructed flow passage at Leslie Street and at the mouth have been replaced by newer structures with greater capacity. New or reconditioned structures are also present at Commonwealth Avenue, Brookfield Road and Waterford Lane (St. John's Bridge).

In addition to improved bridge crossings, recent channelization at the mouth through the CNR yard and in the Donovans area south of the rail line appears to be adequately designed to convey flood flows without significant overbank flooding.

The above works have certainly reduced the potential for flood damage in many areas which were once flood-prone. There are still other areas which are within or close to the limits of flooding, which are summarized below.

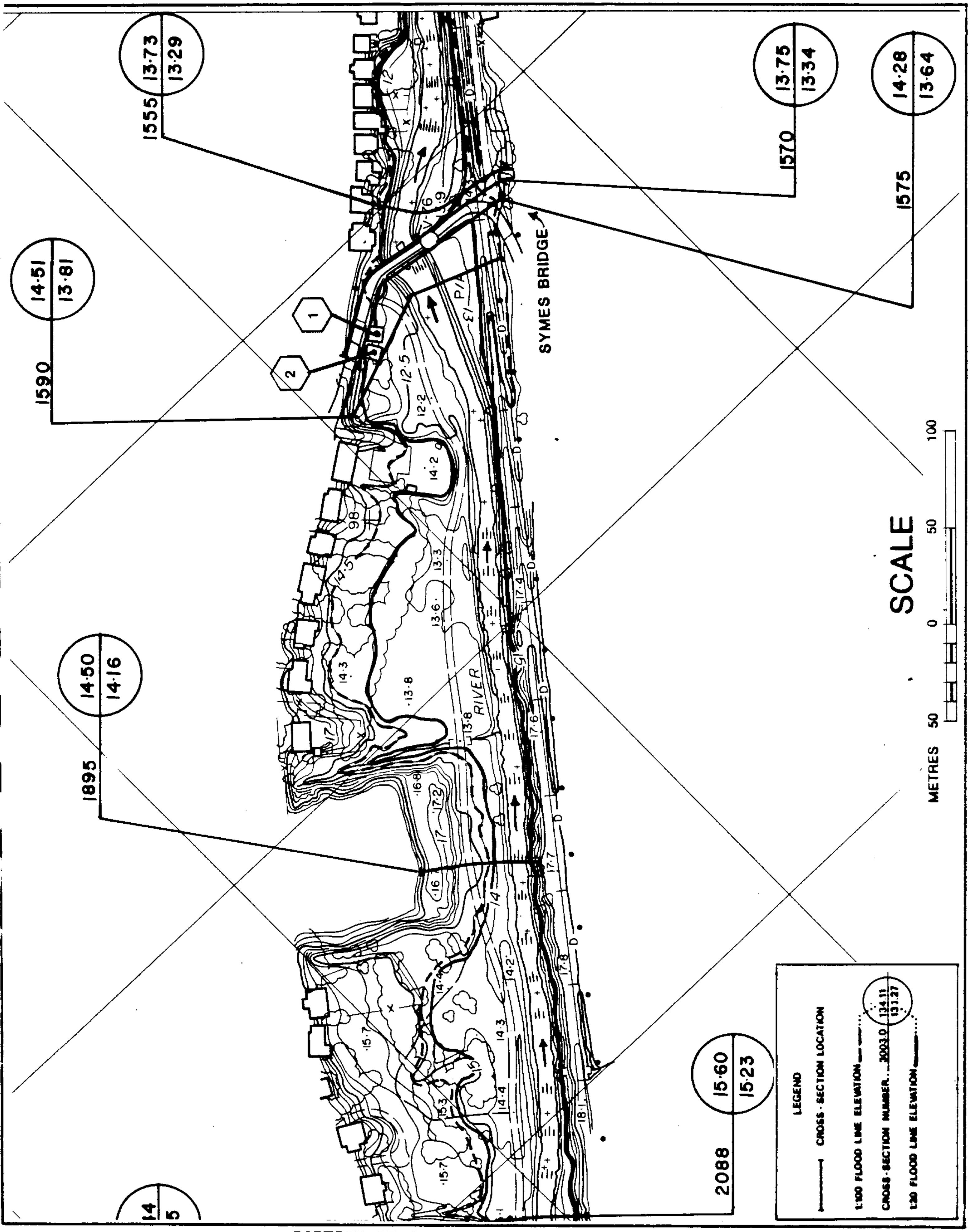
The flood profile mapping of Figure 7-1 shows the river at the Leslie Street overpass. Within the floodplain are portions of four commercial/railroad buildings - two upstream and two downstream of Leslie Street. The first building upstream along the northbank is a massive stone structure with no apparent openings below the flood level. The first two structures below the bridge are elevated storage/transfer facilities of the CNR, and the gate house at the rail line has been built on a raised concrete foundation. The second structure on the north side west of the bridge has an opening below the 20 year level.

Figure 7-2 shows the river reach near Symes Bridge. Two homes on the north bank just west of the bridge are within the 20 year flood risk area,



PORTION OF FLOOD RISK MAPPING

FIGURE 7-1



PORTION OF FLOOD RISK MAPPING

FIGURE 7-2

although only one is likely to sustain flood damage.

At Kilbride (Figure 7-3), the Corpus Christi Church and 4-5 nearby buildings are in the floodplain. Another building just northwest of the Bowring Park Road Bridge will sustain flood damage to its lower/basement level at flows in the range of about the 100 year level.

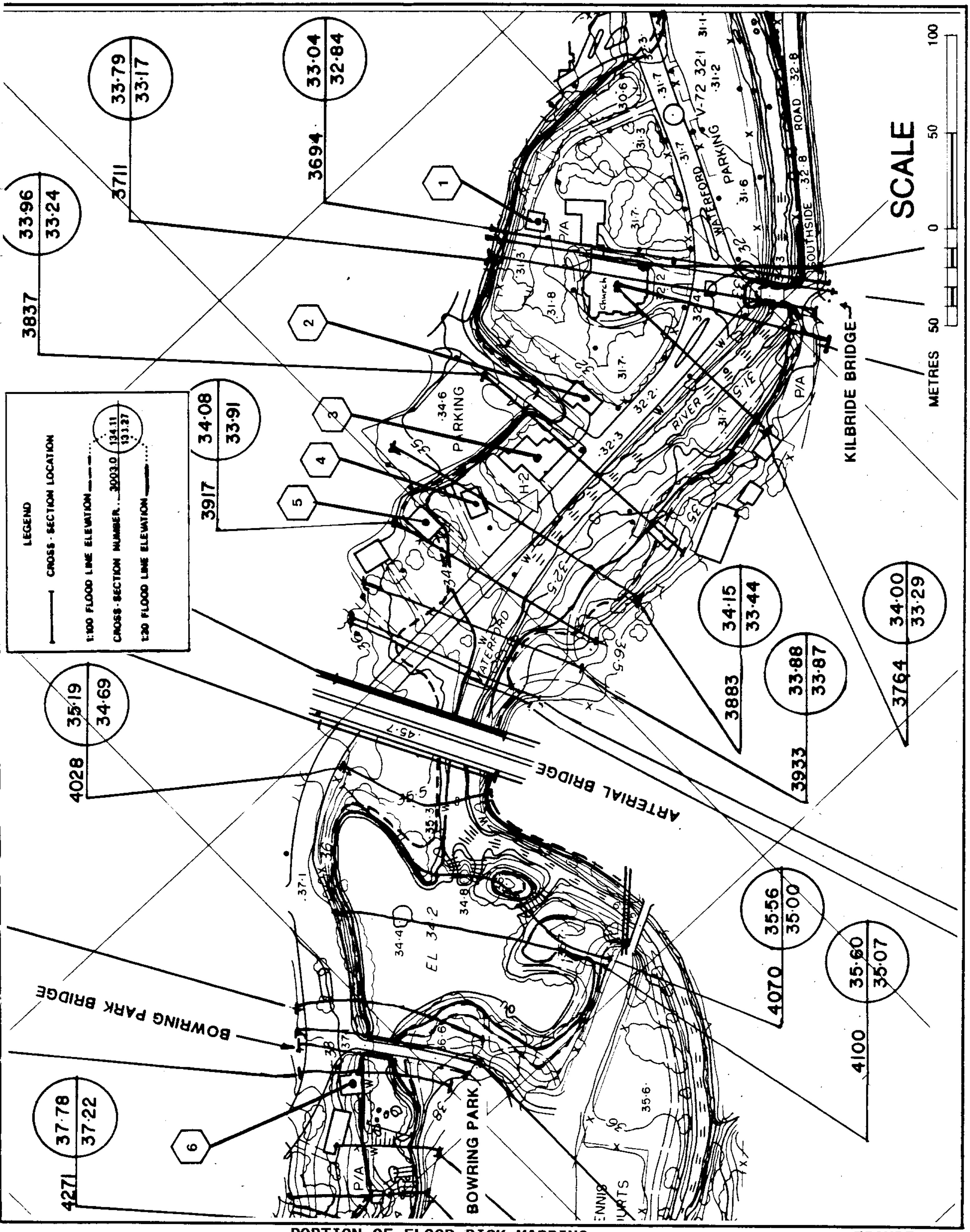
Figure 7-4 shows Brookfield Road Bridge and an industrial/concrete making plant (perhaps abandoned) across from the intersection of Doyle and Bishop Place in Brookfield Estates (400 m upstream of Brookfield Road Bridge). This plant is within the floodway and outbuildings/storage structures are within the flood fringe.

Figure 7-5 shows a portion of the river reach near the Steady Waters Footbridge. There are several structures on the fringe of the floodway (i.e. the end of Forest Avenue) which may sustain flood damage during a 1:100 year flood.

Figure 7-6 is the area immediately west of Figure 7-5, and the Figure shows the large flood prone basin upstream of the footbridge. There appears to be only the one building at the foot of Winston Avenue in a spill area of the floodplain, from this location upstream to Harnum Crescent in Mount Pearl.

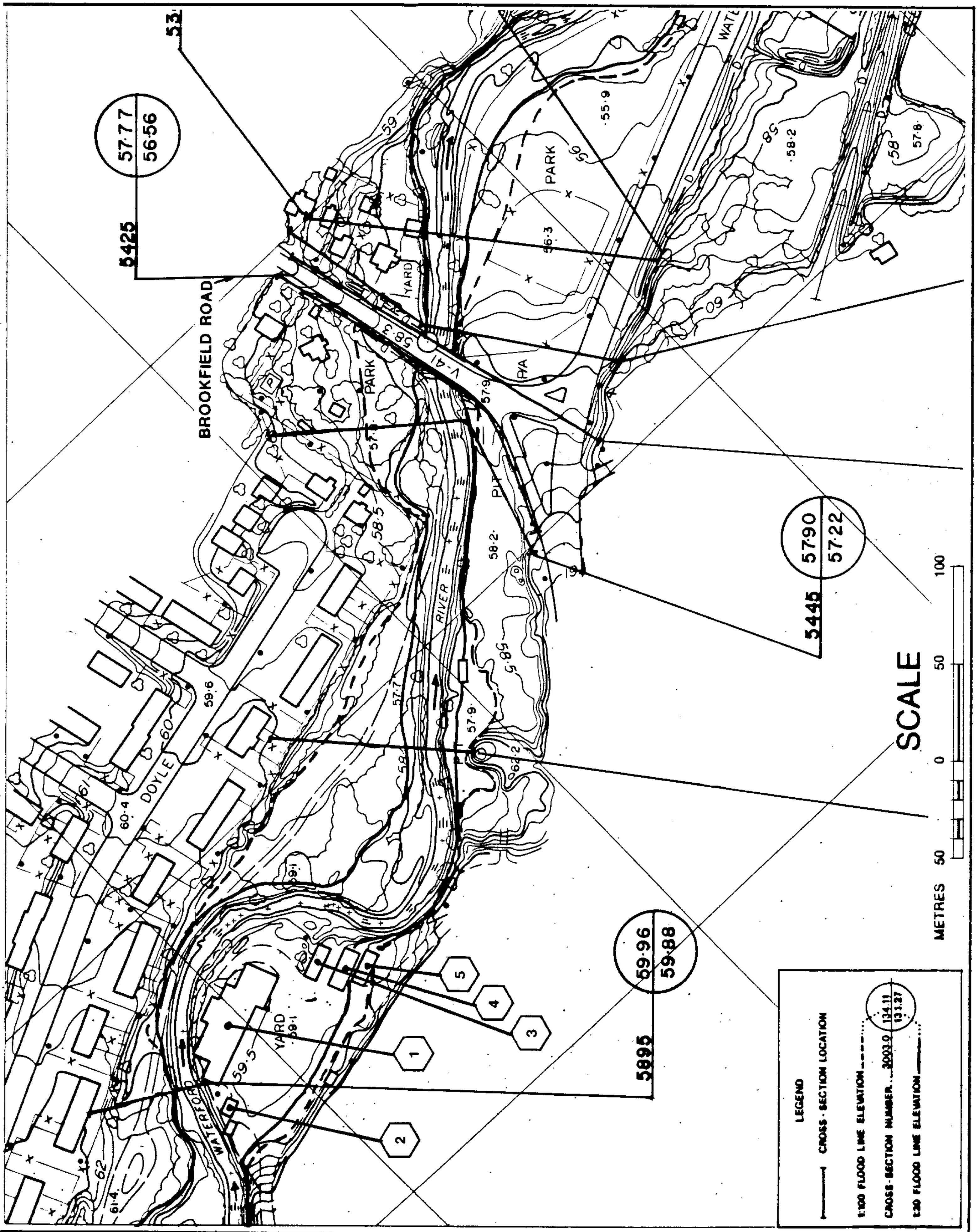
The last flood risk map (Figure 7-7) shows part of of the study area at Donovans Industrial Park. The Newfoundland Fibrply building lies on the floodway fringe and may sustain flood damage at flows in the range of the 1:20 year (or lower) return period. Two houses near the TCH also appear to be in the flood risk area, although only one would be likely to sustain flood damage.

Table 7-3 summarizes the structures at risk (19 to 20 buildings) and provides brief comments relating to possible damages and the flood hazard. It can be tentatively concluded that the average annual flood damage from the Waterford River is low.



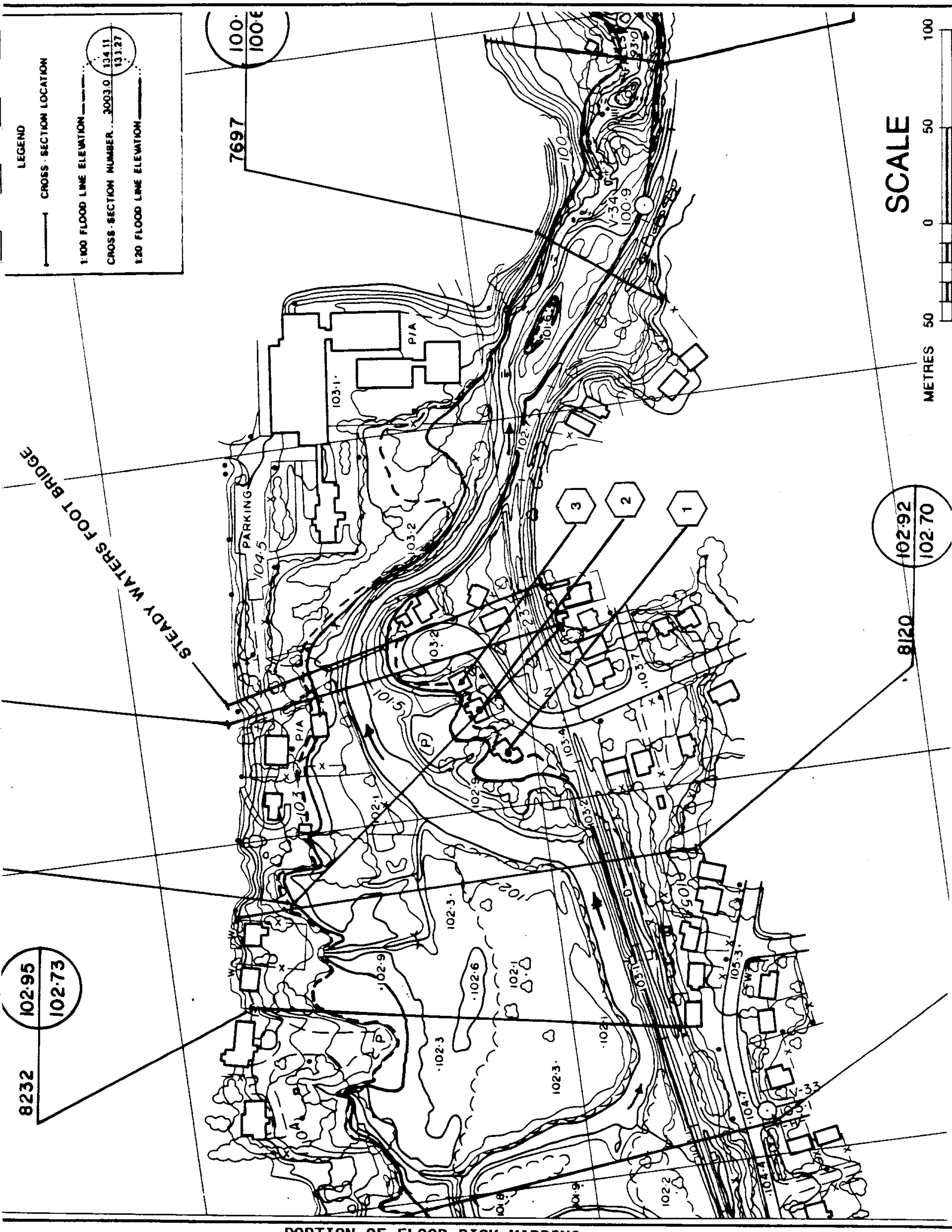
PORTION OF FLOOD RISK MAPPING

FIGURE 7-3



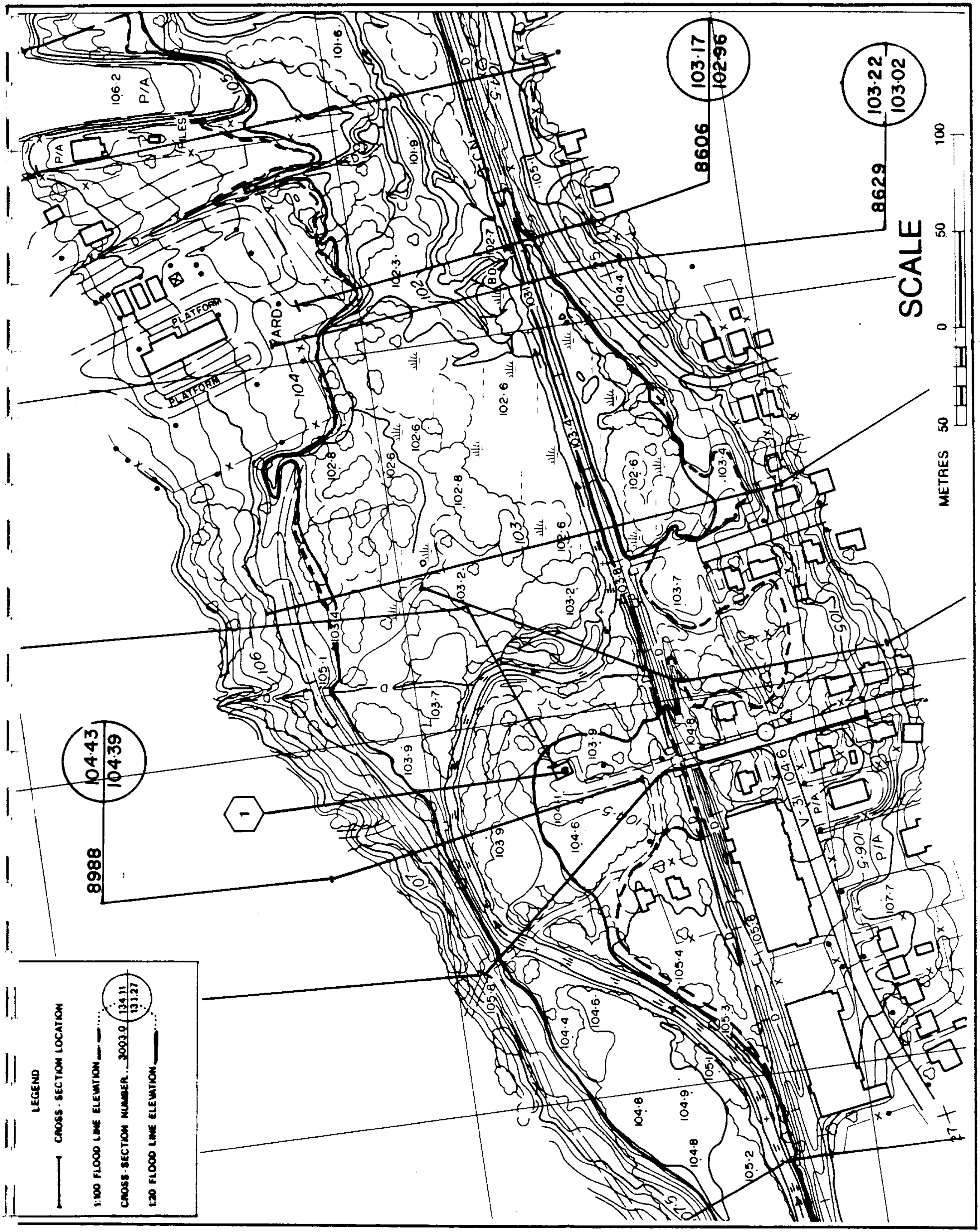
PORTION OF FLOOD RISK MAPPING

FIGURE 7-4



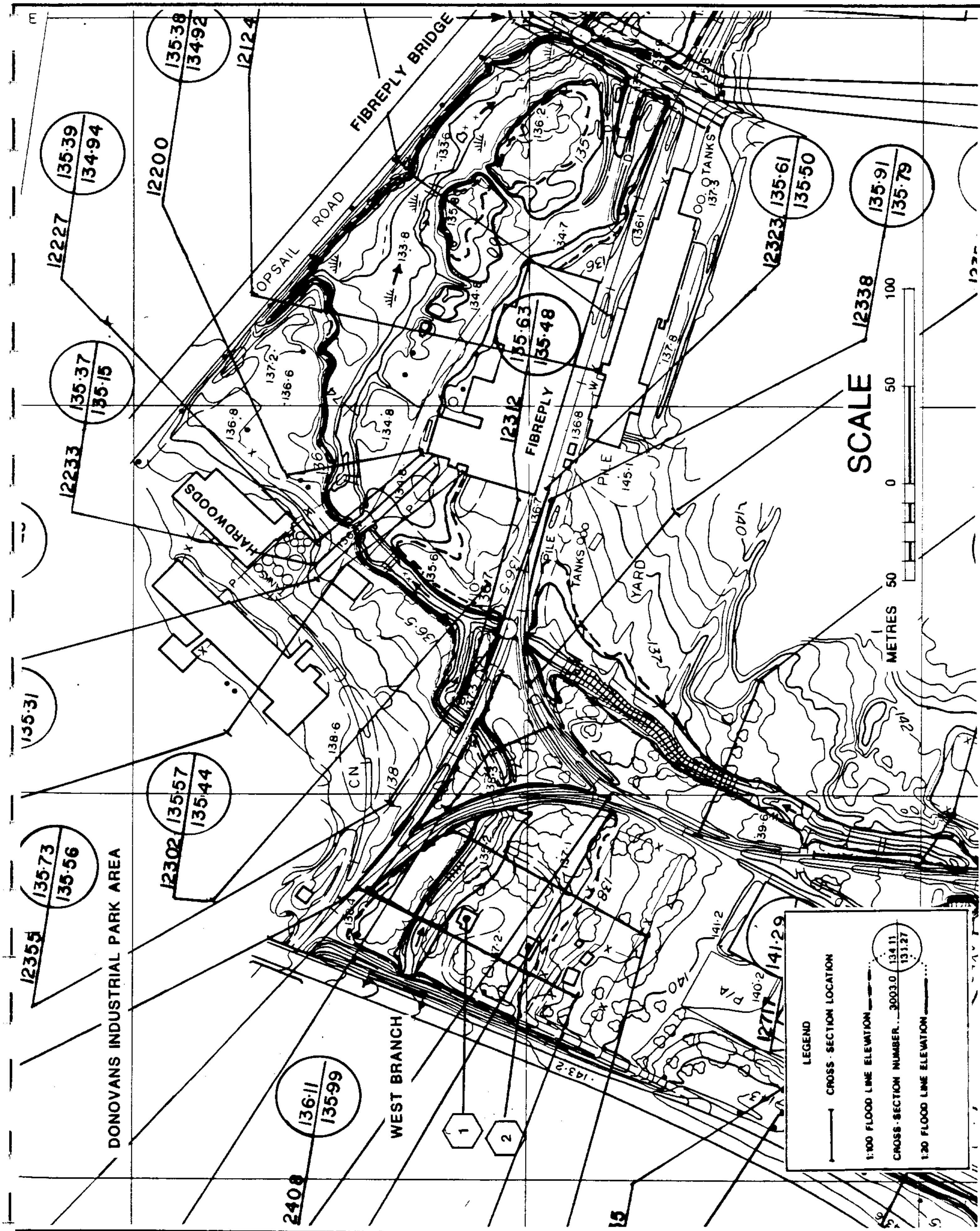
PORTION OF FLOOD RISK MAPPING

FIGURE 7-5



PORTION OF FLOOD RISK MAPPING

FIGURE 7-6



PORTION OF FLOOD RISK MAPPING

FIGURE 7-7

TABLE 7-3

SUMMARY OF FLOOD-PRONE STRUCTURES

Figure	Affected Structures	Comment
7-1	- four in risk area - eleven on fringe	- possible damage to contents of three structures (one a shed)
7-2	- 2 homes in risk area - 8 buildings on fringe	- possible damage to structure and contents of one home
7-3	- 5 buildings and Corpus Christ Church	- in the 1:20 year floodway
	- Bowring Park house	- damage to contents of lower floor during 1:100 year flows
7-4	- 1 commercial/industrial building and several out-buildings	- possibly an abandoned structure
	- flooding of Waterford Bridge Road	- potential traffic hazard
7-5	- 2-3 structures at Forest Avenue	- possible damage during 1:100 year flood. ONe is a garage
7-6	- one structure at Winston Avenue	- possible damage with floods in the 1:20 to 1:100 year range
7-7	- Newfoundland Fibrply	- possible flood damage with flows at, or less than, the 1:20 year case
	- 2 houses near the TCH culvert	- expected flood damage to one house with flows above the 1:20 year level
Total	19 - 20 structures	

8.0 FLOOD DAMAGE REDUCTION OPTIONS

The final step in this study calls for preliminary identification of flood damage reduction alternatives which could be employed along the Waterford River. Noteworthy in considering these alternatives is that the flood-prone areas shown in the final figures of Chapter 7 identify a "floodway" and a "flood fringe".

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

The floodway fringe is that portion of the flood risk area lying between the floodway and the outer limit of the area which is flooded on an average of once in 100 years. This zone generally receives less damage from flooding than the floodway.

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

- floodplain regulations
- acquisition
- flood proofing

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

- flood control dams
- channelization or dyking
- bridge opening expansions

An important factor which enters into the decision making process at this stage is that it is always desireable to pursue alternatives which are economically justifiable. A benefit-cost analysis, (which is beyond the scope of this project), is generally employed to give guidance in weighing all the possible alternatives. However, from experience on similar projects and that the damage-prone structures are distributed along the full length of the river, it appears that the second group of alternatives would present highly unattractive benefit-cost ratios. Hence, the focus of the following is on the first group of alternatives.

8.1 Flood Damage Prevention

A total of about 20 residential dwellings, institutional and commercial buildings currently fall within flood-prone areas of the Waterford River. Much of the river banks are now developed but there still remain some flood-prone areas which might otherwise be considered as desirable river-side locations.

8.1.1 Floodplain Regulations

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

Within the floodway fringe, the objective of reducing future damages can be achieved if effective flood-proofing measures are incorporated in the design of new structures and subsequently carried out. This also applies to existing buildings in the floodway fringe where flood-proofing measures can substantially reduce the amount of future damage during a flood situation. Several of these structures are discussed in Section 8.2.

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

8.1.2 Acquisition

Another alternative to reduce the potential for future damage in flood-prone areas is to acquire the undeveloped lands and properties and damage-prone structures. The option of overall acquisition of all existing properties in the flood hazard zone does not appear immediately feasible because the cost and social disruption would be excessive. It may be advantageous, however, to gradually acquire some of the most damage-prone structures as they come on the market or when they have sustained severe flood damages. Several of these are identified in Section 8.2.

8.1.3 Flood Proofing

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

- installation of permanent or temporary closures at low level openings in structures

. raising structures on fill, columns or piers

. construction of floodwalls or low berms around structures.

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

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

Flood proofing also entails combinations of closure and/or elevation of certain structures, with berming around groups of other structures. The group of structures near Corpus Christi Church are an example of a development which could be protected by a flood wall.

8.2 Recommended Options

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

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

2) It is recommended that the following should be considered if it is desireable to provide physical flood protection for existing buildings.

(a) Flood proofing by elevation for:

- the two houses in the flood risk area just north west of Symes Bridge (Figure 7-2)
- the house/seasonal residence in the 1:100 year flood zone north of Winston Avenue, Mount Pearl (Figure 7-6)
- two houses east of the TCH culvert near Donovans Industrial Park (Figure 7-7)
- the four buildings west of Corpus Christi Church and one building north of it (Figure 7-3)

(b) Flood proofing by the installation of permanent or automatic closures for:

- the second commercial building west of Leslie Street Bridge (Figure 7-1)
- the commercial/industrial building (possibly abandoned) on the south side of the river upstream of Brookfield Road Bridge (Figure 7-4). The possibility of acquiring this property for a park could also be considered
- entry points at the Newfoundland Fibrply facility in Donovans Industrial Park (Figure 7-7)
- the low level openings at Corpus Christi Church (Figure 7-3)

(c) Flood proofing by berms/fill or road regrading for:

- the two railway service or storage buildings in the 1:20 year flood zone east of Leslie Street Bridge. Here, about 40 m of the access road to the Gate house could be regraded and protected by rip rap to reduce the frequency of the flood hazard (Figure 7-1)

- the two houses on the east side of the end of Forest Avenue, Mount Pearl (Figure 7-5). A 25 m long, relatively low berm would possibly benefit these houses in the fringe on the 1:100 year flood zone

- the house on the north bank of the river just west of Bowing Park Road Bridge (Figure 7-3). Lower windows in the finished basement of this home are approximately at the 1:100 year level and a low (±0.3 m) wall or berm might be considered for protection.

- the area near Corpus Christi Church where the church and five buildings appear to be located in a former meander in the river (Figure 7-3). A floodwall (about 1-2 m high) might also be considered, but this and road regrading would be considerably more expensive/less cost beneficial than equally effective flood-proofing approaches.

In conclusion, there are damage reduction options which may be applied to reduce future and existing problems along the Waterford River. Most may be carried out by the individual owners, but regulations to minimize future developing problems and to facilitate correction of existing ones rests in the hands of local government.

9.0 REFERENCES

Batterson, M.J. 1984. "Surficial Geology of the Waterford River Basin St. John's, Newfoundland," Urban Hydrology of the Waterford River Basin Technical Report T-1.

City of St. John's' 1985. "Land Use Zoning and Subdivision Regulations."

Environment Canada. 1987. "Storm Rainfall in Canada," Canadian Climate Centre, Downsview, Ontario.

Environment Canada and Newfoundland Department of Environment. 1987. "Waterford River Basin Urban Hydrology Study - Data Summary Report" Vol. 1 and 2, Water Res. Branch, IWD, Atlantic Region (Draft Report).

Environment Canada and Newfoundland Department of Environment. 1986. "Land Use Report," Urban Hydrology Study of the Waterford River Basin, Technical Report UHS-WRB 1.3, Water Planning and Management Branch, Inland Waters Directorate.

Environment Canada and Newfoundland Department of Environment. 1986. "Waterford River Basin Flood Study," for Waterford River Basin Urban Hydrology Study, WPMB, IWD, Atlantic Region (Draft Report).

Environment Canada. 1986. "Surface Water Data Reference Index," IWD Wat. Res. Br., Water Survey of Canada, Ottawa.

Environment Canada. 1985. "Surface Water Data - Atlantic Provinces," (various years), IWD Wat. Res. Br., Water Survey of Canada, Ottawa.

Environment Canada. 1984. "Historical Streamflow Summary, Atlantic Provinces," IWD Wat. Res. Br., Water Survey of Canada, Ottawa.

Environment Canada. 1985. "Waterford River Basin Urban Hydrology Study Watershed Modelling - HYMO," WPMB-IWD, Atlantic Region (Draft Report).

Environment Canada. 1984. "St. John's Airport Rainfall Intensity - Duration Frequency Values," Atmospheric Environment Service, Canadian Climate Centre, Downsview, Ontario.

Environment Canada. 1974. "Atlantic Provinces Soil Capability for Agriculture." Canada Land Inventory, Lands Directorate.

Henderson, E.P. 1972. "Surficial Geology of the Avalon Peninsula, Newfoundland." Geological Survey of Canada, Memoir 368.

Hariga, P.K. 1981. "Soils of the Avalon Peninsula," Newfoundland Soil Survey Report No. 3, Agriculture Canada.

Kindervater, A.D. 1980. "Flooding Events in Newfoundland and Labrador - An Historical Perspective." Water Planning and Mgt. Branch, Inland Waters Directorate, Atlantic Region, Report 80-WPMB-4.

Newfoundland Department of Environment. 1986. "The Rainstorm Event of April 11, 1986 and Associated Drainage Problems in the Greater St. John's - Mt. Pearl Area," C. Worthman, Water Investigations Branch, Water Resources Division.

Newfoundland Department of Environment. 1986. "Installation and Testing of the Monitoring Well Network for the Waterford River Basin Urban Hydrology Groundwater Branch.

Newfoundland Department of Environment. 1985. "The Rainstorm Event of May 24-25, (1985) and Associated Drainage Problems in the Greater St. John's-Mt. Pearl Area," S. Costello, Water Investigations Branch, Water Resources Division.

Newfoundland Department of Environment and Environment Canada. 1986. "Users Guide Regional Flood Frequency Analysis for the Island of Newfoundland." Canada-Newfoundland Flood Damage Reduction Program.

Newfoundland Department of Environment and Environment Canada. 1984. "Regional Flood Frequency Analysis for the Island of Newfoundland." Canada-Newfoundland Flood Damage Reduction Program.

Rhodenizer, C. 1972. "Bedrock and Surficial Geology of the St. John's, Newfoundland, Area." B.Sc. Thesis, Memorial University, Dept. of Earth Science.

Rowney, A.C. and P.E. Wisner. 1985. "QUALHYMO User Manual," for Ontario Ministry of the Environment.

Vanderveer, D. 1975. "The Surficial Geology of the St. John's Area, Newfoundland, with Special Emphasis on Gravel Resources." Newfoundland Mineral Development Division, Department of Mines and Energy, open file IN/232.

Wisner, P.E. and A.C. Rowney. 1986. "QUALHYMO - An Alternative Continuous Quality Simulation Model," un-referenced paper.

Wisner, P.E. and A.C. Rowney, University of Ottawa, QUALHYMO - An Alternative Continuous Quality and Quantity Simulation Model. Proceedings of Conference on Stormwater and Water Quality Management Modelling, Toronto, 1985.

Rowney, A.C. and P.E. Wisner, University of Ottawa, QUALHYMO User Model - Release No. 1.0, 1984.

Williams, J.R. and Haan, R.W., "HYMO, A Problem Oriented Computer Language for Hydrologic Modelling", ARS-5-9, U.S. Dept. of Agriculture, 1973.

Wisner, P.E., P. Chung, C. P'Ng, A. Kassim, A. Lam, W. Liang and H. Fraser, "IMPSWM Methodology", 2nd Ed., University of Ottawa, Ottawa, 1983.

Soil Conservation Service, "National Engineering Handbook Section 4 Hydrology", S.C.S., U.S. Dept. of Agriculture, 1969.

Nash, J.E., "The Form of the Instantaneous Unit Hydrograph", International Association of Scientific Hydrology, Pub. 45, 3 (1957).

Williams, J.R. and R.W. Haan, "HYMO, A Problem Oriented Computer Language for Hydrologic Modelling", ARS-5-9, U.S. Dept. of Agriculture, 1973.

University of Waterloo Research Institute, "Flood Routing Sensitivity Study", March 1984.

U.S. National Weather Service, "Snow Accumulation and Ablation Model", NOAA Technical Memorandum NWS-HYDRO-17, 1973.

Hogg, W.D. 1980. "Time Distribution of short Duration Storm Rainfall in Canada," Proc. Can. Hydrology Symposium, NRC Canada, pp. 53-63.

Hogg, W.D. 1982a. "Distribution of Rainfall with Time: Design Considerations," Paper presented at the AGU Chapman Conference on Rainfall Rates,. Urbana, IL.

Hogg, W.D. 1982b. "Twelve Hour Storm Statistics Package." Hydrometeorology Division, Atmospheric Environment Service, Downsview.

Environment Canada. 1987. "Short Duration Rainfall IDF data for St. John's Newfoundland." Atmospheric Environment Service, Canadian Climate Centre, Downsview, Ont.

Watt, W.E., Chow, C.A., Hogg, W.D., Lathem, K.W., 1986. "A 1-H Urban Design Storm for Canada." Can. J. Civ. Eng., Vol. 13, No. 23.

SUPPORT DOCUMENTATION

DRAWINGS

Sewers and Drainage

Mount Pearl - Newtown Storm Sewer System - 1978 updated to 1981 - scale 1:4800 - St. John's Metropolitan Area Board

Mount Pearl - Plan/Profile of Services - New Town (8 sheets)

Mount Pearl - Storm Sewer Plan - 1979 updated to 1981 scale 1:15,000

St. John's - Plan of Storm and Sanitary Network, Scale 1:12,000

Kilbride - Storm Sewer Disposal System - 1978 updated to 1981 - scale 1:4800 - St. John's Metropolitan Area Board

Kilbride - Storm Water Drainage 1980, Scale 1:5000, St. John's Metro Area Board

Cowan Heights Development - Streets and Services, Scale 1:1250, St. John's Housing Corp.

Bridges/Roads

Dunnes Lane Culvert - Dept. Highways Bridge Office 1963

Dunnes Corner Bridge - Dept. Highways Bridge Office 1963

Commonwealth Ave. Bridge and Extension - Dept. Trans. and Communication 1923, 1963, 1981

Brookfield Road Bridge - Canadian British Consult. 1972

St. John's Bridge - Dept. of Public Works Bridge Office, 1955

Waterford Bridge/Kilbride - Elliot and Elliot Ltd., 1986

Symes Bridge Renovations - CBCL Limited, 1979

Trans Canada Culvert - Dept. Highways Bridge Office, 1958

Marine Elevator and Transfer Facility - NDAPD, 1981

Street map - St. John's, Kenting Earth Sciences, 1987

Symes Bridge Renovations - CBCL Ltd., 1979

Leslie St. Area - New Overpass and New Bridge, General Layout - Canadian National Railways, 1961

Squire's Avenue Bridge - CBEC Consultants, 1964

Long Bridge - 1927 (now gone)

Brookfield Bridge - Temporary Footbridge, CBCL, 1972

Land Use

Mapping of land use categories 1973, 1981, 1984. Scale 1:12,500, Water Planning and Management Branch, Planning Division

Zoning By-Law Zoning Plan - St. John's 1981. Scale 1:6000. City Planning Office, St. John's

Existing Land Use Survey - 1979. Scale 1:1250. St. John's Municipal Council Planning Office

Kilbride Development Scheme - Proposed Land Use 1980, St. John's Metropolitan Area Board, Scale 1:5000

Topographic and Hydraulic Data Mapping

Cross-section location mapping for Waterford River Basin Flood Study - Mount Pearl, Kilbride, and Donovans Flood Study Reach

Flood Hazard mapping for Waterford River Basin Urban Hydrology Study - Donovans, Mt. Pearl (scale 1:2500), and Kilbride areas (1:1250)

Topographic mapping - 1:50,000 scale c 1977
- 1:25,000 scale c 1973
- 1:12,500 scale c 1975
- 1: 2,500 scale c 1981

Topographic Mapping Base for Flood Risk Mapping, 1985. Scale 1:2500, McElhanney Mapping Services, 6 sheets

Geodetic Bench Marks and City of St. John's Bench Marks in the Vicinity of Waterford River in St. John's

Other Data

Tables of Land Use, Soil Type, Drainage Area and Hydrologic Soil Cover Complex Numbers for 1981 Hydrologic Modelling of the Waterford Basin

Tables of Snow Course Data, 1981-82 and 1982-83 for Waterford River Urban Hydrology Study

Background Data Relating to Snow Course Observation Program for Waterford River Urban Hydrology Study

Newfoundland Dept. of Environment, 1986. Summary File of Newsclippings Relating to Flooding from 1948 to 1986

Newfoundland Dept. of Environment. 1986. Flood Study Comparison Sections (between recent and cross-sections from early 1980's)

Air Photography

- 1966	1:16,800	B&W
Nov 1977	1: 7,000	B&W
May 1984	1: 7,000	B&W
Jun 1985	1: 5,000	B&W

Ground Photography

Newfoundland Department of Environment. 1984-1986. Photographs and Slides of Waterford River and Bridges.

Environment Canada Inland Waters Directorate. Binder of Slides Covering River Areas in 1980 and Photographs and Notes Relating to October 26, 1983 Flood Event.

APPENDIX 1.0

QUALHYMO MODEL AND MODEL ENHANCEMENTS

APPENDIX 1.0 QUALHYMO MODEL AND ENHANCEMENTS

In generating runoff from pervious land segments, the QUALHYMO model uses the U.S. Soil Conservation Service (1969) procedure to determine the excess moisture input from the pervious area. The soil moisture deficit of soils and the initial abstraction are updated by the model for each event to provide an accounting of initial moisture conditions. The initial abstraction is reduced by the antecedent precipitation over the preceding twenty-four hours and will recover to its user-specified maximum in the absence of precipitation over the foregoing period of time. In the case of the soil moisture deficit, current values preceding an event are computed as a function of a variable Antecedent Precipitation Index (API). This index is based on daily rainfall totals weighted by a recession coefficient (having a system memory of approximately thirty days).

Runoff volume computations for impervious areas are carried out by reducing precipitation with a small initial abstraction and subsequently applying a runoff coefficient (i.e. 0.95). Inter-event updating of the initial abstraction is similar to the technique for pervious areas.

The QUALHYMO model calculates flow rates from flow volume by convolution of excess precipitation, with two unit hydrograph shapes proposed by Nash (1957) and Williams (1973). Since runoff from pervious areas is convoluted separately from impervious areas, the same or different unit hydrograph shapes can be used at the discretion of the modeller. Herein, the Williams and Nash approaches were used for the pervious and impervious areas, respectively.

A flow path connected to an outlet is determined for both pervious and impervious areas. The flow path for the impervious areas may be overland, channel and pipe flow or combinations of these. The flow path for pervious areas can be calculated or left to the model to compute based on physical characteristics of the drainage area.

A kinematic approximation is used within QUALHYMO to route flows along river reaches. Hydraulics in the reach are represented either by a Manning flow equation or specified rating curve. Depth-flow velocity and depth-section area relationships for the channel are calculated by the model and used subsequently for flow routing purposes.

Calculation of base flow in the model is carried out with a single reservoir representing groundwater storage. A net inflow and outflow from the reservoir is calculated for each model time step with inflow taken as the difference between precipitation and runoff minus any losses to initial abstraction. Outflow is expressed as a function of a baseflow recession constant times the groundwater reservoir storage. Losses to deep groundwater storage are estimated as a constant proportion of the outflow from the groundwater storage reservoir and effectively reduce the contribution to base flow.

A.1.1 Model Enhancements

In order to accommodate the flow travel and attenuation process within water courses of the Waterford River basin, the kinematic approximation employed in the original QUALHYMO model for streamflow routing was replaced by a hydrologic Muskingum method which incorporates the variable storage coefficient (VSC) method (Williams and Haan, 1973). The latter technique accounts for the variation in water surface slope and has been tested successfully on Ontario streams (Waterloo Research Institute, 1984).

Another enhancement was the addition of the U.S. National Weather Service (NWS) River Forecast System snowmelt model (1973) in order to improve the computational accuracy of melt and moisture totals in the watershed during rainfall and snowmelt occurrances. The NWS snowmelt model employs a temperature index procedure in which melt rates are proportional to the difference between the mean air temperature and a base temperature (typically 0°C) during periods without precipitation. A seasonal variation in the melt factor can be simulated to reflect the increase in solar radiation and the decrease in the albedo of a snow cover as the winter period progresses. During periods of precipitation, a semi-empirical energy balance approach takes into account the net long wave radiation transfer to a snow cover, the latent heat transfer or sublimation of water vapour, sensible heat transfer due to the heat content of the air, and heat transfer to the snow cover caused by precipitation. Short wave radiation is considered to be zero during the occurrence of precipitation due to the presence of cloud cover. Meteorologic data required for computation is limited to air temperature and precipitation intensity during the snowmelt period.

Outflow of liquid water from the snowpack is differentiated from snowmelt in the NWS computational approach by a snowpack heat accounting technique, which indicates whether water will be in a liquid or solid phase. Liquid water is retained in the snowpack against gravity drainage and the portion which exceeds a specified capacity is transmitted as outflow after a time lag which represents routing of meltwater through the snowpack.

The runoff which occurs within small urban catchments can be quite sensitive to short duration intensities. As a final enhancement, QUALHYMO was also modified to allow either 15 minute duration rainfall amounts to be read into the model, or rainfall amounts for any time fraction of hourly inputs. The latter is particularly useful for simulating runoff from design rainfall storms.

A.1.2 CN Transformation for QUALHYMO Input

Rainfall runoff in pervious areas is calculated by the QUALHYMO model using the SCS relation:

$$Q = \frac{(P - ABSPER)^2}{(P - ABSPER + S^*)}$$

where:

Q = cumulative depth of runoff (m)
P = cumulative depth of precipitation (mm)
ABSPER = initial abstraction (mm)
S* = loss parameter (mm)

In the QUALHYMO Model, S* and ABSPER are updated by the model with each precipitation occurrence in order to provide a continuous accounting of initial moisture conditions prior to runoff events. In the case of S*, this is accomplished by expressing S* as a function of a variable Antecedent Precipitation Index, the API.

The API is determined from the following relation:

$$API2 = APIK * API1 + P1$$

where:

APIK is a coefficient (typically taken as 0.9)
P1 is precipitation within time step 1, and the
API subscripts refer to conditions at the beginning of time step 1 and time step 2.

The relationship which relate S* and API is:

$$S^* = SMIN + (SMAX - SMIN) * \exp(-SK * API)$$

where:

SMIN and SMAX represent the range in S*, and SK is a calibration parameter which defines the slope of soil moisture between the maximum and minimum values. These relationships are shown graphically in the following figure and their development is described below.

In order to apply the QUALHYMO model to the Waterford River watershed, the following steps were taken:

i) Hydrologic soil-cover complexes (CN) were computed for the large catchments shown in Figure 4-1. A weighted average hydrologic soil-cover complex (CN) was calculated for each subcatchment to correspond to the Soil Conservation Service Antecedent Moisture Condition I, II and III (AMC condition).

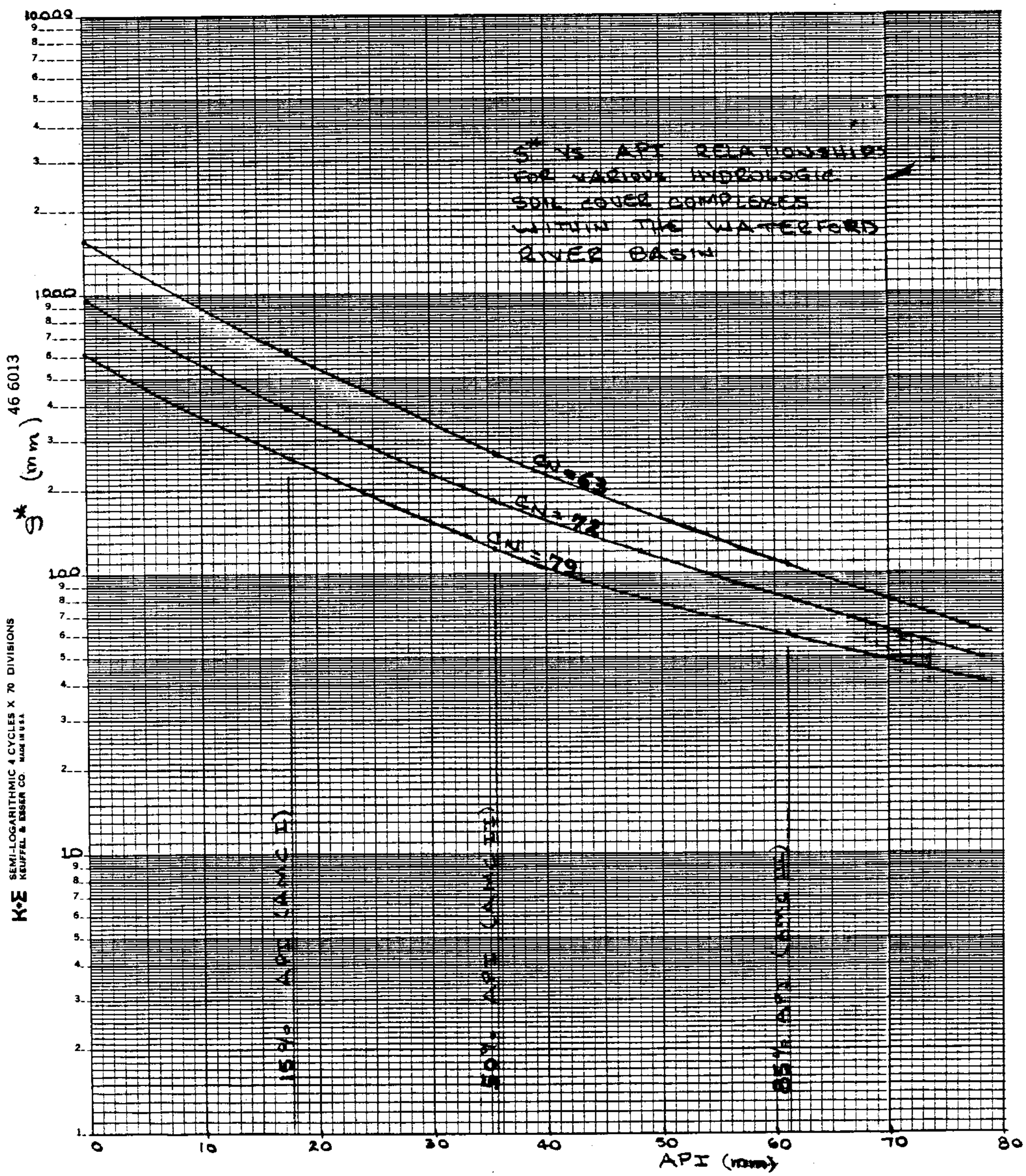
ii) The API sub-routine of the QUALHYMO model was then used in an analysis of the hourly precipitation records between June 1953 and December 1985 for the St. John's Airport station to calculate continuous API (Antecendent Precipitation Index) values. With these values, it is possible to initiate model simulations on any date with the knowledge that the API value for that date integrates all of the precipitation that precedes that date.

iii) In order to develop API values for use in design storm runoff simulations (Section 5.8), an API duration analysis was carried out to determine the API values corresponding to the 15, 50 and 85 percent time of exceedance. It was established that the API values of 17.8 mm, 35.8 mm and 61.6 mm corresponded to the foregoing percentage exceedances, respectively.

iv) An S* versus API curve was established for each subcatchment in Figure 4-1. In order to obtain the general shape of this curve, the hydrologic soil cover (CN) for each subcatchment corresponding to the antecedent moisture conditions, I, II and III (point i) above) was converted to the equivalent S* loss parameter by the relationship following the Soil Conservation Service procedure (1969).

$$S^* = \frac{1000 - 10}{CN}$$

Based on the experience of the model's author, the foregoing S* magnitudes were plotted at the API values corresponding to the 15, 50 and 85 percent points. Values of SMIN and SMAX were subsequently established by graphical curve extension, and SK (the curve slope) was computed from the exponential equation relating this parameter to S*, SMIN and SMAX.



S* VS API RELATIONSHIPS
FOR VARIOUS HYDROLOGIC
SOIL COVER COMPLEXES
WITHIN THE WATERFORD
RIVER BASIN
CN=63
CN=72
CN=79
15% API (AMC I)
50% API (AMC II)
85% API (AMC III)
S* (mm)
API (mm)
K+E SEMI-LOGARITHMIC 4 CYCLES X 70 DIVISIONS
KEUFFEL & ESSER CO. MADE IN U.S.A.
46 6013

Figure A1-1

APPENDIX 2.0

FREQUENCY ANALYSIS RESULTS
KILBRIDE 1959-87
(28 YEARS)

FREQUENCY ANALYSIS - GENERALIZED EXTREME VALUE DISTRIBUTION
kilbride s 124

SAMPLE STATISTICS

	MEAN	S.D.	C.V.	C.S.	C.K.
X SERIES	46.037	16.238	0.353	0.773	3.304
LN X SERIES	3.772	0.342	0.091	0.249	2.362

X(MIN)= 24.400
X(MAX)= 87.990
LOWER OUTLIER LIMIT OF X= 18.295

TOTAL SAMPLE SIZE= 28
NO. OF LOW OUTLIERS= 0
NO. OF ZERO FLOWS= 0

SOLUTION OBTAINED VIA MAXIMUM LIKELIHOOD

GEV PARAMETERS: U= 37.54 A= 11.080 K= -0.183

FLOOD FREQUENCY REGIME

RETURN PERIOD	EXCEEDANCE PROBABILITY	FLOOD
1.003	0.997	20.90
1.050	0.952	26.40
1.250	0.800	32.50
2.000	0.500	41.70
5.000	0.200	56.70
10.000	0.100	68.40
20.000	0.050	81.30
50.000	0.020	101.00
100.000	0.010	118.00
200.000	0.005	137.00
500.000	0.002	166.00

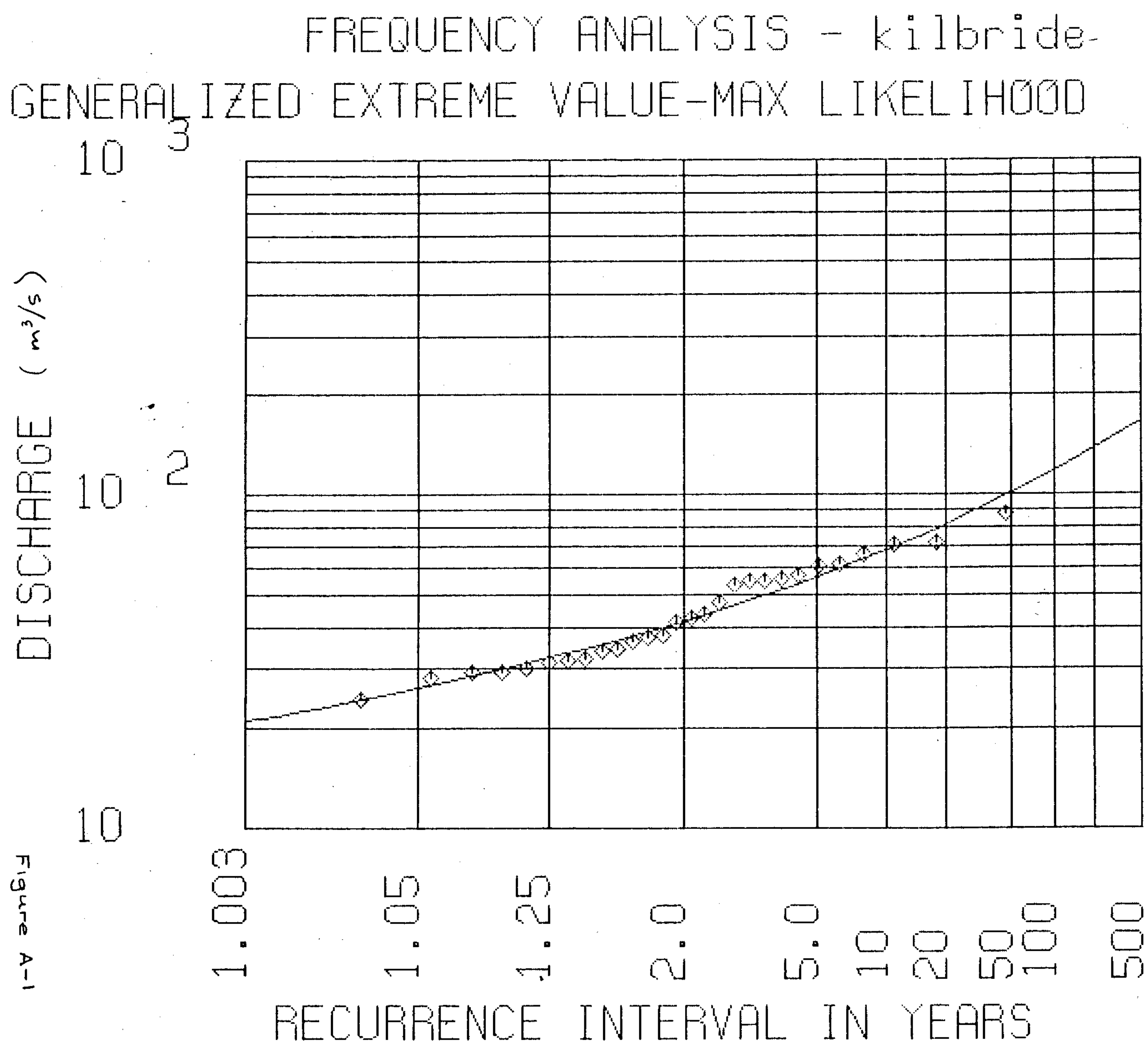

FREQUENCY ANALYSIS - kilbride-
GENERALIZED EXTREME VALUE-MAX LIKELIHOOD
10^3
10^2
10
DISCHARGE (m³/s)
1.003
1.05
1.25
2.0
5.0
10
20
50
100
500
RECURRENCE INTERVAL IN YEARS

Figure A-1

FREQUENCY ANALYSIS - THREE-PARAMETER LOGNORMAL DISTRIBUTION
kilbride s 124

SAMPLE STATISTICS

	MEAN	S.D.	C.V.	C.S.	C.K.
X SERIES	46.037	16.238	0.353	0.773	3.304
LN X SERIES	3.772	0.342	0.091	0.249	2.362
LN(X-A) SERIES	3.127	0.633	0.202	-0.189	2.556

X(MIN)= 24.400
X(MAX)= 87.990
LOWER OUTLIER LIMIT OF X= 18.295

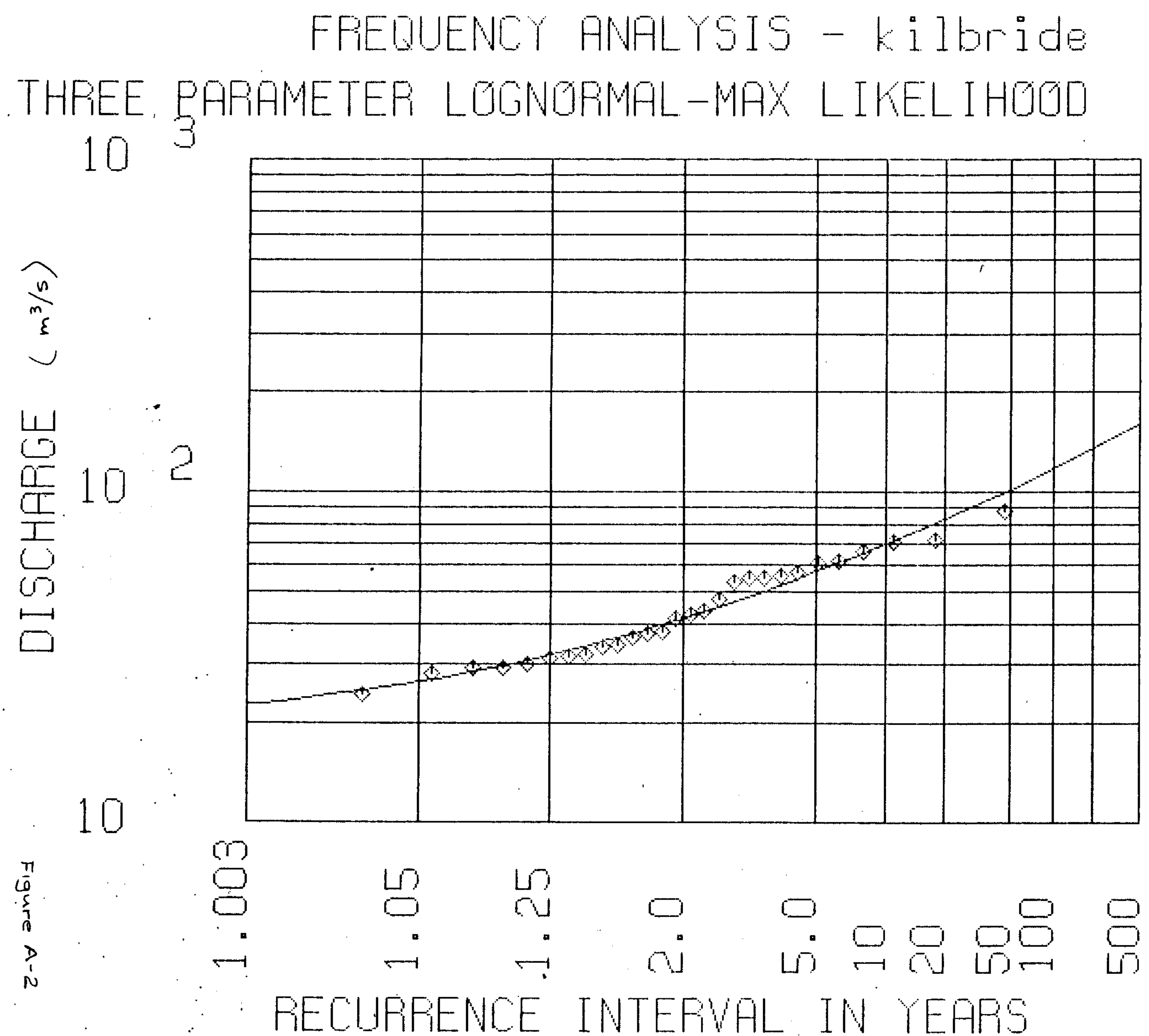
TOTAL SAMPLE SIZE= 28
NO. OF LOW OUTLIERS= 0
NO. OF ZERO FLOWS= 0

SOLUTION OBTAINED VIA MAXIMUM LIKELIHOOD

3LN PARAMETERS: A= 18.693 M= 3.127 S= 0.633

FLOOD FREQUENCY REGIME

RETURN PERIOD	EXCEEDANCE PROBABILITY	FLOOD
1.003	0.997	22.70
1.050	0.952	26.60
1.250	0.800	32.10
2.000	0.500	41.50
5.000	0.200	57.60
10.000	0.100	70.00
20.000	0.050	83.30
50.000	0.020	102.00
100.000	0.010	118.00
200.000	0.005	135.00
500.000	0.002	160.00



FREQUENCY ANALYSIS - kilbride
THREE PARAMETER LOGNORMAL-MAX LIKELIHOOD
10 3
10 2
10
DISCHARGE (m3/s)
1.003
1.05
1.25
2.0
5.0
10
20
50
100
500
RECURRENCE INTERVAL IN YEARS

Figure A-2

FREQUENCY ANALYSIS - LOG PEARSON TYPE III DISTRIBUTION
kilbride s 124

SAMPLE STATISTICS

	MEAN	S.D.	C.V.	C.S.	C.K.
X SERIES	46.037	16.238	0.353	0.773	3.304
LN X SERIES	3.772	0.342	0.091	0.249	2.362

X(MIN)= 24.400
X(MAX)= 87.990
LOWER OUTLIER LIMIT OF X= 18.295

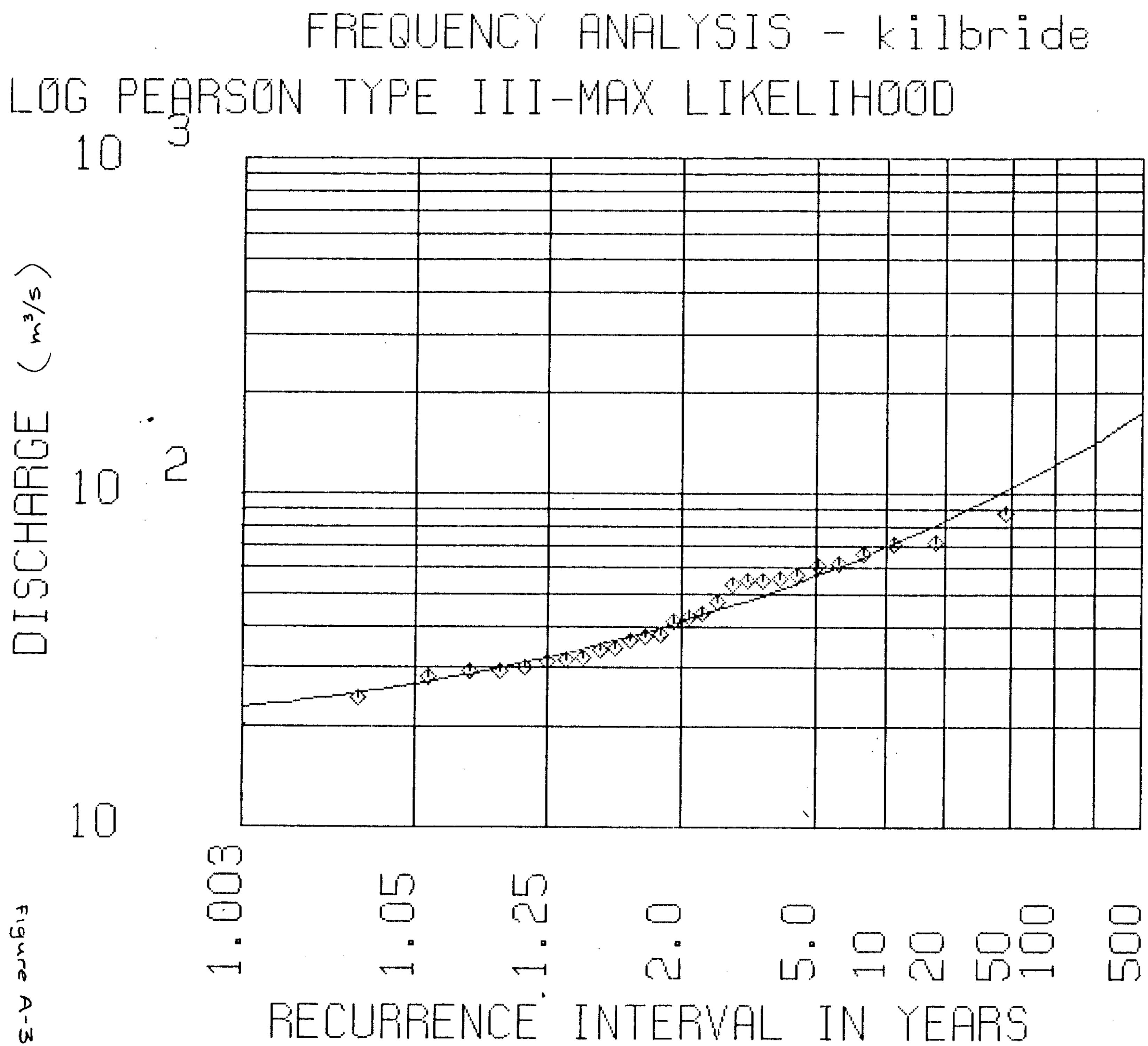
TOTAL SAMPLE SIZE= 28
NO. OF LOW OUTLIERS= 0
NO. OF ZERO FLOWS= 0

SOLUTION OBTAINED VIA MAXIMUM LIKELIHOOD

LP3 PARAMETERS: A= 0.1518 B= 5.331 LOG(M)= 2.963
M = 19.36

FLOOD FREQUENCY REGIME

RETURN PERIOD	EXCEEDANCE PROBABILITY	FLOOD
1.003	0.997	22.70
1.050	0.952	26.80
1.250	0.800	32.30
2.000	0.500	41.40
5.000	0.200	56.90
10.000	0.100	69.40
20.000	0.050	83.10
50.000	0.020	104.00
100.000	0.010	122.00
200.000	0.005	142.00
500.000	0.002	174.00



FREQUENCY ANALYSIS - kilbride
LOG PEARSON TYPE III-MAX LIKELIHOOD
10 3
10 2
10
DISCHARGE (m3/s)
1.003
1.05
1.25
2.0
5.0
10
20
50
100
500
RECURRENCE INTERVAL IN YEARS

Figure A-3

FREQUENCY ANALYSIS - WAKEBY DISTRIBUTION
kilbride s 124

SAMPLE STATISTICS

	MEAN	S.D.	C.V.	C.S.	C.K.
X SERIES	46.037	16.238	0.353	0.773	3.304
LN X SERIES	3.772	0.342	0.091	0.249	2.362

X(MIN)= 24.400
X(MAX)= 87.990
LOWER OUTLIER LIMIT OF X= 18.295

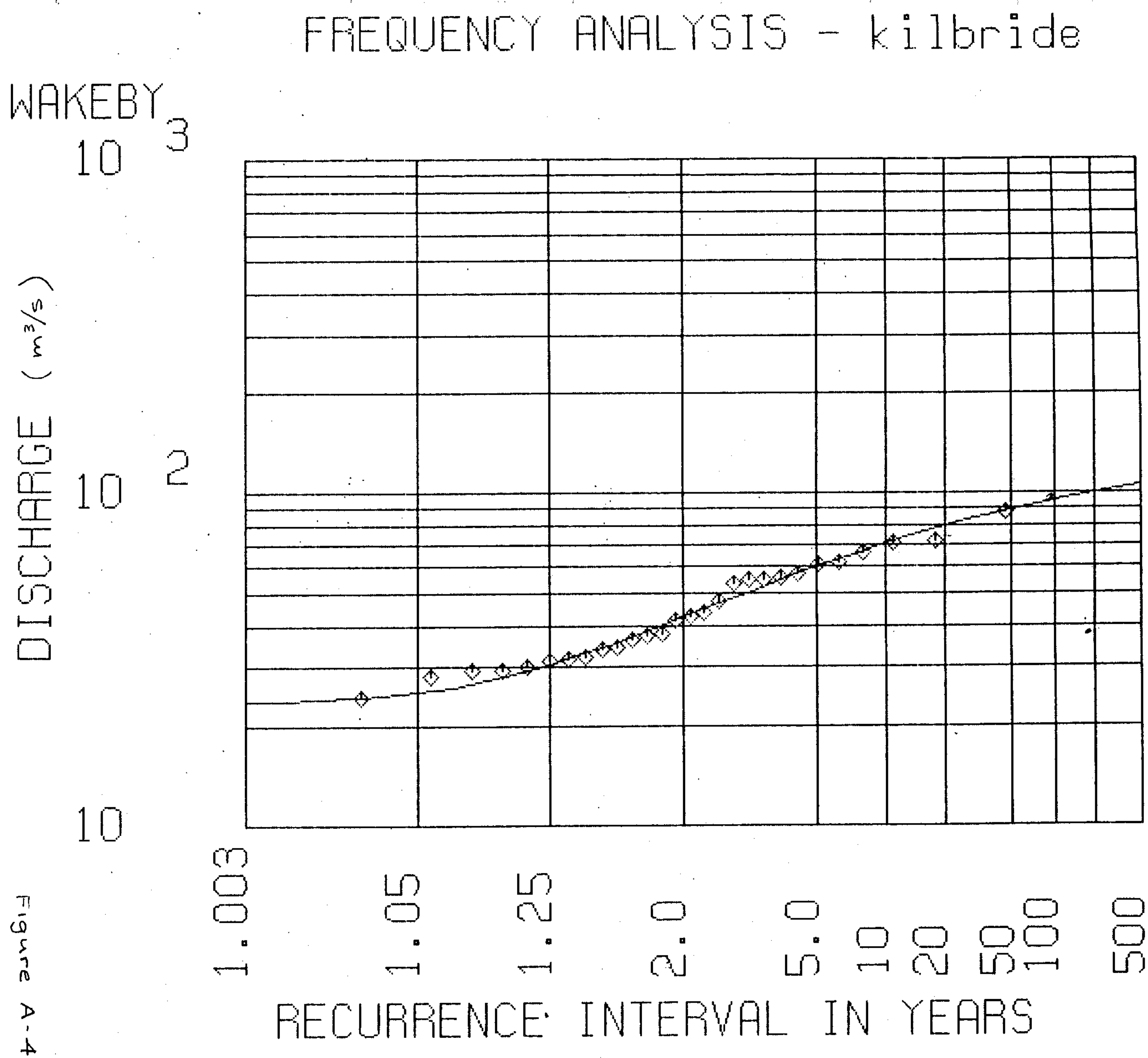
TOTAL SAMPLE SIZE= 28
NO. OF LOW OUTLIERS= 0
NO. OF ZERO FLOWS= 0

THE FOLLOWING WAKEBY PARAMETERS WERE OBTAINED BY ASSUMING M TO BE NON-ZERO. THE ITERATION ALGORITHM WAS NOT REQUIRED.

M= 23.607 A= 3.369 B= 3.01 C= -99.168 D=-0.251
DISTRIBUTION IS UPPER BOUNDED AT E= 0.1261E+03

FLOOD FREQUENCY REGIME

RETURN PERIOD	EXCEEDANCE PROBABILITY	FLOOD
1.003	0.997	23.70
1.050	0.952	25.30
1.250	0.800	30.70
2.000	0.500	42.40
5.000	0.200	59.90
10.000	0.100	70.50
20.000	0.050	79.40
50.000	0.020	89.00
100.000	0.010	94.90
200.000	0.005	99.90
500.000	0.002	105.00



FREQUENCY ANALYSIS - kilbride
WAKEBY
10^3
10^2
10
DISCHARGE (m^3/s)
1.003
1.05
1.25
2.0
5.0
10
20
50
100
500
RECURRENCE INTERVAL IN YEARS

Figure A-4

APPENDIX 3.0

FREQUENCY ANALYSIS RESULTS
KILBRIDE 1974 - 87
(13 YEARS)

WSC STATION NO= 124
WSC STATION NAME== ID NUMBER

MONTH	YEAR	DATA	ORDERED	RANK	PROB.	RET. PERIOD
(1)	(2)	(3) (CMS)	(4) (CMS)	(5)	(6) (%)	(7) (YEARS)
8	74	30.900	62.700	1	4.55	22.000
8	75	21.800	62.100	2	12.12	8.250
1	76	28.300	53.400	3	19.70	5.077
12	77	40.200	41.700	4	27.27	3.667
1	78	30.900	40.200	5	34.85	2.870
1	79	34.500	36.600	6	42.42	2.357
10	80	26.200	34.500	7	50.00	2.000
11	81	62.100	30.900	8	57.58	1.737
10	82	53.400	30.900	9	65.15	1.535
10	83	41.700	28.300	10	72.73	1.375
2	84	36.600	26.200	11	80.30	1.245
4	86	62.700	24.300	12	87.88	1.138
4	87	24.300	21.800	13	95.45	1.048

FREQUENCY ANALYSIS - GENERALIZED EXTREME VALUE DISTRIBUTION
124 = ID NUMBER

SAMPLE STATISTICS

	MEAN	S.D.	C.V.	C.S.	C.K.
X SERIES	37.969	13.678	0.360	0.879	3.570
LN X SERIES	3.581	0.343	0.096	0.415	3.119

X(MIN)= 21.800
X(MAX)= 62.700
LOWER OUTLIER LIMIT OF X= 17.053

TOTAL SAMPLE SIZE= 13
NO. OF LOW OUTLIERS= 0
NO. OF ZERO FLOWS= 0

SOLUTION OBTAINED VIA MAXIMUM LIKELIHOOD

GEV PARAMETERS: U= 30.86 A= 8.595 K= -0.233

FLOOD FREQUENCY REGIME

RETURN PERIOD	EXCEEDANCE PROBABILITY	FLOOD
1.003	0.997	18.50
1.050	0.952	22.40
1.250	0.800	27.00
2.000	0.500	34.20
5.000	0.200	46.30
10.000	0.100	56.30
20.000	0.050	67.70
50.000	0.020	85.60
100.000	0.010	102.00
200.000	0.005	121.00
500.000	0.002	151.00

FREQUENCY ANALYSIS - THREE-PARAMETER LOGNORMAL DISTRIBUTION
124 = ID NUMBER

SAMPLE STATISTICS

	MEAN	S.D.	C.V.	C.S.	C.K.
X SERIES	37.969	13.678	0.360	0.879	3.570
LN X SERIES	3.581	0.343	0.096	0.415	3.119
LN(X-A) SERIES	2.809	0.709	0.252	-0.212	3.386

X(MIN)= 21.800
X(MAX)= 62.700
LOWER OUTLIER LIMIT OF X= 17.053

TOTAL SAMPLE SIZE= 13
NO. OF LOW OUTLIERS= 0
NO. OF ZERO FLOWS= 0

SOLUTION OBTAINED VIA MAXIMUM LIKELIHOOD

3LN PARAMETERS: A= 17.371 M= 2.809 S= 0.709

FLOOD FREQUENCY REGIME

RETURN PERIOD	EXCEEDANCE PROBABILITY	FLOOD
1.003	0.997	19.70
1.050	0.952	22.50
1.250	0.800	26.50
2.000	0.500	34.00
5.000	0.200	47.50
10.000	0.100	58.50
20.000	0.050	70.60
50.000	0.020	88.50
100.000	0.010	104.00
200.000	0.005	120.00
500.000	0.002	145.00

FREQUENCY ANALYSIS - LOG PEARSON TYPE III DISTRIBUTION
124 = ID NUMBER

SAMPLE STATISTICS

	MEAN	S.D.	C.V.	C.S.	C.K.
X SERIES	37.969	13.678	0.360	0.879	3.570
LN X SERIES	3.581	0.343	0.096	0.415	3.119

X(MIN)= 21.800
X(MAX)= 62.700
LOWER OUTLIER LIMIT OF X= 17.053

TOTAL SAMPLE SIZE= 13
NO. OF LOW OUTLIERS= 0
NO. OF ZERO FLOWS= 0

SOLUTION OBTAINED VIA MAXIMUM LIKELIHOOD

LP3 PARAMETERS: A= 0.1907 B= 3.393 LOG(M)= 2.934
M = 18.80

FLOOD FREQUENCY REGIME

RETURN PERIOD	EXCEEDANCE PROBABILITY	FLOOD
1.003	0.997	20.10
1.050	0.952	22.70
1.250	0.800	26.70
2.000	0.500	33.80
5.000	0.200	46.60
10.000	0.100	57.40
20.000	0.050	69.70
50.000	0.020	88.90
100.000	0.010	106.00
200.000	0.005	126.00
500.000	0.002	159.00

APPENDIX 4.0

MANNINGS ROUGHNESS COEFFICIENTS

TABLE A-4

WATERFORD RIVER

MANNINGS ROUGHNESS VALUES

SECTION	LB*	RB	CH	SECTION	LB*	RB	CH
0.100	.050	.050	.040**	4176.0	.040	.040	.025**
0.300	.050	.050	.030***	4356.0	.040	.040	.030***
120.0	.050	.050	.040	4608.0	.040	.040	.040
291.0	.045	.045	.035	4840.0	.040	.040	.025
765.0	.040	.040	.040	4865.0	.030	.030	.030
782.0	.040	.040	.020	4880.0	.045	.045	.035
816.0	.050	.050	.040	5405.0	.045	.045	.025
857.0	.065	.065	.055	5445.0	.045	.045	.035
900.0	.070	.070	.050	6970.0	.040	.040	.030
1575.0	.070	.070	.030	6984.0	.050	.050	.035
1590.0	.070	.070	.040	7363.0	.060	.060	.045
2954.0	.050	.050	.040	7585.0	.060	.060	.050
2965.0	.030	.030	.025	7963.0	.070	.070	.065
2980.0	.050	.050	.040	8232.0	.090	.090	.075
3008.0	.045	.045	.040	8508.0	.095	.095	.080
3694.0	.040	.040	.030	8629.0	.120	.090	.080
3698.0	.030	.030	.015	8842.0	.090	.080	.075
3702.3	.030	.030	.025	8988.0	.035	.035	.025
3722.0	.040	.040	.030	9836.0	.035	.035	.035
3764.0	.030	.030	.020	9870.0	.040	.040	.030
3883.0	.035	.035	.030	10,247.0	.045	.045	.035
3933.0	.035	.035	.025	10,582.0	.080	.080	.065
3945.0	.030	.030	.025	11,758.0	.080	.080	.025
3957.0	.030	.030	.020	11,767.0	.080	.080	.065
3957.2	.040	.040	.030	11,930.0	.080	.080	.030
3985.0	.030	.030	.030	11,942.0	.080	.080	.065
4028.0	.040	.040	.030	12,228.0	.080	.080	.025
4070.0	.040	.040	.040	12,233.0	.080	.080	.035
4158.0	.040	.040	.025	12,235.0	.080	.080	.070

* Mannings "n" for left bank (LB), right bank (RB) and channel (CH)

** values employed for this section and subsequent sections until revised

*** revised value employed for this and subsequent sections until next revision

TABLE A-4 (cont'd)

WATERFORD RIVER

MANNINGS ROUGHNESS VALUES

SECTION	LB*	RB	CH
12,312.0	.050	.050	.040**
12,323.0	.050	.050	.025***
12,330.0	.050	.050	.030
12,330.0	.050	.050	.025
12,354.0	.050	.050	.040
12,572.0	.040	.040	.035
12,593.0	.040	.040	.025
12,607.0	.040	.040	.030
12,735.0	.040	.040	.025
12,763.0	.045	.045	.035
13,117.0	.045	.045	.025
13,146.0	.045	.045	.035
13,244.0	.045	.045	.025
13,266.0	.045	.045	.035
13,470.0	.045	.045	.025
13,569.0	.045	.045	.035
12,355.0	.050	.050	.040
12,366.0	.050	.050	.020
12,384.0	.050	.050	.040
12,408.0	.050	.050	.020
12,429.0	.050	.050	.040

* Mannings "n" for left bank (LB), right bank (RB) and channel (CH)

** values employed for this section and subsequent sections until revised

*** revised value employed for this and subsequent sections until next revision

APPENDIX 5.0

FLOOD LEVEL DATA

100 YEAR EVENT

SUMMARY PRINTOUT TABLE 150

SECNO	Q	CWSEL	DIFWSP	DIFWSX	DIFKWS	TOPWID	XLCH
0.100	145.00	0.62**	0.00	0.00	0.00	32.23	0.00
0.200	145.00	0.68**	0.00	0.06	0.00	30.42	50.00
0.300	145.00	0.73**	0.00	0.05	0.00	30.47	186.55
120.000	145.00	1.43**	0.00	0.70	0.00	21.27	120.00
291.000	143.00	2.76	0.00	1.32	0.00	88.33	171.00
510.000	142.00	3.14	0.00	0.39	0.00	30.16	219.00
745.000	139.00	4.00	0.00	0.86	0.00	25.25	235.00
765.000	139.00	4.08	0.00	0.08	0.00	16.64	20.00
782.000	139.00	4.31	0.00	0.23	0.00	16.64	17.00
816.000	139.00	4.34	0.00	0.03	0.00	39.28	34.00
857.000	139.00	4.93	0.00	0.58	0.00	46.51	41.00
875.000	139.00	4.77	0.00	-0.15	0.00	21.49	18.00
900.000	139.00	5.27	0.00	0.50	0.00	22.57	25.00
928.000	139.00	5.25	0.00	-0.02	0.00	13.32	28.00
1050.000	137.50	7.25	0.00	2.01	0.00	21.73	122.00
1325.000	137.50	11.88	0.00	4.63	0.00	31.55	275.00
1555.000	137.50	13.73	0.00	1.85	0.00	47.95	230.00
1570.000	135.00	13.75	0.00	0.02	0.00	16.91	15.00
1575.000	135.00	14.29	0.00	0.54	0.00	84.54	5.00
1590.000	135.00	14.51	0.00	0.23	0.00	114.19	15.00
1895.000	134.00	14.50	0.00	-0.01	0.00	27.62	305.00
2088.000	134.00	15.60	0.00	1.10	0.00	52.59	75.00
2206.000	133.00	16.44	0.00	0.84	0.00	65.95	118.00
2391.000	130.00	17.26	0.00	0.82	0.00	55.65	195.00

20 YEAR EVENT

SUMMARY PRINTOUT TABLE 150

SECNO	Q	CWSEL	DIFWSP	DIFWSX	DIFKWS	TOPWID	XLCH
0.100	102.00	0.62**	0.00	0.00	0.00	32.23	0.00
0.200	102.00	0.65**	0.00	0.03	0.00	30.38	50.00
0.300	102.00	0.67**	0.00	0.02	0.00	30.40	186.55
120.000	102.00	1.05**	0.00	0.38	0.00	20.03	120.00
291.000	99.70	2.24	0.00	1.19	0.00	21.84	171.00
510.000	98.00	2.66	0.00	0.42	0.00	21.12	219.00
745.000	97.00	3.55	0.00	0.88	0.00	21.80	235.00
765.000	97.00	3.68	0.00	0.13	0.00	16.64	20.00
782.000	97.00	3.81	0.00	0.13	0.00	16.64	17.00
816.000	97.00	3.71	0.00	-0.09	0.00	24.95	34.00
857.000	97.00	4.41	0.00	0.67	0.00	42.95	41.00
875.000	97.00	4.29	0.00	-0.12	0.00	20.68	18.00
900.000	97.00	4.76	0.00	0.47	0.00	21.48	25.00
928.000	97.00	4.82	0.00	0.06	0.00	12.77	28.00
1050.000	96.00	6.78	0.00	1.96	0.00	19.39	122.00
1325.000	96.00	11.62	0.00	4.84	0.00	30.60	275.00
1555.000	96.00	13.29	0.00	1.67	0.00	35.32	230.00
1570.000	94.50	13.34	0.00	0.05	0.00	16.90	15.00
1575.000	94.50	13.64	0.00	0.29	0.00	16.91	5.00
1590.000	94.50	13.81	0.00	0.17	0.00	97.27	15.00
1895.000	93.80	14.16	0.00	0.35	0.00	24.65	305.00
2088.000	93.80	15.23	0.00	1.07	0.00	49.85	75.00
2206.000	93.10	16.15	0.00	0.92	0.00	62.17	118.00
2391.000	92.00	16.95	0.00	0.77	0.00	49.30	195.00

** 1:100 year and 1:20 year maximum instantaneous tide levels are 1.69 m and 1.54 m, respectively

	2491.000	130.00	17.73	0.00	0.47	0.00	56.90	100.00
	2591.000	130.00	18.14	0.00	0.41	0.00	57.09	100.00
	2682.000	127.00	19.09	0.00	0.95	0.00	43.17	291.00
*	2802.000	124.00	21.18	0.00	2.09	0.00	27.17	120.00
*	2902.000	124.00	23.64	0.00	2.45	0.00	27.25	100.00
*	2940.000	124.00	24.54	0.00	0.90	0.00	16.63	38.00
*	2954.000	124.00	25.26	0.00	0.72	0.00	15.30	14.00
	2965.000	124.00	25.32	0.00	0.07	0.00	15.30	11.00
	2980.000	124.00	25.29	0.00	-0.04	0.00	21.50	15.00
*	3008.000	121.00	25.47	0.00	0.18	0.00	14.94	28.00
	3200.000	120.00	27.66	0.00	2.19	0.00	30.46	192.00
*	3475.000	119.00	30.73	0.00	3.08	0.00	20.29	275.00
	3690.000	118.00	33.00	0.00	2.27	0.00	87.63	215.00
	3694.000	118.00	33.04	0.00	0.04	0.00	167.37	4.00
	3698.000	118.00	33.13	0.00	0.09	0.00	142.27	6.00
*	3702.300	118.00	32.82	0.00	-0.31	0.00	11.11	0.30
	3709.000	118.00	33.19	0.00	0.37	0.00	11.19	10.00
	3711.000	118.00	33.79	0.00	0.61	0.00	14.93	0.30
	3722.000	118.00	33.91	0.00	0.11	0.00	24.14	11.00
	3764.000	118.00	34.00	0.00	0.13	0.00	37.96	36.20
	3837.000	118.00	33.96	0.00	-0.04	0.00	38.13	75.00
	3883.000	118.00	34.15	0.00	0.19	0.00	85.55	49.00
	3917.000	118.00	34.08	0.00	0.07	0.00	60.47	31.90
	3917.100	118.00	34.08	0.00	0.00	0.00	60.26	0.10
	3933.000	118.00	33.88	0.00	-0.20	0.00	21.33	17.00
*	3945.000	118.00	34.01	0.00	0.13	0.00	21.17	12.00

1
24-MAR-88 09:17:10

	SECNO	Q	CWSEL	DIFWSP	DIFWSX	DIFKWS	TOPWID	XLCH
*	3957.000	118.00	34.10	0.00	0.07	0.00	20.18	12.50

	2491.000	92.00	17.41	0.00	0.46	0.00	51.09	100.00
	2591.000	92.00	17.81	0.00	0.41	0.00	51.22	100.00
	2682.000	90.00	18.71	0.00	0.90	0.00	39.41	291.00
*	2802.000	87.00	20.70	0.00	1.98	0.00	18.56	120.00
*	2902.000	87.00	23.15	0.00	2.45	0.00	18.55	100.00
*	2940.000	87.00	24.10	0.00	0.95	0.00	15.29	38.00
	2954.000	87.00	24.70	0.00	0.60	0.00	15.30	14.00
	2965.000	87.00	24.78	0.00	0.08	0.00	15.30	11.00
*	2980.000	87.00	24.69	0.00	-0.07	0.00	17.48	15.00
*	3008.000	85.00	25.00	0.00	0.31	0.00	13.14	28.00
	3200.000	84.00	27.20	0.00	2.20	0.00	26.31	192.00
*	3475.000	83.50	30.36	0.00	3.16	0.00	18.86	275.00
	3690.000	83.30	32.75	0.00	2.39	0.00	87.61	215.00
	3694.000	83.30	32.84	0.00	0.09	0.00	132.24	4.00
	3698.000	83.30	32.94	0.00	0.10	0.00	138.66	6.00
	3702.300	83.30	32.65	0.00	-0.29	0.00	11.08	0.30
	3709.000	83.30	32.76	0.00	0.11	0.00	11.10	10.00
	3711.000	83.30	33.17	0.00	0.42	0.00	14.93	0.30
	3722.000	83.30	33.21	0.00	0.04	0.00	22.58	11.00
	3764.000	83.30	33.29	0.00	0.08	0.00	35.63	36.20
*	3837.000	83.30	33.24	0.00	-0.06	0.00	32.77	75.00
*	3883.000	83.30	33.44	0.00	0.20	0.00	21.66	49.00
	3917.000	83.30	33.91	0.00	0.47	0.00	51.28	31.90
	3917.100	83.30	33.92	0.00	0.00	0.00	51.00	0.10
	3933.000	83.30	33.87	0.00	-0.05	0.00	20.92	17.00
	3945.000	83.30	33.74	0.00	-0.13	0.00	15.73	12.00

1
24-MAR-88 09:20:34

SECNO	Q	CWSEL	DIFWSP	DIFWSX	DIFKWS	TOPWID	XLCH
3957.000	83.30	33.70	0.00	-0.04	0.00	12.02	12.50

	3957.100	118.00	34.10	0.00	0.00	0.00	20.19	0.10
	3957.200	118.00	34.92	0.00	0.82	0.00	46.26	0.60
	3960.000	118.00	34.93	0.00	0.01	0.00	46.53	1.50
	3960.100	118.00	34.93	0.00	0.09	0.00	46.66	0.10
	3985.000	118.00	35.03	0.00	0.10	0.00	26.78	25.00
	4008.000	117.00	35.15	0.00	0.12	0.00	27.64	23.00
	4028.000	117.00	35.19	0.00	0.04	0.00	38.26	24.00
	4070.000	71.70	35.56	0.00	0.38	0.00	152.23	38.00
	4100.000	71.70	35.60	0.00	0.03	0.00	136.40	30.00
	4138.000	71.70	35.57	0.00	-0.02	0.00	50.98	38.00
	4158.000	71.70	35.31	0.00	0.26	0.00	9.81	20.00
	4166.000	71.70	35.36	0.00	0.04	0.00	9.81	8.00
	4176.000	71.70	35.83	0.00	0.43	0.00	28.87	10.00
*	4218.000	71.70	35.94	0.00	0.11	0.00	18.17	42.00
*	4238.000	71.70	36.38	0.00	0.44	0.00	16.19	20.00
*	4258.000	71.70	36.99	0.00	0.61	0.00	10.70	20.00
	4261.000	71.70	37.33	0.00	0.34	0.00	10.70	3.00
	4271.000	71.70	37.78	0.00	0.46	0.00	33.13	10.00
*	4356.000	69.50	39.09	0.00	1.30	0.00	13.18	85.00
*	4455.000	69.50	43.45	0.00	4.37	0.00	11.88	99.00
*	4493.000	69.50	46.52	0.00	3.07	0.00	17.54	38.00
*	4593.000	69.50	49.36	0.00	2.84	0.00	24.03	100.00
	4608.000	67.50	49.95	0.00	0.57	0.00	35.32	15.00
*	4688.000	67.50	51.12	0.00	1.17	0.00	15.69	80.00
*	4830.000	67.50	53.47	0.00	2.35	0.00	107.53	142.00

1

24-MAR-88 09:17:10

SECNO	Q	CWSEL	DIFWSP	DIFWSX	DIFKWS	TOPWID	XLCH
4840.000	67.50	53.34	0.00	0.13	0.00	14.33	10.00
4865.000	67.50	53.40	0.00	0.06	0.00	14.47	25.00

	3957.100	83.30	33.70	0.00	0.00	0.00	12.02	0.10
	3957.200	83.30	33.71	0.00	0.01	0.00	12.02	0.60
	3960.000	83.30	33.78	0.00	0.07	0.00	12.31	1.50
	3960.100	83.30	33.78	0.00	0.00	0.00	12.28	0.10
*	3985.000	83.30	33.97	0.00	0.19	0.00	17.02	25.00
	4008.000	82.70	34.61	0.00	0.64	0.00	23.84	23.00
	4028.000	82.70	34.69	0.00	0.08	0.00	16.48	24.00
	4070.000	52.20	35.00	0.00	0.31	0.00	59.70	38.00
	4100.000	52.20	35.07	0.00	0.07	0.00	100.15	30.00
	4138.000	52.20	35.05	0.00	-0.02	0.00	40.65	38.00
	4158.000	52.20	34.83	0.00	-0.22	0.00	9.81	20.00
	4166.000	52.20	34.93	0.00	0.11	0.00	9.81	8.00
	4176.000	52.20	35.24	0.00	0.31	0.00	22.46	10.00
*	4218.000	52.20	35.70	0.00	0.46	0.00	17.28	42.00
*	4238.000	52.20	36.11	0.00	0.40	0.00	15.10	20.00
*	4258.000	52.20	36.67	0.00	0.57	0.00	10.70	20.00
	4261.000	52.20	36.97	0.00	0.30	0.00	10.70	3.00
	4271.000	52.20	37.22	0.00	0.25	0.00	20.55	10.00
*	4356.000	51.00	38.80	0.00	1.58	0.00	12.69	85.00
*	4455.000	51.00	43.16	0.00	4.34	0.00	11.58	99.00
*	4493.000	51.00	46.26	0.00	3.11	0.00	15.57	38.00
*	4593.000	51.00	48.92	0.00	2.66	0.00	12.58	100.00
	4608.000	49.00	49.60	0.00	0.68	0.00	31.35	15.00
*	4688.000	49.00	50.78	0.00	1.18	0.00	12.95	80.00
*	4830.000	49.00	52.97	0.00	2.19	0.00	13.54	142.00

1

24-MAR-88 09:20:34

SECNO	Q	CWSEL	DIFWSP	DIFWSX	DIFKWS	TOPWID	XLCH
4840.000	49.00	53.29	0.00	0.32	0.00	14.21	10.00
4865.000	49.00	53.47	0.00	0.18	0.00	14.63	25.00

*	4880.000	67.50	53.54	0.00	0.15	0.00	21.27	15.00
	5135.000	65.50	55.26	0.00	1.72	0.00	59.32	265.00
*	5235.000	65.50	55.61	0.00	0.35	0.00	47.89	100.00
	5340.000	65.50	56.28	0.00	0.67	0.00	59.65	105.00
*	5390.000	65.50	56.54	0.00	0.27	0.00	16.85	50.00
*	5405.000	64.50	56.74	0.00	0.20	0.00	17.55	15.00
	5425.000	64.50	57.77	0.00	1.03	0.00	57.04	20.00
	5445.000	64.50	57.90	0.00	0.12	0.00	63.42	20.00
*	5615.000	64.50	58.05	0.00	0.15	0.00	58.86	170.00
*	5895.000	63.00	59.96	0.00	1.91	0.00	15.03	280.00
	6200.000	63.00	63.00	0.00	3.04	0.00	22.35	305.00
	6450.000	61.40	64.29	0.00	1.29	0.00	21.73	250.00
	6736.000	61.40	65.50	0.00	1.20	0.00	17.88	286.00
*	6910.000	61.40	75.60	0.00	10.10	0.00	23.60	460.00
	6940.000	61.40	76.02	0.00	0.42	0.00	18.36	30.00
	6955.000	52.40	76.04	0.00	0.01	0.00	10.63	15.00
	6970.000	52.40	75.97	0.00	-0.07	0.00	8.30	15.00
*	6984.000	49.00	76.73	0.00	0.75	0.00	13.61	14.00
*	7070.000	44.20	78.91	0.00	2.18	0.00	57.29	86.00
*	7193.000	44.20	85.56	0.00	6.65	0.00	17.97	123.00
	7363.000	44.20	87.69	0.00	2.13	0.00	18.59	170.00
*	7585.000	42.20	95.61	0.00	7.92	0.00	23.11	222.00
*	7697.000	42.20	100.72	0.00	5.11	0.00	50.22	112.00
	7947.000	40.30	102.77	0.00	2.05	0.00	47.72	248.00

1
24-MAR-88 09:17:10

SECNO	Q	CWSEL	DIFWSP	DIFWSX	DIFKWS	TOPWID	XLCH
7963.000	40.30	102.85	0.00	0.07	0.00	56.22	15.00
8022.000	40.30	102.91	0.00	0.06	0.00	210.30	61.00
8120.000	40.30	102.92	0.00	0.01	0.00	177.17	101.00

	4880.000	49.00	53.48	0.00	0.01	0.00	20.57	15.00
	5135.000	48.00	54.99	0.00	1.51	0.00	46.50	265.00
*	5235.000	48.00	55.46	0.00	0.47	0.00	36.21	100.00
	5340.000	48.00	56.07	0.00	0.61	0.00	30.89	105.00
*	5390.000	48.00	56.32	0.00	0.25	0.00	15.64	50.00
*	5405.000	47.00	56.46	0.00	0.15	0.00	15.69	15.00
*	5425.000	47.00	56.56	0.00	0.10	0.00	13.57	20.00
	5445.000	47.00	57.22	0.00	0.66	0.00	26.58	20.00
*	5615.000	47.00	57.69	0.00	0.47	0.00	21.55	170.00
	5895.000	45.00	59.88	0.00	2.19	0.00	14.88	280.00
*	6200.000	45.00	62.63	0.00	2.74	0.00	19.63	305.00
	6450.000	44.90	64.06	0.00	1.43	0.00	20.45	250.00
	6736.000	44.90	65.21	0.00	1.16	0.00	17.14	286.00
*	6910.000	44.90	75.40	0.00	10.15	0.00	21.97	460.00
	6940.000	44.90	75.80	0.00	0.40	0.00	16.65	30.00
	6955.000	38.30	75.83	0.00	0.02	0.00	10.18	15.00
	6970.000	38.30	75.79	0.00	-0.04	0.00	8.30	15.00
*	6984.000	37.00	76.51	0.00	0.72	0.00	12.59	14.00
*	7070.000	30.70	78.82	0.00	2.31	0.00	56.13	86.00
*	7193.000	30.70	85.29	0.00	6.47	0.00	12.68	123.00
	7363.000	30.70	87.45	0.00	2.16	0.00	17.66	170.00
*	7585.000	30.70	94.99	0.00	7.54	0.00	6.33	222.00
*	7697.000	30.70	100.64	0.00	5.64	0.00	44.39	112.00
	7947.000	29.20	102.58	0.00	1.94	0.00	33.66	248.00

1
24-MAR-88 09:20:34

SECNO	Q	CWSEL	DIFWSP	DIFWSX	DIFKWS	TOPWID	XLCH
7963.000	29.20	102.64	0.00	0.06	0.00	33.73	15.00
8022.000	29.20	102.69	0.00	0.05	0.00	178.03	61.00
8120.000	29.20	102.70	0.00	0.01	0.00	165.70	101.00

8232.000	40.30	102.95	0.00	0.03	0.00	149.13	100.00
8357.000	40.30	103.00	0.00	0.06	0.00	177.37	132.00
8508.000	40.30	103.08	0.00	0.08	0.00	144.83	144.00
8606.000	40.30	103.17	0.00	0.09	0.00	157.83	105.00
8629.000	40.30	103.22	0.00	0.05	0.00	116.40	68.00
8776.000	40.30	103.48	0.00	0.25	0.00	141.72	133.00
8842.000	40.30	103.75	0.00	0.26	0.00	174.93	66.00
8896.000	39.50	103.95	0.00	0.21	0.00	137.80	53.00
8988.000	39.50	104.43	0.00	0.48	0.00	105.54	91.00
9048.000	39.50	104.66	0.00	0.22	0.00	122.12	59.00
* 9260.000	39.50	105.47	0.00	0.81	0.00	51.57	196.00
* 9415.000	37.00	106.44	0.00	0.97	0.00	30.65	160.00
* 9645.000	37.00	108.99	0.00	2.55	0.00	51.33	230.00
* 9836.000	37.00	110.81	0.00	1.82	0.00	11.65	191.00
* 9851.000	37.00	111.84	0.00	1.03	0.00	4.70	15.00
* 9870.000	37.00	111.95	0.00	0.10	0.00	4.70	19.00
9876.000	36.20	113.43	0.00	1.49	0.00	98.36	6.00
9885.000	36.20	113.43	0.00	0.00	0.00	98.30	9.00
10030.000	35.50	113.40	0.00	0.03	0.00	33.22	145.00
* 10247.000	33.00	113.98	0.00	0.58	0.00	23.58	217.00
10582.000	31.00	122.77	0.00	8.80	0.00	14.13	595.00
10840.000	29.00	124.80	0.00	2.03	0.00	51.17	258.00
11060.000	27.50	126.03	0.00	1.23	0.00	29.62	210.00

1
24-MAR-88 09:17:10

SECNO	Q	CWSEL	DIFWSP	DIFWSX	DIFKWS	TOPWID	XLCH
11340.000	25.50	129.14	0.00	3.11	0.00	67.03	290.00
11480.000	25.50	130.51	0.00	1.36	0.00	21.63	140.00
11650.000	23.60	132.93	0.00	2.43	0.00	5.25	170.00
11758.000	22.80	133.41	0.00	0.48	0.00	11.13	90.00

8232.000	29.20	102.73	0.00	0.03	0.00	136.60	100.00
8357.000	29.20	102.79	0.00	0.06	0.00	151.91	132.00
8508.000	29.20	102.87	0.00	0.08	0.00	126.23	144.00
8606.000	29.20	102.96	0.00	0.09	0.00	144.42	105.00
8629.000	29.20	103.02	0.00	0.06	0.00	115.33	68.00
8776.000	29.20	103.30	0.00	0.29	0.00	137.90	133.00
8842.000	29.20	103.63	0.00	0.32	0.00	166.48	66.00
8896.000	28.50	103.85	0.00	0.22	0.00	113.10	53.00
* 8988.000	28.50	104.39	0.00	0.54	0.00	100.40	91.00
9048.000	28.50	104.57	0.00	0.18	0.00	119.75	59.00
* 9260.000	28.50	105.38	0.00	0.81	0.00	50.82	196.00
* 9415.000	27.00	106.32	0.00	0.94	0.00	29.97	160.00
* 9645.000	27.00	108.80	0.00	2.48	0.00	35.48	230.00
* 9836.000	27.00	110.61	0.00	1.81	0.00	11.30	191.00
* 9851.000	27.00	111.49	0.00	0.88	0.00	4.70	15.00
9870.000	27.00	111.68	0.00	0.19	0.00	4.70	19.00
9876.000	26.20	112.66	0.00	0.98	0.00	92.65	6.00
9885.000	26.20	112.66	0.00	0.00	0.00	92.60	9.00
10030.000	25.50	112.60	0.00	0.06	0.00	17.15	145.00
* 10247.000	24.00	113.83	0.00	1.22	0.00	20.95	217.00
10582.000	22.00	122.57	0.00	8.74	0.00	12.87	595.00
10840.000	20.00	124.60	0.00	2.04	0.00	49.19	258.00
11060.000	19.00	125.90	0.00	1.29	0.00	27.99	210.00

1
24-MAR-88 09:20:34

SECNO	Q	CWSEL	DIFWSP	DIFWSX	DIFKWS	TOPWID	XLCH
11340.000	17.50	128.99	0.00	3.09	0.00	61.91	290.00
11480.000	17.50	130.38	0.00	1.39	0.00	19.66	140.00
11650.000	17.00	132.46	0.00	2.08	0.00	5.03	170.00
11758.000	16.90	133.07	0.00	0.61	0.00	8.08	90.00

* 11759.000	22.80	133.87	0.00	0.45	0.00	3.36	1.00
11762.000	22.80	134.63	0.00	0.77	0.00	23.61	3.40
11767.000	22.80	134.89	0.00	0.25	0.00	28.75	0.50
11774.000	22.80	134.92	0.00	0.03	0.00	30.73	7.00
11873.000	22.80	135.05	0.00	0.14	0.00	31.90	99.00
11918.000	22.80	135.12	0.00	0.07	0.00	63.21	46.00
11924.000	22.80	135.09	0.00	-0.03	0.00	28.87	7.00
11930.000	22.80	135.05	0.00	-0.04	0.00	5.08	0.60
11933.000	22.80	135.20	0.00	0.15	0.00	5.08	3.35
11938.000	22.80	135.22	0.00	0.02	0.00	13.72	0.50
11942.000	22.80	135.34	0.00	0.12	0.00	79.21	6.00
12045.000	22.80	135.35	0.00	0.01	0.00	65.48	96.00
12124.000	15.60	135.37	0.00	0.03	0.00	82.29	83.00
12200.000	15.60	135.38	0.00	0.01	0.00	52.25	73.00
12227.000	15.60	135.39	0.00	0.03	0.00	45.26	20.00
12228.000	15.60	135.38	0.00	-0.01	0.00	47.09	0.10
12233.000	15.60	135.38	0.00	0.00	0.00	46.87	6.23
12233.000	15.60	135.37	0.00	-0.01	0.00	46.21	0.10
12234.000	15.60	135.37	0.00	0.00	0.00	45.95	0.10
12235.000	15.60	135.41	0.00	0.04	0.00	46.70	0.40
12244.000	15.60	135.43	0.00	0.02	0.00	56.28	7.00
12302.000	15.60	135.57	0.00	0.14	0.00	16.68	55.00

1
24-MAR-88 09:17:10

SECNO	Q	CWSEL	DIFWSP	DIFWSX	DIFKWS	TOPWID	XLCH
12312.000	6.43	135.63	0.00	0.06	0.00	5.45	15.00
12323.000	6.43	135.60	0.00	-0.03	0.00	2.45	11.00
12323.000	6.43	135.61	0.00	0.01	0.00	2.45	0.30
12327.000	6.43	135.71	0.00	0.13	0.00	2.46	3.70
12330.000	6.43	135.89	0.00	0.15	0.00	3.76	3.70

* 11759.000	16.90	133.56	0.00	0.49	0.00	3.33	1.00
11762.000	16.90	133.94	0.00	0.38	0.00	3.37	3.40
11767.000	16.90	134.34	0.00	0.40	0.00	21.14	0.50
11774.000	16.90	134.39	0.00	0.05	0.00	26.19	7.00
11873.000	16.90	134.66	0.00	0.26	0.00	26.54	99.00
11918.000	16.90	134.77	0.00	0.11	0.00	57.67	46.00
11924.000	16.90	134.74	0.00	-0.03	0.00	20.04	7.00
11930.000	16.90	134.72	0.00	-0.02	0.00	5.08	0.60
11933.000	16.90	134.72	0.00	0.00	0.00	5.08	3.35
11938.000	16.90	134.72	0.00	0.00	0.00	7.81	0.50
11942.000	16.90	134.83	0.00	0.12	0.00	71.16	6.00
12045.000	16.90	134.84	0.00	0.00	0.00	49.89	96.00
12124.000	11.70	134.90	0.00	0.06	0.00	76.37	83.00
12200.000	11.70	134.92	0.00	0.02	0.00	50.42	73.00
12227.000	11.70	134.94	0.00	0.01	0.00	37.79	20.00
12228.000	11.70	134.92	0.00	-0.01	0.00	38.29	0.10
* 12233.000	11.70	134.89	0.00	-0.03	0.00	37.28	6.23
* 12233.000	11.70	135.15	0.00	0.26	0.00	39.41	0.10
12234.000	11.70	135.22	0.00	0.07	0.00	41.40	0.10
12235.000	11.70	135.28	0.00	0.06	0.00	43.46	0.40
12244.000	11.70	135.31	0.00	0.02	0.00	52.59	7.00
12302.000	11.70	135.44	0.00	0.13	0.00	15.38	55.00

1
24-MAR-88 09:20:34

SECNO	Q	CWSEL	DIFWSP	DIFWSX	DIFKWS	TOPWID	XLCH
12312.000	5.53	135.48	0.00	0.04	0.00	4.88	15.00
12323.000	5.53	135.49	0.00	0.01	0.00	2.44	11.00
12323.000	5.53	135.50	0.00	0.01	0.00	2.44	0.30
12327.000	5.53	135.61	0.00	0.11	0.00	2.45	3.70
12330.000	5.53	135.76	0.00	0.15	0.00	3.74	3.70

12330.000	6.43	135.89	0.00	0.00	0.00	3.76	0.30
12338.000	6.43	135.91	0.00	0.02	0.00	3.76	7.40
12354.000	6.43	136.05	0.00	0.13	0.00	33.88	16.00
* 12458.000	6.43	137.78	0.00	1.74	0.00	4.39	104.00
12572.000	6.43	139.71	0.00	1.93	0.00	13.28	114.00
12593.000	6.43	139.91	0.00	0.19	0.00	6.60	21.00
* 12593.000	6.43	139.97	0.00	0.06	0.00	8.87	0.30
12600.000	6.43	140.39	0.00	0.43	0.00	13.34	7.00
12607.000	6.43	140.58	0.00	0.19	0.00	6.60	7.00
12610.000	6.43	140.83	0.00	0.25	0.00	19.35	3.00
12615.000	6.43	140.79	0.00	0.04	0.00	9.26	5.00
* 12717.000	6.43	141.21	0.00	0.42	0.00	7.17	102.00
12735.000	6.43	141.43	0.00	0.22	0.00	5.60	18.00
12735.000	6.43	141.39	0.00	0.04	0.00	9.48	0.30
12747.000	6.43	141.51	0.00	0.12	0.00	10.50	12.00
12763.000	6.43	141.63	0.00	0.12	0.00	6.98	16.00
13002.000	6.43	143.37	0.00	1.74	0.00	8.41	239.00
13090.000	6.43	144.13	0.00	0.76	0.00	7.34	88.00
13117.000	6.43	144.34	0.00	0.21	0.00	7.94	27.00
13117.000	6.43	144.27	0.00	0.08	0.00	9.13	0.30
13140.000	6.43	144.65	0.00	0.39	0.00	10.65	23.00

1

24-MAR-88 09:17:10

SECNO	Q	CWSEL	DIFWSP	DIFWSX	DIFKWS	TOPWID	XLCH
* 13146.000	6.43	144.90	0.00	0.24	0.00	10.39	6.00
13200.000	6.43	145.39	0.00	0.49	0.00	39.06	54.00
* 13229.000	6.43	145.95	0.00	0.56	0.00	6.68	29.00
13244.000	6.43	146.22	0.00	0.28	0.00	8.41	15.00
13244.000	6.43	146.08	0.00	0.14	0.00	8.11	0.30
13255.000	6.43	146.51	0.00	0.42	0.00	32.30	11.00

12330.000	5.53	135.76	0.00	0.00	0.00	3.74	0.30
12338.000	5.53	135.79	0.00	0.03	0.00	3.74	7.40
12354.000	5.53	135.91	0.00	0.13	0.00	30.51	16.00
* 12458.000	5.53	137.72	0.00	1.80	0.00	4.20	104.00
12572.000	5.53	139.68	0.00	1.96	0.00	12.57	114.00
12593.000	5.53	139.88	0.00	0.20	0.00	6.60	21.00
* 12593.000	5.53	139.91	0.00	0.03	0.00	8.65	0.30
12600.000	5.53	140.29	0.00	0.37	0.00	12.15	7.00
12607.000	5.53	140.43	0.00	0.15	0.00	6.60	7.00
12610.000	5.53	140.62	0.00	0.18	0.00	16.16	3.00
12615.000	5.53	140.51	0.00	0.11	0.00	7.56	5.00
12717.000	5.53	141.29	0.00	0.79	0.00	7.70	102.00
12735.000	5.53	141.39	0.00	0.10	0.00	5.60	18.00
12735.000	5.53	141.37	0.00	0.03	0.00	9.34	0.30
12747.000	5.53	141.46	0.00	0.09	0.00	9.78	12.00
12763.000	5.53	141.54	0.00	0.09	0.00	6.62	16.00
13002.000	5.53	143.32	0.00	1.78	0.00	7.83	239.00
13090.000	5.53	144.06	0.00	0.74	0.00	6.65	88.00
13117.000	5.53	144.28	0.00	0.22	0.00	7.85	27.00
13117.000	5.53	144.22	0.00	0.06	0.00	8.93	0.30
13140.000	5.53	144.54	0.00	0.32	0.00	10.18	23.00

1

24-MAR-88 09:20:34

SECNO	Q	CWSEL	DIFWSP	DIFWSX	DIFKWS	TOPWID	XLCH
* 13146.000	5.53	144.86	0.00	0.32	0.00	9.76	6.00
13200.000	5.53	145.36	0.00	0.59	0.00	37.52	54.00
* 13229.000	5.53	145.90	0.00	0.54	0.00	6.41	29.00
13244.000	5.53	146.16	0.00	0.26	0.00	8.27	15.00
13244.000	5.53	146.05	0.00	0.11	0.00	8.03	0.30
13255.000	5.53	146.40	0.00	0.34	0.00	8.84	11.00

	13266.000	6.43	146.73	0.00	0.22	0.00	7.46	11.00
*	13330.000	6.43	147.35	0.00	0.62	0.00	10.00	64.00
*	13445.000	6.43	150.39	0.00	3.04	0.00	5.53	115.00
*	13470.000	6.43	151.80	0.00	1.41	0.00	6.45	25.00
*	13470.000	6.43	151.86	0.00	0.06	0.00	8.82	0.30
	13550.000	6.43	153.09	0.00	1.23	0.00	47.22	80.00
	13569.000	6.43	153.16	0.00	0.07	0.00	8.95	19.00
	13579.000	6.43	153.14	0.00	-0.02	0.00	4.10	10.00
	13579.000	6.43	153.17	0.00	0.02	0.00	8.30	0.30
	13597.000	6.43	153.17	0.00	0.00	0.00	8.32	18.00
*	13607.000	6.43	153.31	0.00	0.14	0.00	5.55	10.00
*	13629.000	6.43	154.02	0.00	0.71	0.00	8.95	22.00
*	13720.000	6.43	156.29	0.00	2.27	0.00	6.16	91.00
	12302.000	15.60	135.57	0.00	-20.72	0.00	16.68	55.00
*	12355.000	9.13	135.73	0.00	0.16	0.00	5.57	50.00
	12365.000	9.13	135.89	0.00	0.16	0.00	2.40	10.00
	12366.000	9.13	135.88	0.00	-0.01	0.00	5.63	0.10
	12369.000	9.13	135.90	0.00	0.01	0.00	5.65	3.70
	12384.000	9.13	136.19	0.00	0.29	0.00	23.47	15.00
*	12408.000	9.13	136.11	0.00	-0.07	0.00	6.90	23.00

1
24-MAR-88 09:17:10

	SECNO	Q	CWSEL	DIFWSP	DIFWSX	DIFKWS	TOPWID	XLCH
*	12418.000	9.13	137.75	0.00	1.64	0.00	6.90	10.00
	12429.000	9.13	137.77	0.00	0.02	0.00	64.94	11.00
	12570.000	9.13	137.77	0.00	0.00	0.00	72.83	41.00

	13266.000	5.53	146.56	0.00	0.17	0.00	6.74	11.00
*	13330.000	5.53	147.31	0.00	0.75	0.00	9.47	64.00
*	13445.000	5.53	150.32	0.00	3.01	0.00	5.07	115.00
*	13470.000	5.53	151.76	0.00	1.43	0.00	6.45	25.00
*	13470.000	5.53	151.81	0.00	0.06	0.00	8.54	0.30
	13550.000	5.53	152.93	0.00	1.12	0.00	20.54	80.00
	13569.000	5.53	153.04	0.00	0.11	0.00	8.57	19.00
	13579.000	5.53	153.03	0.00	-0.01	0.00	4.10	10.00
	13579.000	5.53	153.05	0.00	0.02	0.00	7.97	0.30
	13597.000	5.53	153.05	0.00	0.00	0.00	8.02	18.00
*	13607.000	5.53	153.26	0.00	0.21	0.00	5.33	10.00
*	13629.000	5.53	153.98	0.00	0.72	0.00	8.45	22.00
*	13720.000	5.53	156.24	0.00	2.25	0.00	5.90	91.00
	12302.000	11.70	135.44	0.00	-20.80	0.00	15.38	55.00
*	12355.000	6.13	135.56	0.00	0.12	0.00	4.99	50.00
	12365.000	6.13	135.76	0.00	0.20	0.00	2.40	10.00
	12366.000	6.13	135.76	0.00	0.00	0.00	5.23	0.10
	12369.000	6.13	135.76	0.00	0.01	0.00	5.24	3.70
	12384.000	6.13	135.91	0.00	0.15	0.00	19.99	15.00
*	12408.000	6.13	135.99	0.00	0.07	0.00	6.87	23.00

1
24-MAR-88 09:20:34

	SECNO	Q	CWSEL	DIFWSP	DIFWSX	DIFKWS	TOPWID	XLCH
*	12418.000	6.13	136.71	0.00	0.72	0.00	6.90	10.00
	12429.000	6.13	136.72	0.00	0.02	0.00	17.49	11.00
	12570.000	6.13	136.75	0.00	0.03	0.00	12.95	41.00