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Canada-Newfoundland Flood Damage Reduction Program
Government of Newfoundland and Labrador
c/o Department of Environment and Lands
Confederation Complex
4th Floor West Block
P.O. Box 8700
St. John's, Newfoundland
AIB 4J6

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

Re: Codroy Valley - Flood Risk Mapping Study

Dear Dr. Ullah:

We take pleasure in submitting the enclosed report and Technical Appendix on the above-titled study. The most up-to-date procedures were employed in determining the hydrologic and hydraulic regime of the Codroy Valley area, and we anticipate that the enclosed will serve the Program well in the coming years.

The main report initially provides details about the physiography of the watershed and the hydrologic modelling of the 20 and 100 year return period flood flows. These flows are then evaluated through hydraulic analysis to determine flood levels and the effect of ice accumulations. The last section of the report describes the flood-prone areas and flood damage reduction alternatives. A Technical Appendix (under separate cover) provides additional particulars about the study for use by your technical specialists.

It has been a distinct pleasure working on this project with members of the Flood Damage Reduction Program team. We particularly wish to thank you, Mr. Picco and Ms. Langley for the considerable assistance and constructive comments provided throughout the course of this study.

Yours very truly,

FENCO NEWFOUNDLAND LTD.

Clarence Hewitt, P.Eng.
Project Director

Douglas B. Hodgins, P.Eng.
Project Manager

Lavalin

Flood Risk Mapping Study
Codroy Valley Area

for

Canada-Newfoundland
Flood Damage Reduction Program

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1.0 SUMMARY OF FINDINGS, CONCLUSIONS AND RECOMMENDATIONS

1.1 Introduction

The Codroy Valley area in southwestern Newfoundland has a broad and relatively flat floodplain which is prone to flooding. A number of bends, shallows and islands in the river also cause the formation of ice accumulations and result in additional flooding. These accumulations frequently raise flood levels near the Trans-Canada Highway crossing near Coal Brook, but they also contribute to flooding along the complete length of the river and are partially responsible for the destruction of the Grand Bay Bridge in 1978. Direct and indirect damages from that single event exceed $6 million.

In view of the potential for loss of life and damages resulting from floods and ice problems, the Province of Newfoundland and the Government of Canada have entered into an agreement to identify and delineate flood hazard areas. This study is part of that initiative and provides the necessary information for the identification of flood prone lands.

Since initiation of the project, considerable effort has been focused on obtaining high quality field data along more than 29 km of the river. This data was then combined with those from many other local, provincial and federal sources to provide accurate information for the analysis of past flooding problems. This in turn enabled projection of the 1:20 year and 1:100 year return period flood levels and preliminary identification of flood damage reduction alternatives.

1.2 Summary

1. Review of historical flooding problems (Chapter 3.0) confirms that high water levels can result from either snowmelt and rainfall in the winter and early spring, heavy rainfall alone, or ice accumulations during flows which accompany spring break-up.

2. There are sufficient hydrometric and meteorological data near the study area (Chapter 4.0) to determine flood flows by computer simulation using an advanced hydrologic model called QUALHYMO. There are also soils, land use and topographic data to permit these flows to be estimated for different tributary areas, and combined as the tributaries enter the main river.

3. The conditions which lead to flooding have been simulated with the hydrologic model for four high flow periods in order to ensure that it was calibrated and verified to replicate known flood flows (Chapter 5.0). The verification showed close agreement between observed and simulated discharges for a range of flood peaks and gave confidence in the model's capability to simulate runoff in the study area.

4. High flows for the 26 years from 1962 were simulated in the model to establish flows at various points of interest along the river course. This analysis confirmed that winter and spring months are the most likely to have high flows, but that summer and fall storms may also result in flooding.

5. Frequency analyses of the annual series of simulated high flows were conducted to give estimates of the 1:20 and 1:100 year flood flows along the river. Another set of estimates was developed by simulating the runoff from the 1:20 and 1:100 year storm rainfalls, and two other sets were determined by use of regional flood frequency analysis prepared by the Canada-Newfoundland Flood Damage Reduction Program and the Department of Environment and Lands. All four estimates were found to provide similar flood flows.

6. The selected flood flows for the 1:20 and 1:100 year return periods at several locations in the Codroy Valley are summarized below:

Return Period and Flood Flow

Location	1:20 Year	1:100 Year
North Branch	471 m^3/s	541 m^3/s
South Branch	451	578
Confluence	851	1045
Upper Ferry	929	1098

7. Information to determine the water levels developed by these flows was obtained by field surveys, local observers, historical records and records of the Marine Environmental Data Service. Analysis of the latter information determined the following levels at the river mouth (Chapter 6.0):

Return Period Flood Level

Location	1:20 Year	1:100 Year
Upper Ferry	1.72 m	1.84 m

8. Analysis of the effects of ice accumulations, bridge constrictions, islands and other factors influencing flood levels was conducted with the HEC-2 backwater model. The model was calibrated and verified, and it was determined that physical data from 32 cross-sections could be used to determine flood levels in the study area (Chapter 7.0).

9. Separate analyses of the effect of tidal flows and ice accumulations concluded that:

- it is appropriate to add an ebb tide component (70 m^3/s) to anticipated 1:20 and 1:100 year flood flows for determining flood levels near the mouth

- ice effect flood levels are realistically accounted for in the backwater area defined by open water flood flows

10. The water surface profiles for the 1:20 and 1:100 year return period floods were delineated on 1:2,500 scale topographic mapping (Chapter 7.0). This work determined that there are currently seven structures within the flood-prone area of the Grand Codroy River. Three

are located near the Trans-Canada Highway crossing northeast of Coal Brook, and four are in the Coal Brook area. Although this number is now limited, there are many other flood-prone areas which might otherwise be considered desirable riverside locations for cottages or farm buildings.

11. As there are these developable, flood-prone areas in the Codroy Valley, it is recommended that the flood elevations advanced herein (Chapter 8.0) be adopted by the Municipality so that flood-prone areas can be zoned in the near future for special attention, restrictions or design consideration (e.g., flood proofing by elevation).

12. In order to alleviate future flood damage at those structures which are now in the flood prone area, it is recommended that consideration be given to flood proofing or relocating these buildings. There are options and alternatives for each structure and these are discussed in Chapter 9.0.

2.0 INTRODUCTION

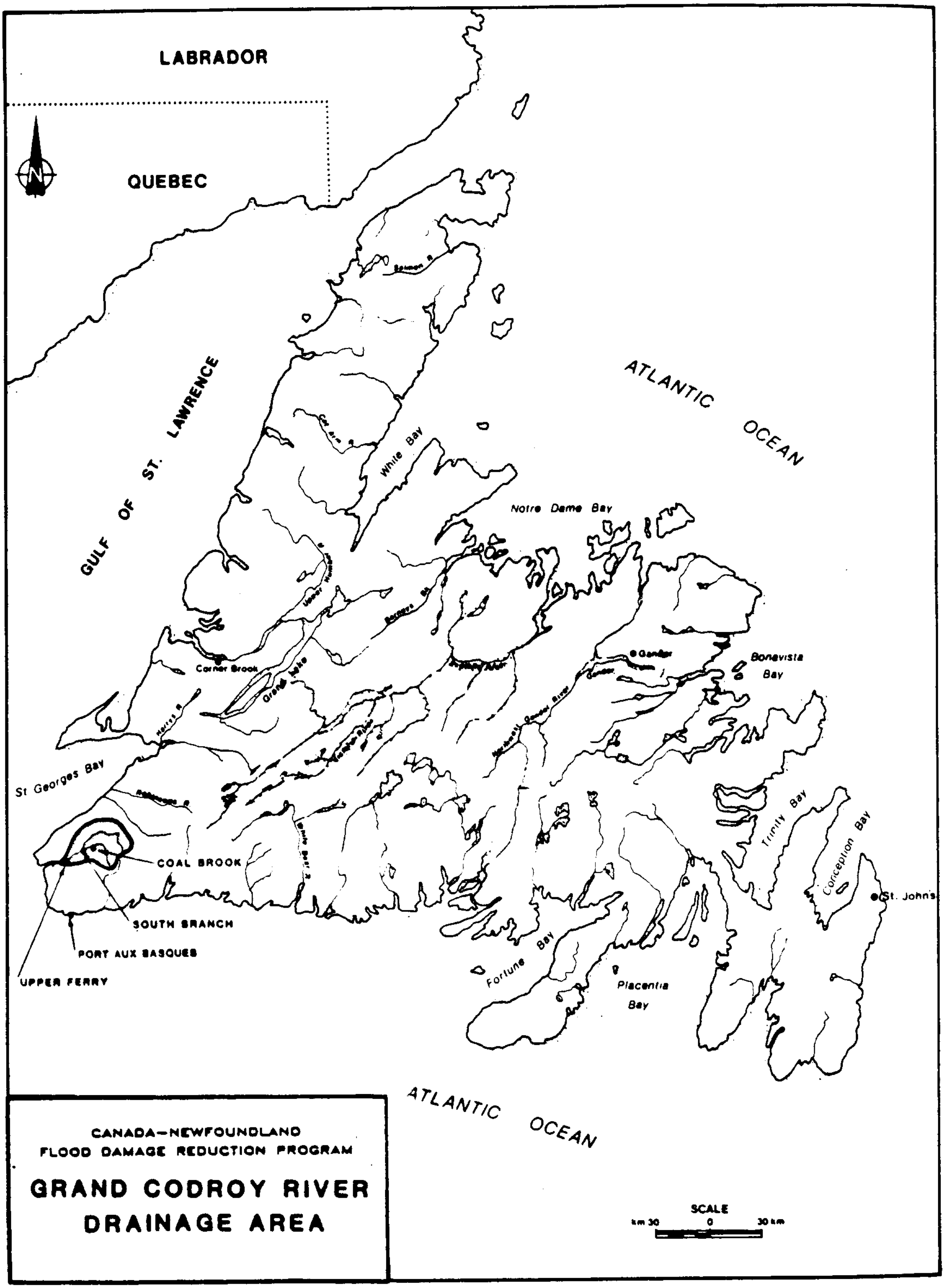
2.1 General

The Codroy Valley is located in the southwestern corner of the Province of Newfoundland, approximately 20 km north of Port aux Basques. Figure 2-1 outlines the drainage area of the Grand Codroy River and its main tributaries upstream of its mouth. The principal communities along the river include Upper Ferry and Great Codroy, near the river mouth (Figure 2-2), and the hamlets of South Branch and Coal Brook. These are further upstream near the confluence of the North Branch and South Branch tributaries of the Grand Codroy River (Figure 2-3).

The Codroy Valley is broad and flat along much of its length near the river, and as a result it is subject to flooding. The river is also prone to break-up ice jams which regularly increase flood levels at the Trans-Canada Highway Bridge near Coal Brook and a number of locations further downstream. One of these locations was the Grand Bay Bridge, Upper Ferry, where ice and high flows combined in January 1979 to destroy the bridge and isolate about 1,500 residents. Direct and indirect damages from this flood exceeded $6 million.

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 flood plains along the shores of lakes, rivers and the sea. This Agreement also recognizes that the potential for flood damages can be reduced by control of 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 land in the Codroy Valley area - specifically near Upper Ferry, and between the hamlet of South Branch and the Trans-Canada Highway bridge upstream.

2.2 Study Objectives



CANADA–NEWFOUNDLAND
FLOOD DAMAGE REDUCTION PROGRAM

GRAND CODROY RIVER DRAINAGE AREA

FIGURE 2-1

2.2 Study Objectives

The objectives of the study are essentially four-fold, and are well summarized in the Terms of Reference:

(1) To identify and assess the potential for ice-related problems in the Codroy Valley study area, shown in Figures 2-2 and 2-3.

(2) To provide estimates of the 1:20 and 1:100 year recurrence interval flood levels and the extent of flooding associated with each. Flooding in the area is reported to be the result of elevated water levels caused by ice constrictions and high river flows.

(3) To determine the extent of flooding associated with these ice-related or high flow conditions and place these levels on large scale topographic maps (1:2,500 scale with 1 m contours) to be provided by the Technical Committee of the Flood Damage Reduction Program.

(4) To suggest, for possible future studies and action, suitable remedial and preventive measures to alleviate potential flood damage problems in the future.

The study consisted of: a thorough review of existing information, the selection of appropriate mathematical models to simulate runoff of river hydraulics, the identification of data voids, the collection and compilation of new data necessary for the study, the analysis and interpretation of the data, and derivation of the 1:20 and 1:100 year flood profiles.

This volume (Volume 1) presents all of the major findings of the study and examples of mapping of the flood prone areas in the Codroy Valley. Volume 2, which is a compendium of technical notes, drawings, field data and computer programs, is available to 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 Historical Flood Occasions

The history of flooding and high flows on the Grand Codroy River is one which has gone largely unrecorded. Fortunately, several riverside residents recall the dates and approximate levels of recent floods, and general flood data is available through the report "Flooding Events in Newfoundland and Labrador - An Historical Perspective" (Kindervater, 1980).

It is evident from local discussions and the above report that there is growing interest and concern about flood effects in the Codroy Valley Area, most likely a direct result of destruction of the Grand Bay Bridge during the 1978 ice break-up during high flows. Since that time, there have been four reports of ice-related floods as compared to eight reports in the previous 30 years.

Table 3-1 summarizes the known and possible occasions of high flows, flooding and ice problems in the Codroy Valley area. It shows that flooding problems generally result from snowmelt and rainfall in the winter and early spring, although heavy rainfall alone appears to have caused flooding in November, 1949 and December, 1972.

The Table also notes the frequency of ice jams as a contributor or cause of flooding. It is certain that ice jams at the Trans-Canada Highway (TCH) bridge near Coal Brook have caused flooding in that area, and that break-up ice was a major factor in the loss of the Grand Bay Bridge in 1978.

In view of these historical problems with ice and high flows, and the possibility of damaging floods in the future, it is appropriate that the Technical Committee of the Canada-Newfoundland Flood Damage Reduction Program has initiated this flood risk mapping study of the Codroy Valley area.

TABLE 3.1

SUMMARY OF HISTORICAL FLOODING
AND POSSIBLE HIGH FLOW PERIODS

Date	Description of Known of Possible Flood Conditions in the Study Area
1948	temporary ice jam at South Branch railway bridge[2]
1949 Nov 26 - 27	heavy rains cause rail service disruption/washout at South Branch[1]. The Bridge was subsequently replaced by a wider span[2]
1952 Feb and 1955 Feb	possible minor flooding as surrounding regions reported washouts and minor flooding
1969 Mar 19 - 21	railway line reported to be damaged at Codroy Pond due to rainfall and snowmelt, but no damage estimates are recorded[3]
1971 Feb 14-15	areas around the Codroy Valley were affected by rainfall/ snowmelt and ice jams[1] but do not appear to have affected the study area
1972 Dec 9 - 10	heavy rainfall caused a flash flood on the South Branch of the Codroy River and inundated the St Croix Farm near Cold Brook[1]

1976 spring	• rainfall and rafting ice caused road and bridge washouts in surrounding areas, but there are no reports of problems in the study area[1]
1978 Jan 14 - 16	• high flows and break-up ice combined to destroy a 500 foot section of the Grand Bay Bridge isolating four communities and 1500 residents. A temporary bridge was subsequently constructed 8 km downstream and opened 3 months later.
	• direct flood damage includes $0.35 million for the temporary bridge and $5.3 million for the new bridge which was opened in December 1983
1981 Feb 5 - 7	• an unconsolidated ice jam formed at the South Branch TCH bridge and flooded two nearby homes and surrounding farmlands. The downstream point of the ice blockage was 100-200m down-stream of the bridge, was in the order of 3.6 m thick (12 feet) and was alleviated by explosives[1]
1985 Apr - May	• bankfull flood elevations (about 2 m above summer levels) reported to have occurred at Grand Codroy Provincial Park[5] (road to upper Ferry, however, has never been flooded)

4.0 HYDROLOGIC STUDIES

One of the basic purposes of this study is to provide reliable estimates of the 20-year and 100-year recurrence interval flood levels, and illustrate the extent of flooding on Flood Risk Maps for the Codroy Valley area. As these flood levels may arise from high flows on the river or be associated with ice problems (which are affected by streamflow), the first task was to derive information on flood flows in the study area.

Flood flow estimates can be derived by various techniques, including:

- frequency analysis of streamflow records from a hydrometric station at or near the site of interest
- computer simulation of flood flows using a mathematical watershed model and long-term weather records
- regional frequency analysis of flow records taken from surrounding hydrometric stations in the area of interest.

The choice of the most appropriate method is often constrained by the availability of data. In the Codroy Valley, there is a streamflow monitoring station on the Little Codroy River near Doyles (station 02ZA003). The station has 7 years of natural flow record, but is not in the valley of the main study area and has relatively small drainage area (139 km^2). As a result, direct extrapolation of data from this location to the Grand Codroy River may not provide completely reliable estimates of flood flows.

However, there are meteorological data near the site (beginning in 1960 at St. Andrews) and there are streamflow gauges nearby. These make it possible to estimate flood flows by computer simulation using a hydrologic model and to check the estimates of flood flows with a regional frequency analysis.

1986 Jan 16	1700 m long ice accumulation observed at TCH bridge near Coal Brook[6]. This accumulation was about 3.5 m above normal levels[4] (10-12 feet) and was followed by progressive break-up and ice blockages between 1-2 km downstream of the bridge. Subsequent observations[7] (Jan 23) showed that the accumulation was about 0.76 thick[7] (2-3 feet) at a nearby cattle barn which could not be reached for 2 days during this flood.
1988 Mar 29	ice jamming at the South Branch TCH bridge at about the same point as in 1981. The ice accumulation was initiated by a consolidated ice sheet downstream of the bridge (about 200 m) and extended about 150 m upstream of the bridge. Water levels were approximately 5.75 m below the top of the bridge (elevation 40.8[1] m) and high water levels lasted for about 3 weeks.[8]

1 "Flooding Events in Newfoundland and Labrador - An Historical Perspective" Env. Canada, Water Planning and Management Branch, IWD Report 80-WPMB-4, 1980 Halifax (report on Codroy Area, St George's Bay Area)

2 Carol Brake, Personal Communication, 1988

3 R. Picco, report to FDRP Committee, 1985

4 L. Vasallo, Newfoundland Environment, Files on Flood Damage Reduction

5 Park Warden, Grand Codroy Provincial Park

6 H. Gale, Personal Communication, 1988

7 M. Goebel, Newfoundland Environment, File on Flood Damage Reduction

8 J. Greer, Personal Communication, 1988

4.1 Watershed Modelling

4.1.1 Data Base

Most hydrologic models which simulate streamflow require input data describing:

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

Monitored streamflow 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.

4.1.2 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 1:50,000 scale and 1:250,000 scale mapping developed by the Federal Department of Energy, Mines and Resources, and 1:2,500 scale mapping of the river channel prepared by Kenting Earth Sciences International Inc. in 1988 for the Canada-Newfoundland Flood Damage Reduction Program.

Figure 4-1, a fold out map at the back of the report, delineates the entire Grand Codroy River Basin and the boundaries of several of the principal subcatchments. The Figure also shows the catchment area of the Little Codroy River above Doyles (catchment number 1).

This adjacent catchment is of interest to the study because there is a streamflow monitoring station at Doyles, and the characteristics of the catchment are similar to most subcatchments in the Grand

Codroy Basin. Its headwaters rise in the Long Range Mountains at elevation 565 m, its surface area is approximately 139 km^2 and the watershed slope (from topographic divide to outlet) is approximately 2.3%.

The Grand Codroy River Basin was discretized into seven sub-basins. Two sub-basins were delineated for the South Branch, two for the North Branch of the Grand Codroy River Basin and the remaining three sub-basins for the main branch of the Grand Codroy River downstream of the confluence of the North and South Branch.

Catchment number 5 represents the North Branch Tributary watershed area. This 365 km^2 tributary drains the headwaters originating in the Long Range Mountains at elevation 518 m to the confluence of the South Branch Tributary at the Town of South Branch. The watershed slope is 2%.

The South Branch Tributary (sub-basin numbers 4 and 3) is slightly smaller with drainage area of 276 km^2. Its headwaters also originate in the Long Range Mountains at elevation 550 m and watershed slopes for sub-basin number 4 and 3 are 1.5% and 3.2%, respectively.

The total downstream section of the study area is delineated by catchment number 2 which has a total drainage area of 172.3 km^2 and a watershed slope of 3%.

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. Figure 4-1 and Figure 4-2 show that the river channel is steep, as are the basin slopes - particularly along the eastern watershed boundary from Doyles to northeast of Coal Brook. 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.

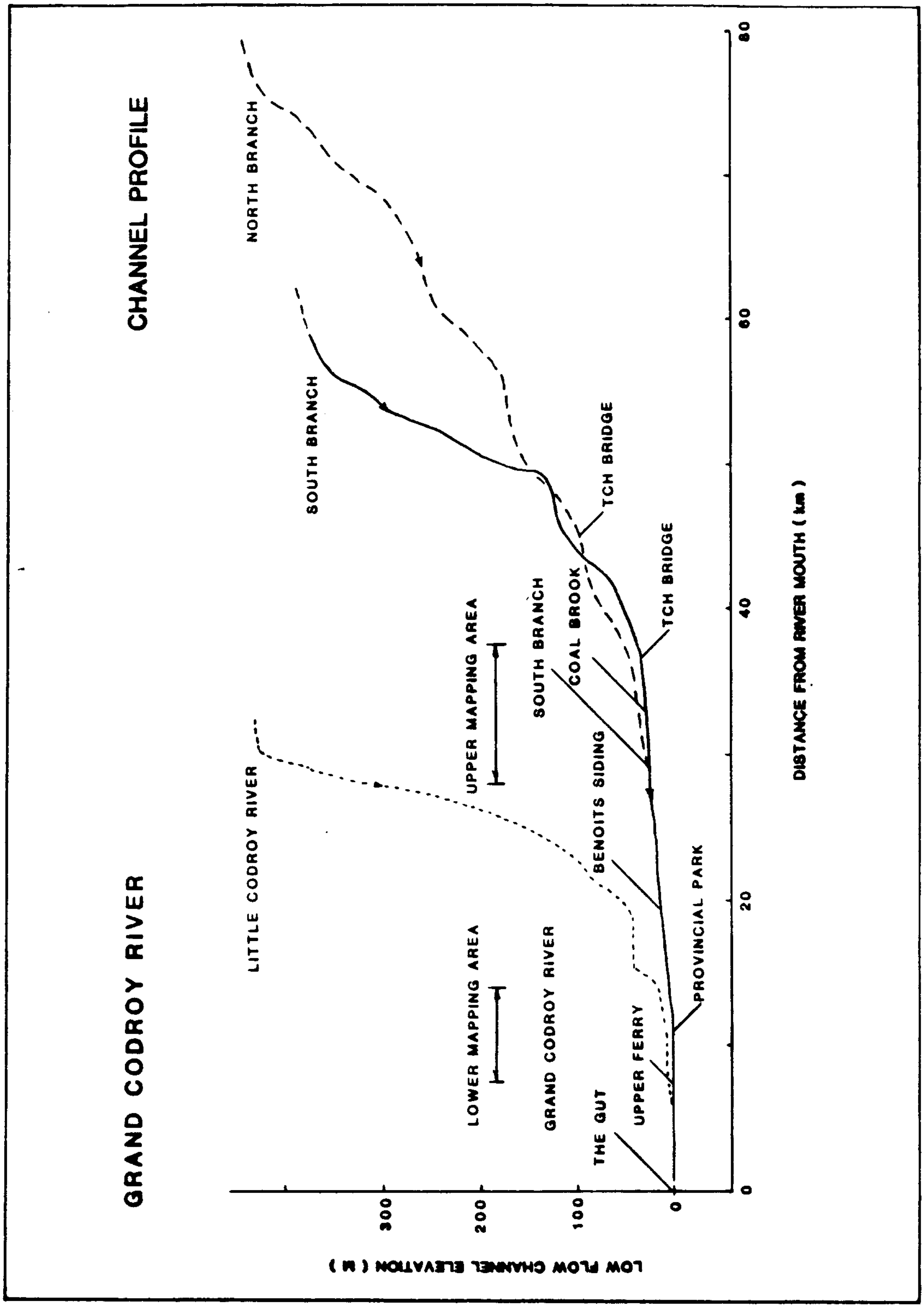

GRAND CODROY RIVER
CHANNEL PROFILE
LOW FLOW CHANNEL ELEVATION (M)
0
100
200
300
DISTANCE FROM RIVER MOUTH (km)
0
20
40
60
80
NORTH BRANCH
SOUTH BRANCH
TCH BRIDGE
TCH BRIDGE
COAL BROOK
SOUTH BRANCH
UPPER MAPPING AREA
LITTLE CODROY RIVER
BENOITS SIDING
PROVINCIAL PARK
LOWER MAPPING AREA
GRAND CODROY RIVER
UPPER FERRY
THE GUT

FIGURE 4-2

4.1.3 Surficial Soils

Surficial soils of the Codroy Valley are described in published and unpublished reports by several authors. Those of particular value contain maps of the extent and distribution of various soil groupings, because these maps can be overlain on topographic mapping to assist in the delineation of hydrologic modelling units.

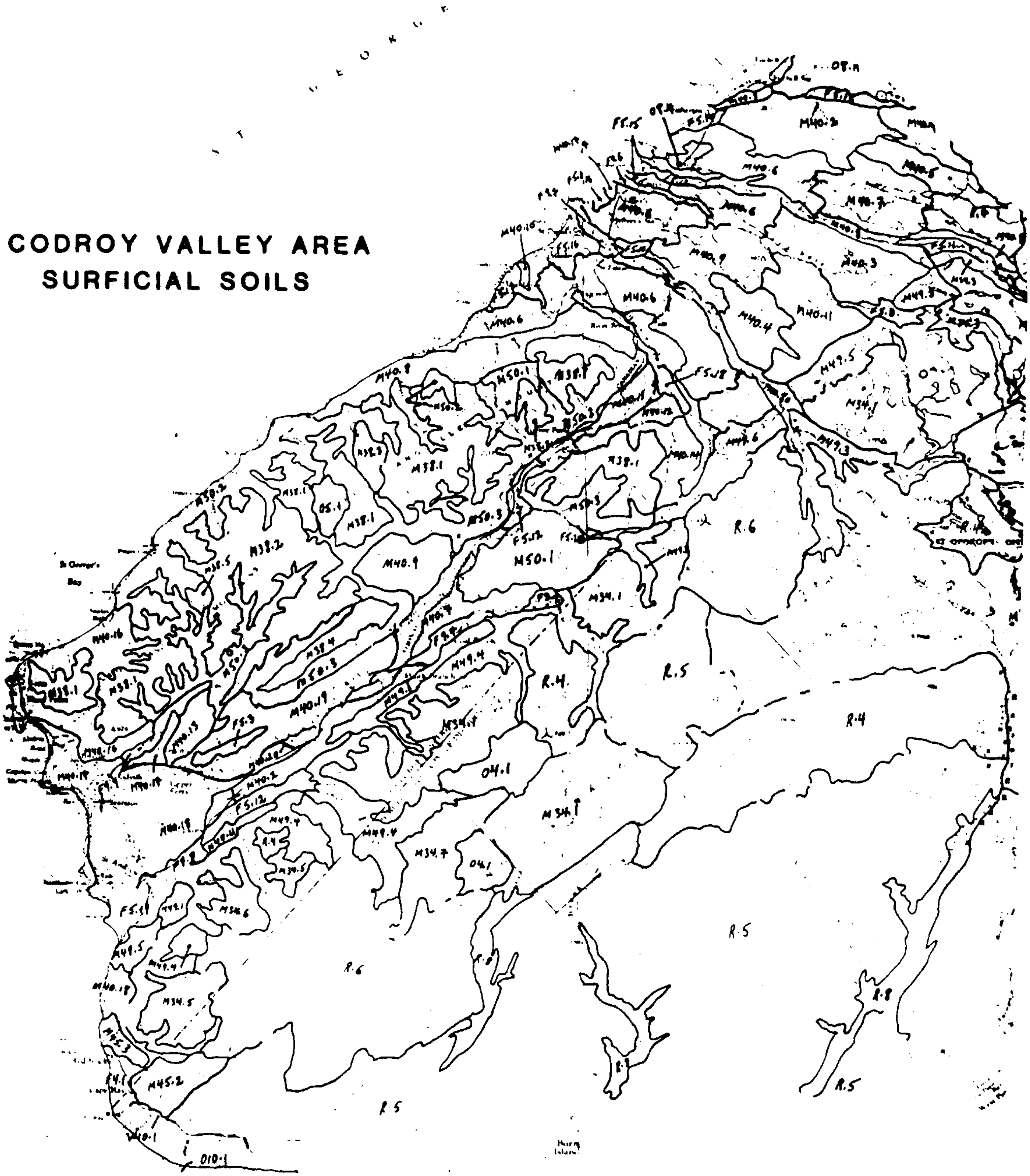
The unpublished report of surficial soils in the Stephenville-Port aux Basques area and accompanying mapping at 1:250,000 scale (Hender, 1988) is unique in that it is the only mapping report which describes all of the study area and does so in a consistent manner. Hence, this mapping report has been selected for use in this study and a copy of the draft mapping is enclosed as Figure 4-3.

Additional soils data are given in the publication, "The Soils of the Codroy Valley" (Woodrow et al, 1979). This report and mapping at 1:50,000 scale initially appears to provide a better set of data for describing hydrologic soils conditions. Unfortunately, however, most soil categories in this report combine more than three, hydrologically different soils into each mapping unit, making it impossible to confidently identify most drainage categories for modelling. However, this mapping report provided a good check on some of the drainage categories in the 1:250,000 scale study.

The 1:250,000 scale mapping report describes the drainage classification of 33 map units in the Codroy Valley. These drainage classes range from "well drained" soils (e.g. sandy loam) to "very poorly drained" soils, such as are associated with fens and bogs. These drainage classifications are linked to hydrologic soils infiltration classifications by Table 4-1, which gives information on both and a description of soil moisture content and run-off.

Table 4-2 lists all 33 soils map units in the Codroy Valley watershed along with the drainage class and average hydrologic soil condition.

CODROY VALLEY AREA SURFICIAL SOILS



SURFICIAL SOILS -STEPHENVILLE PORT AUX BASQUES

SOIL DIVISION BOUNDARY

M50.1 SOIL UNIT CODE

(HENDER,1988)

FIGURE 4-3

TABLE 4-1

CODROY VALLEY

DRAINAGE AND HYDROLOGIC SOILS CLASSIFICATIONS

DRAINAGE CLASSIFICATION	DESCRIPTION OF MOISTURE CONTENT AND STORAGE CAPACITY	HYDROLOGIC SOIL CLASSIFICATION
Rapidly Drained	Soil moisture content seldom exceeds field capacity* in any horizon, except immediately after water additions. Water removed rapidly with low water storage capacity and moisture deficit in dry years. Soils have low runoff potential even when saturated.	A
Well Drained (Good Drainage)	Soil moisture content does not normally meet field capacity for a significant part of the year. Soil water is removed readily, the water storage capacity is intermediate, and soils retain sufficient moisture for agricultural use.	AB
Moderately Well Drained	Soil moisture is in excess of field capacity for a small, but significant period of the year, and is removed slowly. Water storage is intermediate to high. Soils have moderate infiltration rates even when wet.	B
Imperfectly Drained	Soil moisture is in excess of field capacity and remains so in subsurface horizons for moderately long periods of the year. Soils remain wet for a significant part of the growing season.	BC
Poorly Drained	Soil moisture in excess of field capacity remains in all soils level for a large part of the year. Soils are wet most of the time that they are not frozen, and have low infiltration rates.	C
Very Poorly Drained	Free water remains at or within 30 cm of the soil surface most of the year. Soils have high runoff potential/very low infiltration rates.	D

* field capacity is the amount of moisture retained by a soil (initially at a high water content) after is has drained for 2-3 days.

TABLE 4-2
HYDROLOGIC SOIL CONDITIONS

SOILS MAP UNIT	MATERIAL (AND SLOPE%)	DOMINANT SOIL DRAINAGE CLASS	HYDROLOGIC SOIL
M 34-1	Morainal veneer over/with hummocky bedrock (16-30%)	Moderately well drained to poorly drained	BC/C
M 34-5	Morainal veneer over hummocky bedrock (10-15%)	Moderately well drained	B
M 34-7	Moraine veneer, hummocky bedrock and sloping bog (2-30%)	Moderately well to poor drainage	BC
M 38-1	Morainal blanket over rolling bedrock (6-9%)	Imperfectly drained	BC
M 38-2	Morainal veneer over rolling bedrock and sloping fen (0-5%)	Imperfect to poor drainage	BC/C
M 38-3	Morainal veneer over ridged bedrock and rock (31-45%)	Imperfectly drained	BC/C
M 38-4	Morainal veneer over inclined bedrock (31-45%)	Imperfectly drained	BC
M 40-2	Inclined moraine and sloping bog (2-9%)	Poorly drained	C
M 40-7	Ridged moraine (6-9%)	Poorly drained	C
M 40-9	Rolling moraine (6-9%)	Poorly drained	C
M 40-11	Hummocky moraine/ glaciofluvian (16-30%)	Moderately well (70%) to well drained (30%)	B/AB
M 40-12	Inclined moraine (16-30%)	Moderately well drained	B
M 40-13	Morainal blanket over ridged bedrock (16-30%)	Moderately well drained	B
M 40-14	Morainal blanket over rolling bedrock (10-15%)	Poorly drained	C
M 40-17	Hummocky moriane (6-9%)	Moderately well drained	B
M 40-18	Rolling moraine (6-9%)	Imperfectly drained	BC

SOILS MAP UNIT	MATERIAL (AND SLOPE%)	DOMINANT SOIL DRAINAGE CLASS	HYDROLOGIC SOIL
M 40-19	Morianal blanket over rolling bedrock and glaciofluvial terrace (6-9%)	Poorly drained (70%) to well drained (30%)	BC
M 40-20	Hummocky moriane with glacio-fluvial terrace and sloping bog (2-15%)	Poorly to well drained	BC/C
M 49-1	Inclined moraine (16-30%)	Moderately well drained	B
M 49-3	Morainal veneer over inclined bedrock (46-70%)	Moderately well drained	B
M 49-4	Morainal veneer over inclined bedrock (31-45%) with steep bedrock (71-100%)	Moderately well to very poorly drained	BC/C
M 49-6	Morainal blanket over hummocky bedrock with some bog (16-30%)	Well to poorly drained	B
M 50-1	Morainal veneer over rolling bedrock (16-30%)	Imperfectly drained	BC
M 50-3	Morainal veneer over steep bedrock (46-90%)	Imperfectly drained	B
R-4	Rock (30%) with thin shallow till and steep slopes (30-60%)	Poor to very poor drainage	CD
R-5	Exposed bedrock (90%)	Very poor	D
R-6	Exposed bedrock (40%) with shallow till and horizontal fen	Poor to very poor	CD
O 4-1	Flat bog and fen	Very poorly drained	D
O 5-1	Flat bog and fen	Very poorly drained	D
F 5-3	Hummocky glaciofluvial (6-9%)	Well drained	AB
F 5-12	Hummocky glaciofluvial with hummocky moraine (10-15%)	Well drained to moderately well	AB
F 5-20	Undulating glaciofluvial (2-5%)	Poorly drained	C
F 9-7	Level Fluvial (0-0.5%)	Well drained	AB

Overall, the presence of bedrock near the soil surface at higher elevations, and the predominance of imperfectly drained soils with high runoff potential in the valley are expected to contribute to a relatively high rainfall-runoff ratio.

4.1.4 Land Use

The watershed modelling investigations employed the most reliable land use data available at present. This included:

- cultural information from 1970 topographic mapping
- information from 1984 aerial photography
- data from 1988 topographic mapping (Kenting, 1988)
- information from the Newfoundland Soil Survey Report (Woodrow et al, 1979).

It is reported (Woodrow et al, 1979) that the first European settlers in the area were French-speaking Acadians, involved in fishing and combined farming and fishing as their main occupations. Industries today include agriculture, fishing, tourism and service industries.

Most of the agricultural development in the valley started in the 20th Century. Productive land is in hay and pasture and there is some dairying and poultry farming. Potatoes, cabbages and turnips are also grown and small areas are in oats. These uses have contributed to slight changes in the surface imperviousness of the area, but not to significant changes in surface runoff.

The lowlands and escarpment slopes are well wooded except for cleared agricultural land, organic and poorly drained soils, and mountain tops. Balsam fir and black spruce are commercial tree species found in the area, along with spruce, birch and other scattered tree types.

The forest in the uplands and mountains consists mainly of wind pruned, scrubby black spruce, balsam fir and larch. Sedges, ericaceous shrubs and peat deposits are common and rock outcrops occur quite frequently.

4.1.5 Hydrometeorology

4.1.5 Hydrometeorology

Precipitation data is available for the Codroy Valley area from the Atmospheric Environment Service, Environment Canada for stations at Doyles, St. Andrews, Stephenville Airport and Port aux Basques. Records of daily, 6-hourly and hourly total precipitation were available from the above stations. Some stations such as Stephenville Airport provided quality long-term records for more than one of the three above record formats (daily, 6-hourly, and hourly records). These Airport records are from a continuously recording, tipping bucket gauge which was first employed in 1967. Supplementing this is an excellent record of 24-hour rainfall since 1950 and 6-hour totals since 1966 at other stations. However, the remaining stations are limited in period of record and format type (Figure 4-4).

It must be noted, however, that the precipitation gauges correctly measure the volume and timing of rainfall at only one location - the gauge site. Different precipitation amounts and timing occur at other locations in a basin and, consequently, precipitation runoff cannot be perfectly simulated from a single gauge site. This is shown in Figure 4-5 which illustrates that the snowpack water equivalent in the headwater areas is greater than the coastal areas (which contain the meteorological gauges), and in Figure 4-6 which projects these precipitation variations in terms of runoff.

The data base selected for modelling purposes is a blend of information from most of the above-mentioned monitoring sites. The years from 1960 to mid-1966 (6 years) are covered by data from St. Andrews, and those from 1981-1987 (7 years) are taken from daily records at Doyles and distributed by Stephenville Airport hourly records. Records from Stephenville Airport were used for the 1966-1981 period because of missing precipitation records at Doyles and St. Andrews.

Initially 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. However, during the calibration/validation of the hydrologic model (described in later sections of this report) it was noted that the response of the watershed did not always reflect that observed at the streamflow

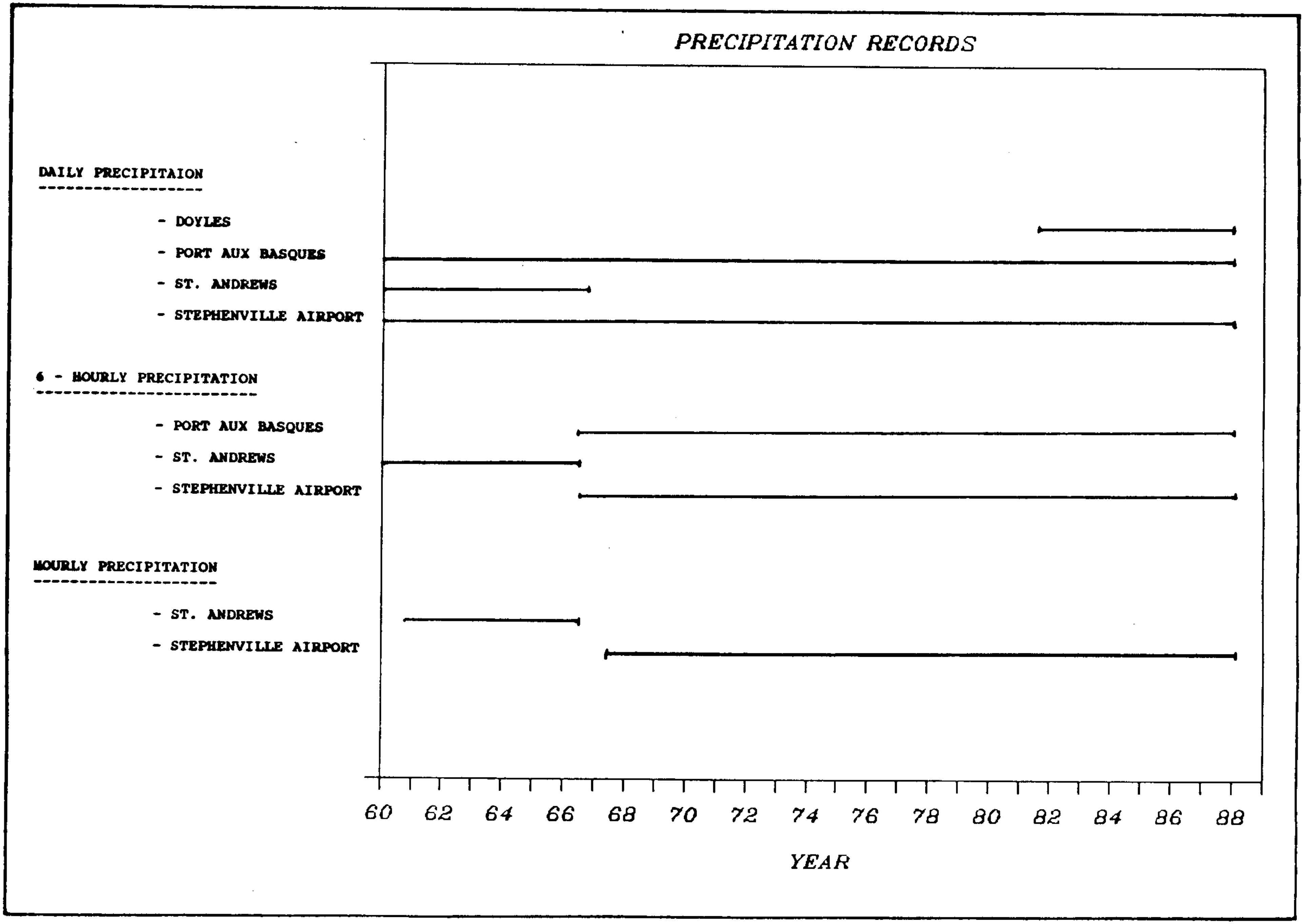
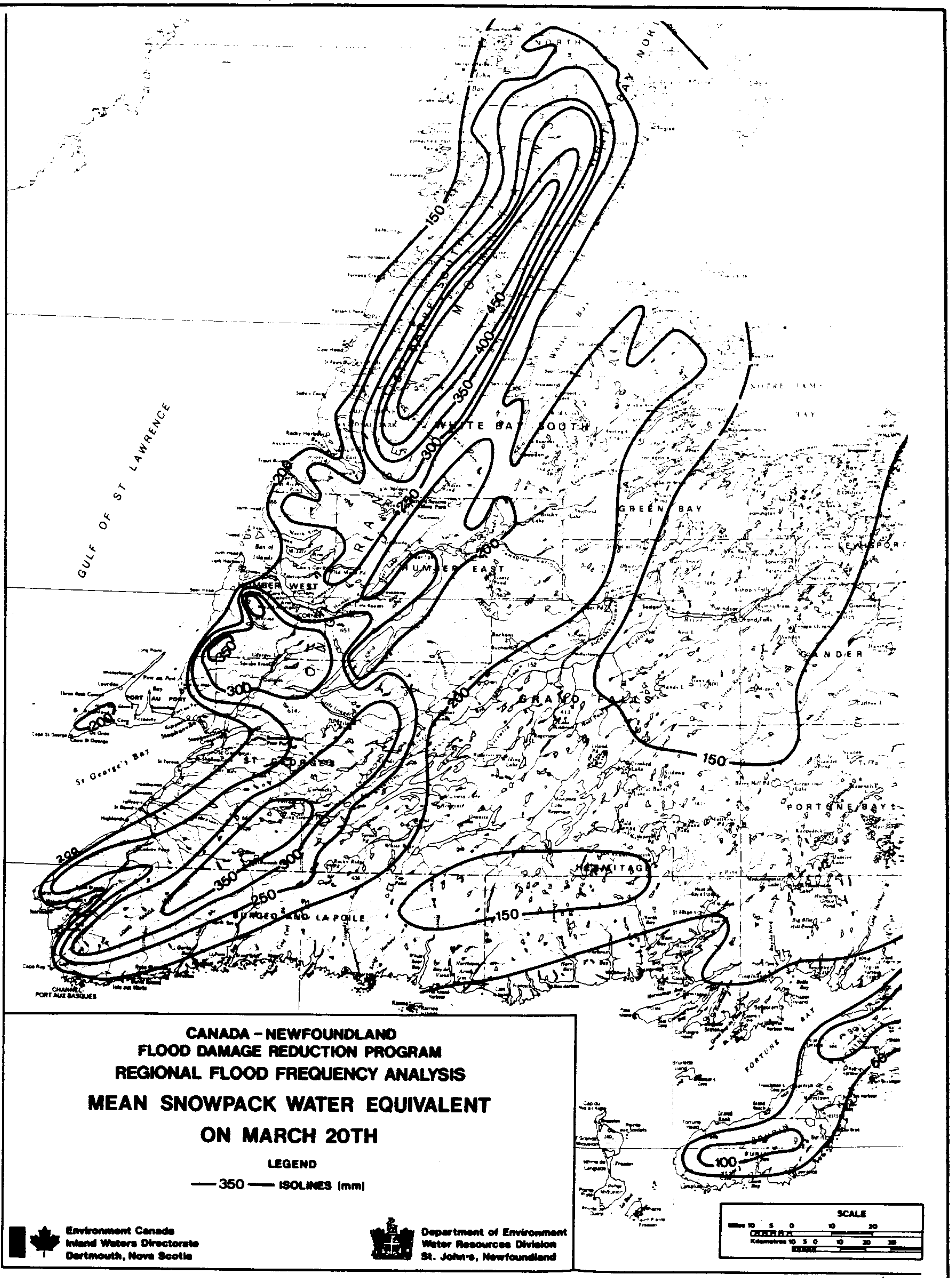

PRECIPITATION RECORDS
DAILY PRECIPITAION
- DOYLES
- PORT AUX BASQUES
- ST. ANDREWS
- STEPHENVILLE AIRPORT
6 - HOURLY PRECIPITATION
- PORT AUX BASQUES
- ST. ANDREWS
- STEPHENVILLE AIRPORT
HOURLY PRECIPITATION
- ST. ANDREWS
- STEPHENVILLE AIRPORT
60 62 64 66 68 70 72 74 76 78 80 82 84 86 88
YEAR

FIGURE 4-4



CANADA - NEWFOUNDLAND
FLOOD DAMAGE REDUCTION PROGRAM
REGIONAL FLOOD FREQUENCY ANALYSIS

MEAN SNOWPACK WATER EQUIVALENT ON MARCH 20TH

LEGEND

—350— ISOLINES (mm)

Environment Canada
Inland Waters Directorate
Dartmouth, Nova Scotia

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

FIGURE 4-5

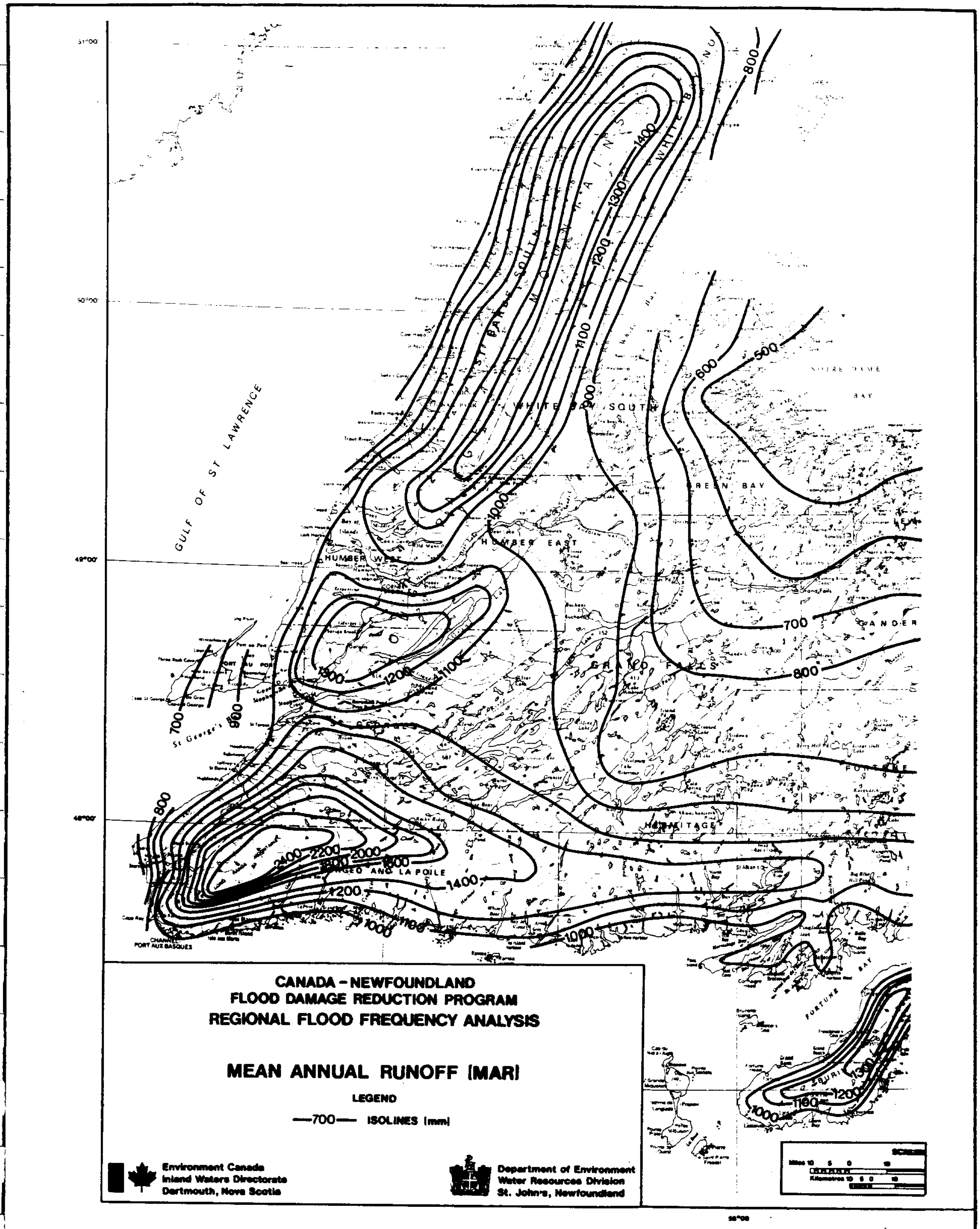

CANADA - NEWFOUNDLAND
FLOOD DAMAGE REDUCTION PROGRAM
REGIONAL FLOOD FREQUENCY ANALYSIS
MEAN ANNUAL RUNOFF (MAR)
LEGEND
—700— ISOLINES (mm)
Environment Canada
Inland Waters Directorate
Dartmouth, Nova Scotia
Department of Environment
Water Resources Division
St. John's, Newfoundland
GULF OF ST LAWRENCE
LONG RANGE MOUNTAINS
WHITE BAY NORTH
WHITE BAY SOUTH
NOTRE DAME BAY
HUMBER WEST
HUMBER EAST
HERMITAGE
BURGEO AND LA POILE
FORTUNE BAY
CHANNEL PORT AUX BASQUES
St George's Bay
49°00'
48°00'
50°00'
51°00'
700
800
900
1000
1100
1200
1300
1400
1600
1800
2000
2200
2400
600
500

FIGURE 4-6

gauge on the Little Codroy. Consequently, the relationships shown in Figures 4-5 and 4-6 were employed to areally distribute precipitation within the study area (as further discussed in Section 5.5.4).

4.1.6 Streamflow Data

Streamflow has not been monitored in the study area but is gauged nearby in the Little Codroy River near Doyles (station number 02ZA003). The 139 km^2 basin is immediately adjacent to the Grand Codroy basin, and has flow records from 1982 to the present. Data from this location was available to the study on magnetic tape for the period of record until November, 1987. More recent data was obtained in paper copy.

Streamflow data is the only information which can be used to verify hydrologic simulations. Consequently, several discussions were held with Water Survey of Canada (WSC) personnel in the Corner Brook and Dartmouth offices regarding the quality of data obtained from measurements at this site. This was warranted in view of the possibility of revisions to flow estimates and to obtain information on the quality of the monitored record.

The quality of data was described as good up to 140 m^3/sec, but it was suggested that discharge estimates greater than 200 m^3/sec may vary by about ± 25% because of mobile bed conditions at the gauge site.

5.0 HYDROLOGIC MODELLING

5.1 The QUALHYMO Model

The hydrologic model, QUALHYMO (Wisner and Rowney, 1985), was chosen to estimate peak flows throughout the watershed after careful consideration of the requirement for hydrologic information concerning the magnitude and frequency of flood discharges. 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 runoff but is most suited to analysis of river basins where a continuous flow record is required in order to select peak flows. Such was the case in a previous study along the Waterford River in St. John's, for example, where QUALHYMO proved well suited for long-term simulations (Fenco Newfoundland, 1988).

Unlike other single event rainfall-runoff models (which require the modeler to approximate watershed conditions before each storm), the continuous simulation capability of QUALHYMO enables hour-by-hour updating of soil moisture, snowmelt and baseflow. 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 the Technical Appendix.

5.2 Basin Discretization

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

- the subcatchments shown in Figure 4-1 should be subdivided into additional areas to obtain a better definition of flood flows. Consequently:

- subcatchment No. 5 was divided into two subcatchments to obtain an intermediate flow point at the Trans-Canada Highway crossing of the North Branch;
- subcatchment No. 2 was divided into three subcatchments to give intermediate flow points at Benoit Siding and the outlet of Ryans Brook.

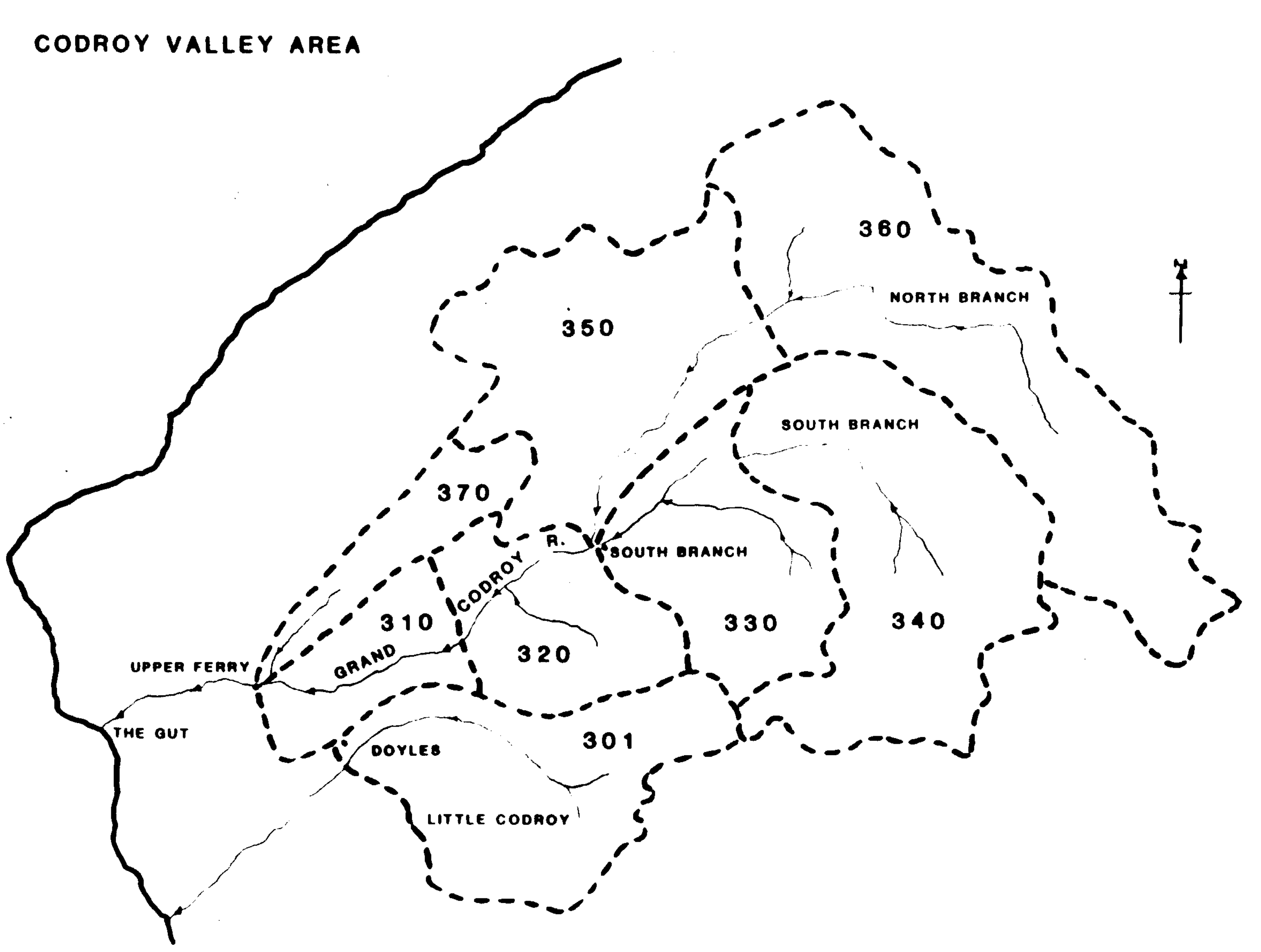
Figure 5-1 shows the geographical distribution of the discretized sub-catchments.

5.3 Hydrologic Soil Complexes

In similar watershed modelling studies, land use is combined with hydrologic soils classifications to obtain a weighted hydrologic soil complex number (i.e. CN number) for each subcatchment. For this study, the land use component was determined to be primarily wooded, agricultural, and wetland with a very limited amount of urban development. For significantly larger areas or urban land use a percentage of the subcatchments are normally assigned as impervious. However, as the amount of urban development in the Codroy Valley Basin does not significantly affect discharge rates in the Grand Codroy River it was not considered. Table 5-1 presents our initial estimates of weighted CN values for each subcatchment.

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

The QUALHYMO model uses the U.S. Soil Conservation Service (SCS) procedure to determine runoff from pervious land segments. This involves an approach which continuously keeps track of soil moisture conditions for each hydrologic soil cover complex number (CN). The model transforms that single number into a set of numbers which include a maximum, minimum and rate or change in soil moisture conditions (SMAX, SMIN and SK) which change each day depending on the antecedent precipitation.



HYDROLOGIC MODEL SUB BASIN UNITS

FIGURE 5-1

<u>TABLE 5-1</u>

<u>CODROY VALLEY HYDROTECHNICAL STUDY</u>

<u>INITIAL</u> ESTIMATES OF

<u>SUBCATCHMENT AREAS, COMPLEX NUMBER (CN) MODEL INPUT DATA</u>

Sub-Basin No.	Area (ha)	Weighted CN (AMC II)*
301	12642	85
310	5516	74
320	7453	80
330	4900	68
340	22700	88
350	14912	83
360	21654	84
370	4259	81

* AMC II (Antecedent Moisture Condition II) is average antecedent moisture conditions.

5.5 Hydrologic Modelling

5.5.1 Model Parameters

Hydrologic-soil cover complexes for each of the subcatchments given in Table 5-1 were calculated and are provided in Table 5-2, along with the drainage areas of each lumped catchment, the longest river length and corresponding difference in elevation for each subcatchment. Calculated values of hydro-graph peak and shape parameters (Tp and K) and preliminary estimates of soil moisture parameters (SMIN, SMAX and SK) are also given in Table 5-2.

Lastly, preliminary estimates of the baseflow recession constant were developed by analyzing the recession of various observed hydrographs at the WSC gauge on the Little Codroy River.

5.5.2 Hydrologic Modelling - Little Codroy River

The discretized basins outlined in Table 5-2 include the Little Codroy River basin (subcatchment number 301). This sub-basin is hydrologically similar to sub-basins in the Grand Codroy River system and was our initial focus for modelling.

The model of this basin was assembled for two purposes:

- The main purpose was to generate a full sequence of streamflows from 1962 to the present - to identify peak flows in the 20 years before the WSC streamflow gauge was put in operation in the Little Codroy basin (1982 to present). The value of this work is that flood peaks from these peak flow periods quadruple 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 the one WSC-monitored catchment rather than all the detailed catchments). The detailed modelling described later benefits from this strategy by beginning

TABLE 5-2

LUMPED MODEL - INITIAL INPUT PARAMETERS

CODROY RIVER BASIN

Sub-Catch Number	Drainage Area (Km^2)	CN Value (AMC II)	Longest River Length (Km)	Difference Elevation (m)	Unadjusted K (hr)	Unadjusted Tp (hr)	Unadjusted SMIN (mm)	Unadjusted SMIX (mm)	SK
301*	139.0	85	22.8	533.4	0.71	3.58	9	360	.057
310	55.2	74	13.6	335.3	0.61	2.54	43	1225	.105
320	74.5	80	14.6	503.0	0.50	2.68	15	500	.076
330	49.0	68	12.2	396.2	0.48	2.29	33	945	.081
340	227.0	88	27.3	403.9	1.22	4.96	7	270	.074
350	149.1	83	22.7	472.4	0.84	3.91	8	388	.071
360	216.5	84	40.3	411.5	1.37	5.23	11	410	.078
370	42.6	81	16.6	335.3	0.59	2.38	11	460	.074

The year 1962 was selected as the beginning year for simulation as it was the first year that complete detailed precipitation began to become available in the study area (at St. Andrews).

5.5.3 Calibration Procedure

The preliminary estimates of model parameters for the Little Codroy River (Table 5-2) 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.

The following series of long-term runoff periods were selected for calibrating the shape of recession limb, because they contained an uninterrupted record of both streamflow and precipitation data.

- 1 November - 1 December 1983
- 1 May - 15 June 1985
- 1 September - 15 November 1987

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 0.000005 mm/sec/mm provided fairly good agreement.

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 subcatchments are shown in Table 5-3.

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.

TABLE 5-3

LUMPED MODEL - FINAL CALIBRATION PARAMETERS**

CODROY RIVER BASIN

Sub-Catch Number	Drainage Area (Km^2)	CN Value (AMC II)	Longest River Length (Km)	Difference Elevation (m)	Corrected K (hr)	Corrected Tp (hr)	Corrected SMIN (mm)	Corrected SMIX (mm)	SK
301*	139.0	85	22.8	553.4	3.6	17.9	3	250	.042
310	55.2	74	13.6	335.3	3.1	12.7	43	1225	.105
320	74.5	80	14.6	503.0	2.5	13.4	15	500	.076
330	49.0	68	12.2	396.2	2.4	11.45	33	945	.081
340	227.0	88	27.3	403.9	6.1	24.8	3	250	.042
350	149.1	83	22.7	472.4	4.2	19.6	3	250	.042
360	216.5	84	40.3	411.5	6.9	26.2	3	250	.042
370	42.6	81	16.6	335.3	2.9	11.9	11	460	.074

* Little Codroy River Basin

** Values Defined By Precipitation:

	Rainfall Event	Amount (mm)	Duration (hours)
1 November - 1 December 1983:	Nov 6	27.7	9
	Nov 12	22.6	17
	Nov 17	61.0	58
1 May - 15 June 1985:	May 22	21.3	10
	June 2	42.5	2
	June 7	28.8	1
1 September - 15 November 1987:	Sept 2	27.0	11
	Oct 1	50.2	30
	Oct 28	71.6	15

5.5.4 Calibration of High Flow Periods

Rainfall

It was noted in this calibration process, and previously (Section 4.1.5), that the timing and magnitude of simulated flows for the Little Codroy watershed did not always correspond with those observed at the Water Survey streamflow gauge. Further reflection supported by Figures 4-5 and 4-6 suggested that the highland portions of the basin would receive more rainfall than the lowland locations of the precipitation gauges. After a number of tests, it was concluded that the runoff relationships shown in Figures 4-5 and 4-6 were well suited to areally distribute precipitation within the study area. For example, the precipitation runoff from subcatchment area 301 is approximately 1580 mm, or 1.4 times that of the St. Andrews/Stephenville area (1127 mm).

In all, the observed hourly rainfall at St. Andrews and Stephenville (which have 30-year norms within ± 3%) was prorated to the subcatchments in the study area by the following factors:

Area/ Subcatchment	Multiplication Factor for Rainfall from St. Andrews and Stephenville
301	1.4
310	1.0
320	1.2
330	1.5
340	1.7
350	1.0
360	1.4
370	1.0

It did not prove necessary to adjust snowfall amounts in the same manner, because cooler temperatures at higher elevations provided a snowpack water equivalent distribution from the St. Andrews/Stephenville data which is similar to that given in Figure 4-5.

Calibration

It is desirable to set certain model parameters before their influence is masked by the effect of the snowmelt. Consequently, calibration was initially undertaken with the following rainfall event:

- 30 October 1987.

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. Final values were selected for SMAX, SMIN, and other parameters (given in Table 5-3) following several runs involving adjustments to improve the correspondence between the observed and simulated hydrographs.

Figure 5-2 presents plots for showing reasonably good agreement between the simulated and observed hydrographs.

It is appropriate to make one comment at this point:

1. The precipitation input for the calibration period (coincident with streamflow records at the gauge) is based on 6-hourly data at Doyles, distributed by the hourly data at Stephenville Airport.

Therefore, it was not unexpected that simulations of runoff hydrographs may exhibit peaks which are not coincident (in time) with the observed hydrographs. This was the case for the October, 1987 simulation and the rainfall input was shifted back in a 6-hour block to align the simulated and observed peaks. Perfect alignment would have resulted with only a two to four hour shift.

Snowmelt

Missing streamflow data during some snowmelt-induced high flow periods (e.g. 1983) limited the selection of a good snowmelt event for calibration to:

- 28 January 1986

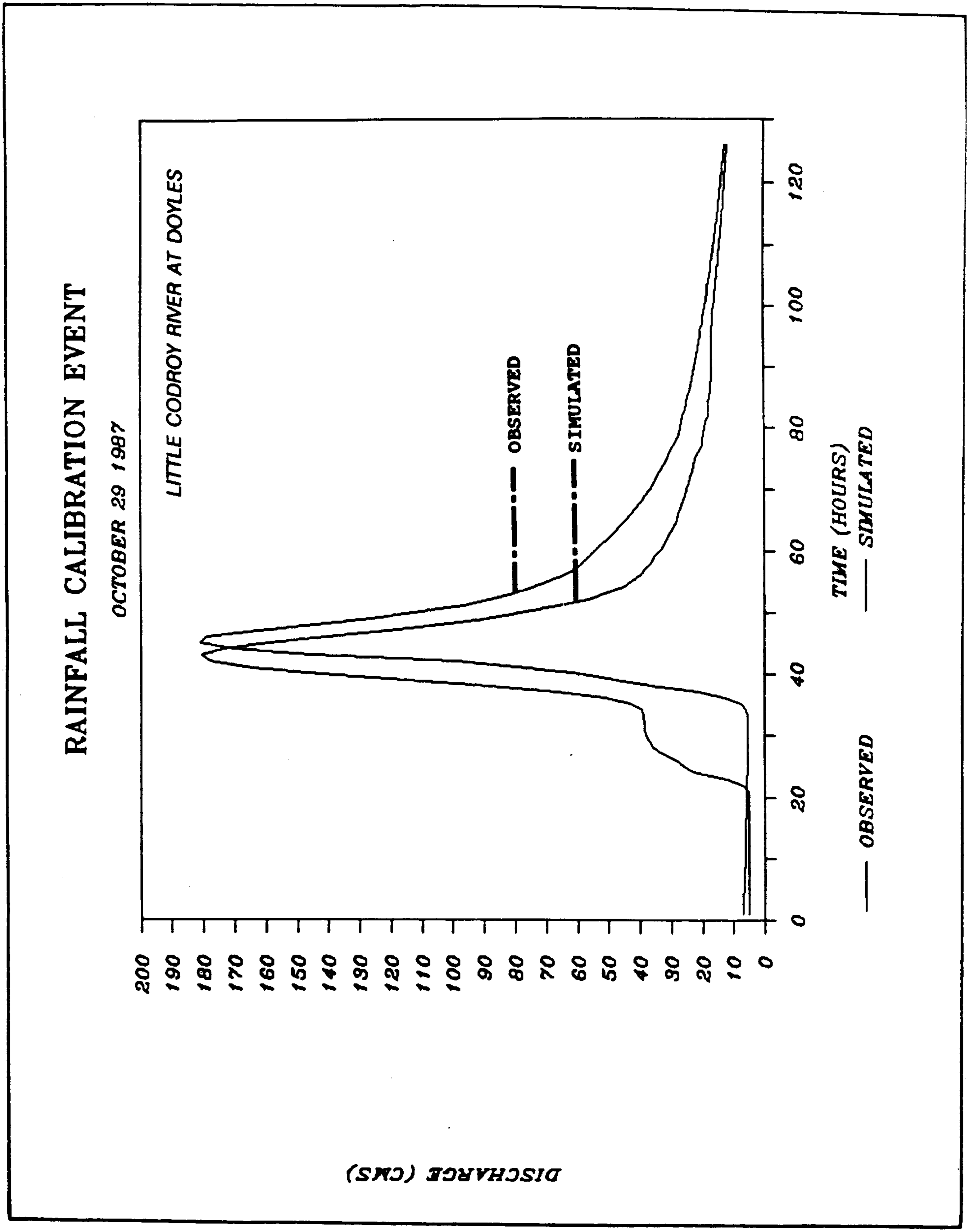

RAINFALL CALIBRATION EVENT
OCTOBER 29 1987
LITTLE CODROY RIVER AT DOYLES
OBSERVED
SIMULATED
DISCHARGE (CMS)
200 190 180 170 160 150 140 130 120 110 100 90 80 70 60 50 40 30 20 10 0
TIME (HOURS)
0 20 40 60 80 100 120
OBSERVED
SIMULATED

FIGURE 5-2

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. The snowmelt parameters, developed for somewhat warmer areas and lower elevations, were adjusted during this calibration and the selected values are as follows:

MFMAX -	maximum non-rain melt factor (0.004)
MFMIN -	minimum non-rain melt factor (0.002)
UADJ -	mean wind function value during rain or snow periods in mm/millibar (0.057)
SI -	areal water equivalent above which there is always complete areal snow cover (20 mm)

Figure 5-3 plots the calibration results which show reasonable agreement in time and magnitude of this snowmelt-runoff flood event.

5.5.5 Model Verification

Other high flow sequences were simulated to verify the model using the parameters established through the calibration procedures. These sequences included:

- 1 September 1984 (a second rainfall occasion)
- 30 April 1982 (a second rainfall-snowmelt sequence).

Plots of these simulations are given in Figure 5-4 and Figure 5-5, and Figure 5-6 compares observed and simulated peaks for all the verification and calibration simulations. The latter Figure also includes the peak flow period in 1983 and 1985 when monitored streamflow and rainfall are noted in records as being estimated.

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.

SNOWMELT CALIBRATION EVENT

28 JANUARY 1986

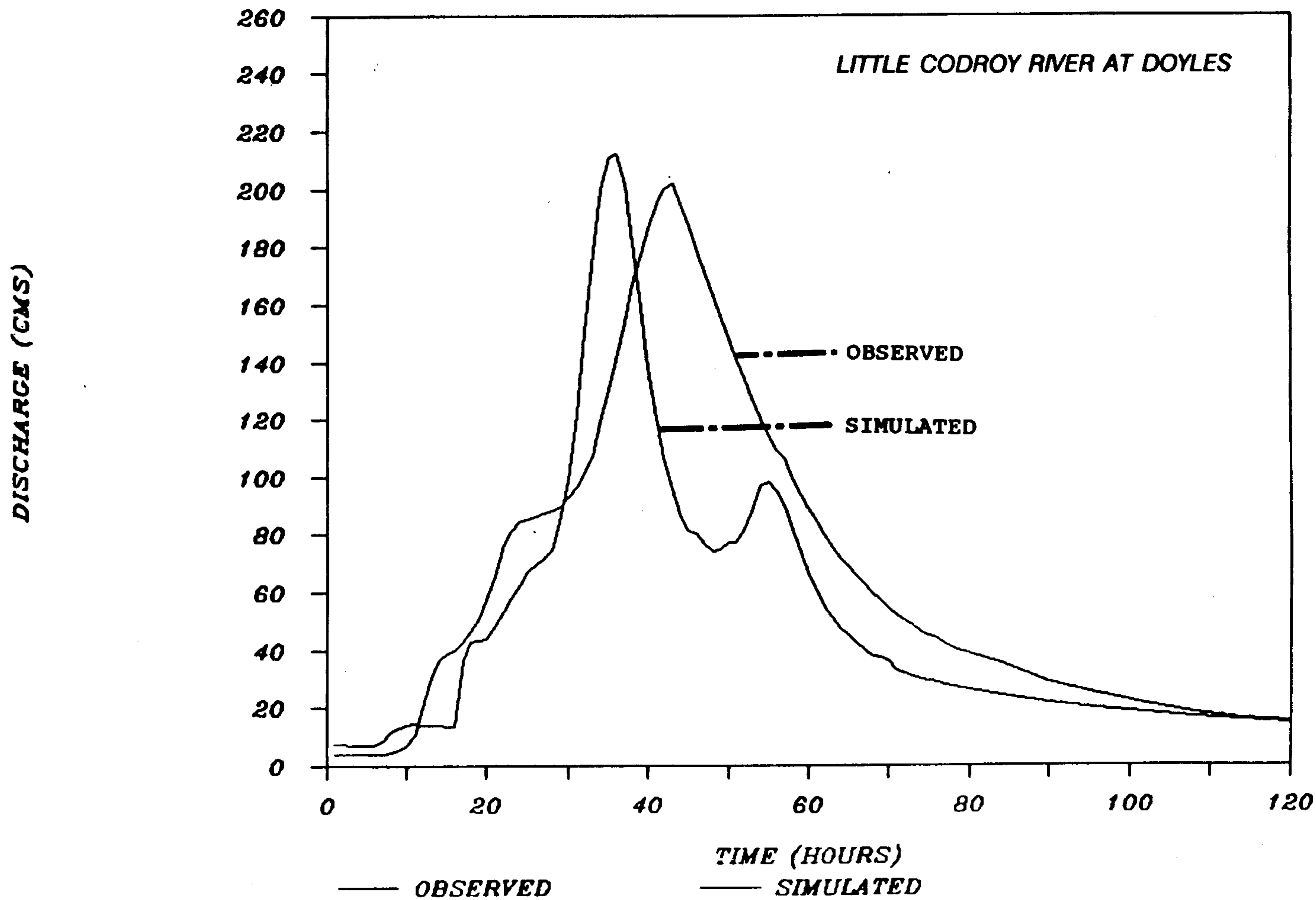


FIGURE 6-3

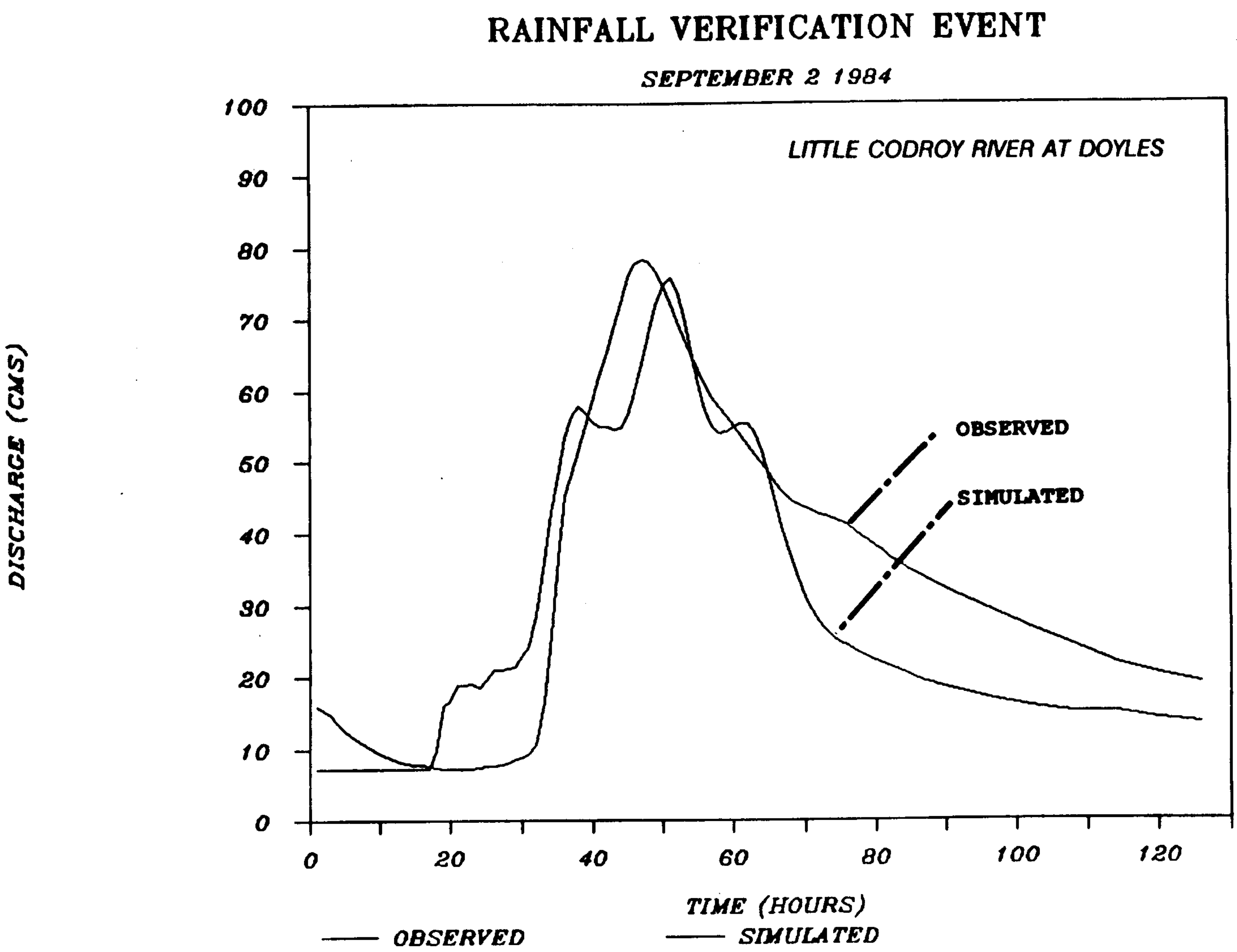

RAINFALL VERIFICATION EVENT
SEPTEMBER 2 1984
LITTLE CODROY RIVER AT DOYLES
DISCHARGE (CMS)
100
90
80
70
60
50
40
30
20
10
0
0
20
40
60
80
100
120
TIME (HOURS)
OBSERVED
SIMULATED
OBSERVED
SIMULATED

FIGURE 5-4

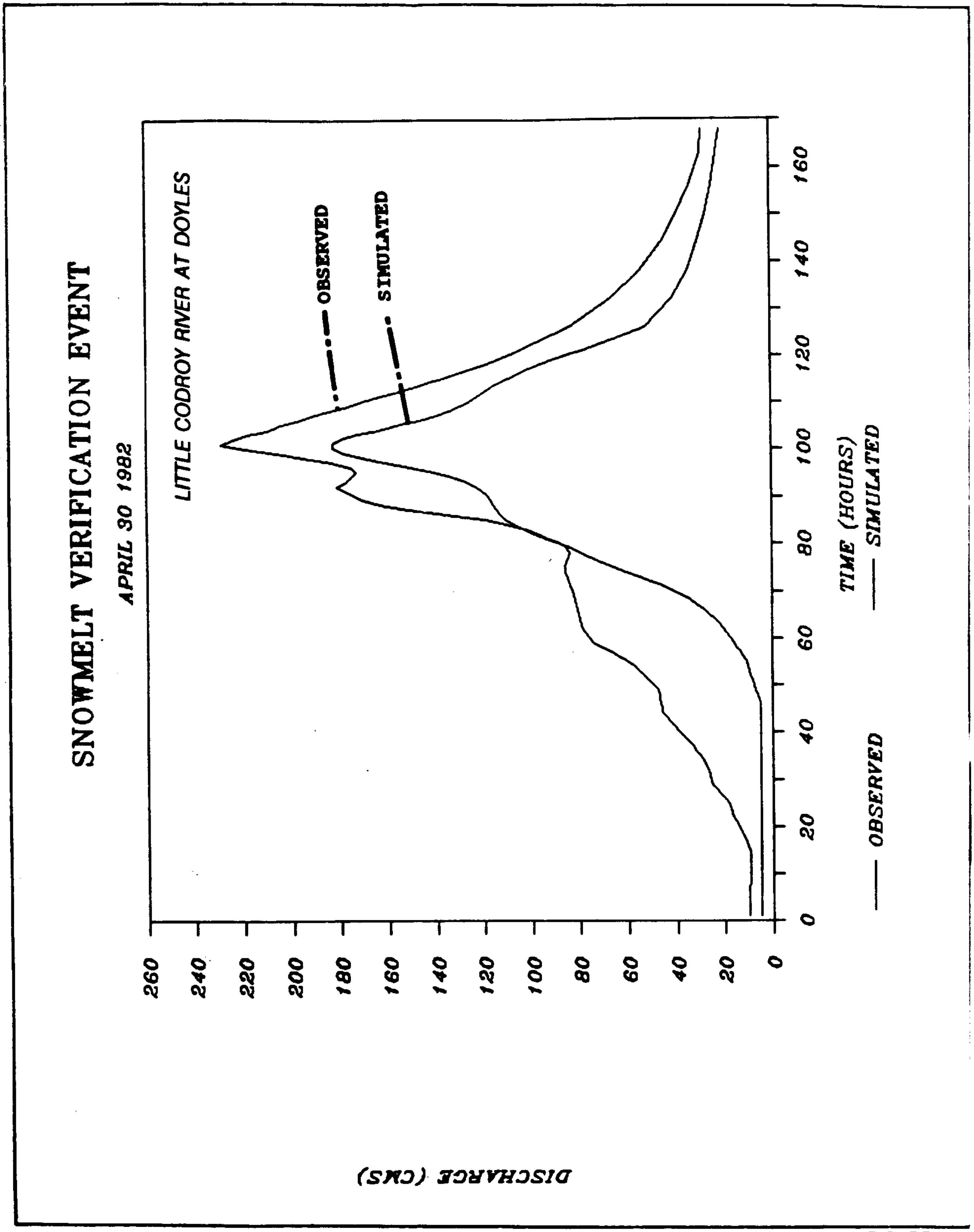
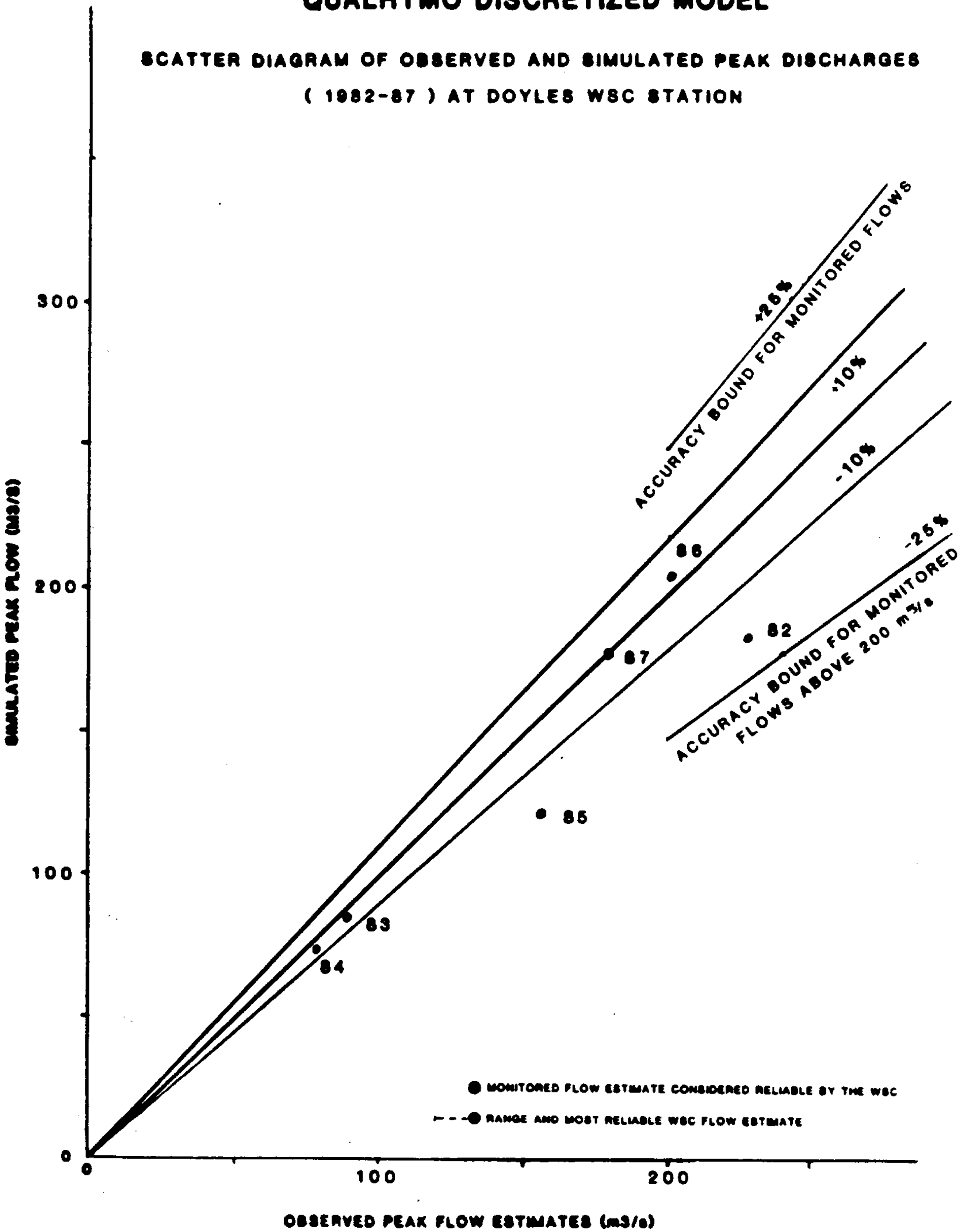

SNOWMELT VERIFICATION EVENT
APRIL 30 1982
LITTLE CODROY RIVER AT DOYLES
OBSERVED
SIMULATED
DISCHARGE (CMS)
TIME (HOURS)
0
20
40
60
80
100
120
140
160
180
200
220
240
260
OBSERVED
SIMULATED

FIGURE 5-5

QUALHYMO DISCRETIZED MODEL

SCATTER DIAGRAM OF OBSERVED AND SIMULATED PEAK DISCHARGES

(1982-87) AT DOYLES WSC STATION



5.5.6 Identification of High Flow Periods (1960-88)

The next step in this study was to extend the analysis to identify those periods in the past when the dates of high flows were not known (i.e., from 1962 to 1981). There are precipitation data for these years, however, making it possible to locate these high flow periods by simulating the complete 20-year sequence.

This simulation process often clearly identified one flow period as the time of maximum discharge in each 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. In other years, which had two or more occasions when high flows were similar, all similar high flow occasions were retained for subsequent modelling to ultimately select the highest flow period.

Table 5-4 summarizes all of the anticipated high flow occasions from 1962 to 1988 in the Little Codroy Valley. As there is historical flood data to suggest that these dates were also high flow periods or the Grand Codroy River, these flow periods were selected for simulating flood flows on the Grand Codroy River.

5.6 Hydrologic Modelling - Grand Codroy River

5.6.1 Initialization

The calibrated parameters from the Little Codroy model were employed as input to the Grand Codroy model, as they represent reasonable estimates. The values are shown in Table 5-3. Routing reaches and flow summing points for the full basin (shown in Figure 5-1) were then assembled and are shown in schematic form in Figure 5-7.

5.6.2 Simulation of High Flow Periods (1961-1968)

The calibrated/verified model parameters were employed without adjustment for simulating streamflow in these years. Precipitation data was taken from St. Andrews, Stephenville and Doyles, and API values were taken from earlier runs to initialize the model.

TABLE 5-4

HISTORICAL FLOOD FLOW PERIODS CODROY RIVER AREA

Year	Date (Rainfall Event)	Date (Rainfall-Snowmelt)
1962	early December*	-
1963	early November	mid May
1964	late November, mid October	mid April
1965	mid November	-
1966	late October, early November	-
1967	late November, early October	-
1968	mid November	mid December*
1969	-	mid March**
	-	mid May
1970	mid August	-
	mid November	-
1971	mid October, mid September	mid April
1972	mid November, early October	mid May
	early December**	-
1973	early December	early May
1974	early June, or September	-
	early December	-
1975	late November	late December*
1976	-	early May**
1977	October-December	-
	August	-
1978	early June	mid January**
	-	early May
1979	December or July	early March
1980	early October	late April
1981	early December	early February**
	late November	
		late April[1]
1982		late April[1]
1983	-	early January[1]
	-	late April
1984	early September	early April[1]
1985	early June[1]	April-May**
1986	mid October	late January[1],**
1987	late October[1]	early April
1988	-	mid March[1]

\- no high flow period projected
* some snowmelt with rainfall
[1] same date as WSC observed peak flow
** historical flood flow period

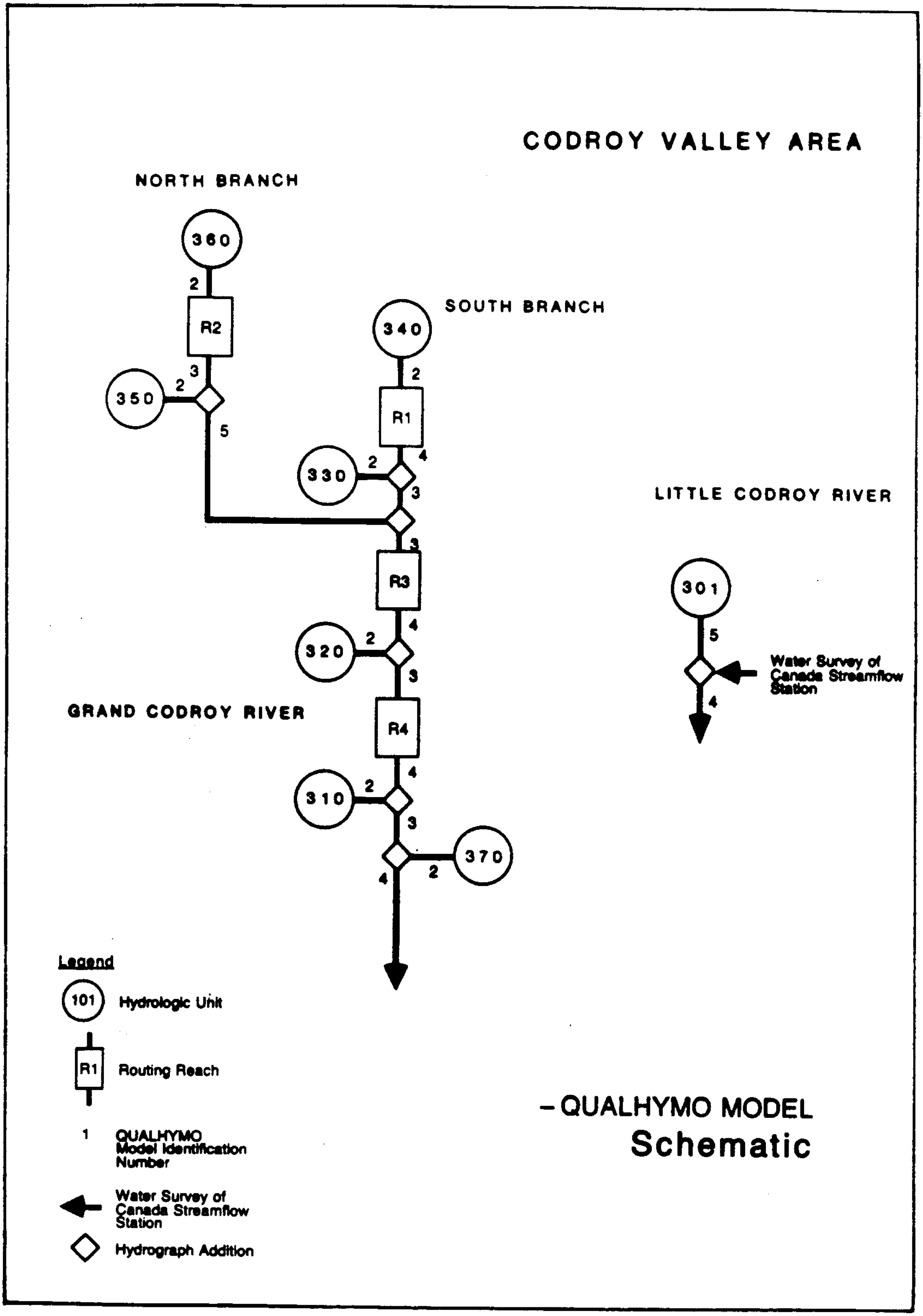

CODROY VALLEY AREA
NORTH BRANCH
360
2
R2
3
2
350
5
SOUTH BRANCH
340
2
R1
4
330
2
3
3
R3
4
320
2
3
GRAND CODROY RIVER
R4
4
310
2
3
370
2
4
LITTLE CODROY RIVER
301
5
Water Survey of Canada Streamflow Station
4
Legend
101 Hydrologic Unit
R1 Routing Reach
1 QUALHYMO Model Identification Number
Water Survey of Canada Streamflow Station
Hydrograph Addition
- QUALHYMO MODEL
Schematic

FIGURE 5-7

Each possible high flow period listed in Table 5-4 was simulated and the peak flow for each year are listed in Tables 5-5a and 5-5b at several locations of interest.

5.7 Frequency Analysis of Flood Flows

5.7.1 Analysis of Modelled Data

Values in Table 5-5(a) and Table 5-5(b) illustrate only the results at several points in the watershed. Similar results were also derived at all 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.

The annual flood flows at each of the flow points were subject to frequency analyses to determine the 1:20 year and 1:100 year open water 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 Person Type III (LP3)
- Wakeby Distribution (Wakeby).

The frequency analysis results for the above distributions are summarized in Table 5-6. 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, however, the 3PLN distribution provides the best fit to the data (both graphically and statistically), and is the distribution which is best suited to match the regional statistics of other nearby rivers in Southwest Newfoundland. This latter consistency with other rivers in the region 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 Codroy Valley area.

TABLE 5-5(a)

SUMMARY OF SIMULATED PEAK FLOWS CODROY VALLEY AREA (1962-87)

	LITTLE CODROY		NORTH BRANCH*		SOUTH BRANCH*	
Year	Month	Flow	Month	Flow	Month	Flow
1962	12	232.07	12	448.51	12	404.91
1963	11	161.69	11	336.82	11	295.66
1964	10	93.82	10	179.44	10	182.58
1965	11	164.96	11	337.80	11	284.54
1966	10	70.29	10	147.37	10	136.03
1967	10	113.36	10	254.58	10	213.73
1968	11	199.52	11	390.51	11	353.14
1969	5	101.67	11	348.36	5	287.16
1970	7	75.12	8	147.93	8	141.91
1971	9	110.21	9	190.45	10	193.20
1972	10	171.30	10	293.80	10	274.92
1973	8	224.01	6	414.00	8	359.73
1974	9	169.94	9	309.76	9	301.76
1975	11	157.62	11	288.32	11	265.72
1976	12	147.34	12	333.08	12	218.81
1977	12	171.36	8	232.42	8	215.31
1978	1	111.26	1	358.67	1	341.86
1979	7	224.83	7	526.87	7	426.25
1980	10	215.76	10	382.21	10	344.96
1981	11	155.98	11	315.69	11	272.96
1982	4	182.70	4	397.91	4	290.63
1983	1	85.60	1	141.19	1	111.06
1984	4	107.00	4	216.01	4	163.12
1985	6	121.97	6	141.33	6	137.38
1986	1	211.87	1	365.22	1	282.68
1987	10	180.50	10	296.52	10	284.90

* flows on North and South Branch Codroy River just above their confluence near South Branch

TABLE 5-5(b)

SUMMARY OF SIMULATED PEAK FLOWS CODROY VALLEY AREA (1962-87)

	NORTH AND SOUTH CONFLUENCE		AT UPPER FERRY BRIDGE	
Year	Month	Flow	Month	Flow
1962	12	822.31	12	869.18
1963	11	614.41	11	652.01
1964	10	343.83	10	370.73
1965	11	612.08	11	665.14
1966	10	275.47	11	296.82
1967	10	436.69	10	525.72
1968	11	730.64	11	815.89
1969	5	630.18	5	682.16
1970	8	270.70	8	295.10
1971	10	353.22	10	373.92
1972	10	530.42	10	558.54
1973	8	740.17	6	830.03
1974	9	584.74	9	619.73
1975	11	540.64	11	584.06
1976	12	490.93	12	527.14
1977	8	424.24	8	448.13
1978	1	701.80	1	806.99
1979	7	946.48	7	1012.85
1980	10	694.33	10	732.34
1981	11	562.25	11	596.20
1982	4	679.98	4	825.00
1983	1	244.35	1	276.88
1984	4	359.13	4	476.75
1985	6	258.80	6	277.30
1986	1	614.10	1	710.60
1987	10	542.13	10	605.60

Figure 5-8 shows the excellent fit of the 3PLN distribution to the flows at Upper Ferry, for example, and the statistics and curves for the other locations are provided in the Technical Appendix.

5.7.2 Comparison with Other Data

The first comparison was relatively direct, and made possible by the presence of six years of streamflow data on the Little Codroy River. Although this record length of peak flows is too short to use with great confidence for projecting return period flood flows, the monitored peak flows can be used to give some measure of the appropriateness of the flood flow statistics derived from simulated flows (Table 5-6).

The comparison involved frequency analysis of the monitored six years of peak flow data, the same six years of simulated flows, and the full 26 years of simulated flows. The following table and Technical Appendix summarize the results for the Little Codroy River:

Frequency of Flood Flows	Observed Data (6 yrs)	Simulated Flows (6 yrs)	Full 26 years of Simulated Flows
1:20 yr	235 m^3/s	229 m^3/s	234 m^3/s
1:100 yr	254 m^3/s	259 m^3/s	263 m^3/s

All data sets give flood flow frequency values which are very similar, and suggest from this first comparison that the simulated peaks (6 or 26 years) produce sample statistics which parallel the observed data in the Codroy Valley area.

The next comparisons involved use of regional flood frequency analyses prepared by the Canada-Newfoundland Flood Damage Reduction Program (1984) and by the Newfoundland Department of Environment and Lands (1989). The report and users' guide from the 1984 study, for example, provide a procedure for estimating peak flows based on physiographic and hydrometeorlogical characteristics of a watershed.

Application of the regional flood frequency analyses approaches at several points in the Codroy Valley area are detailed in the Technical Appendix and give flood flow estimates which are shown

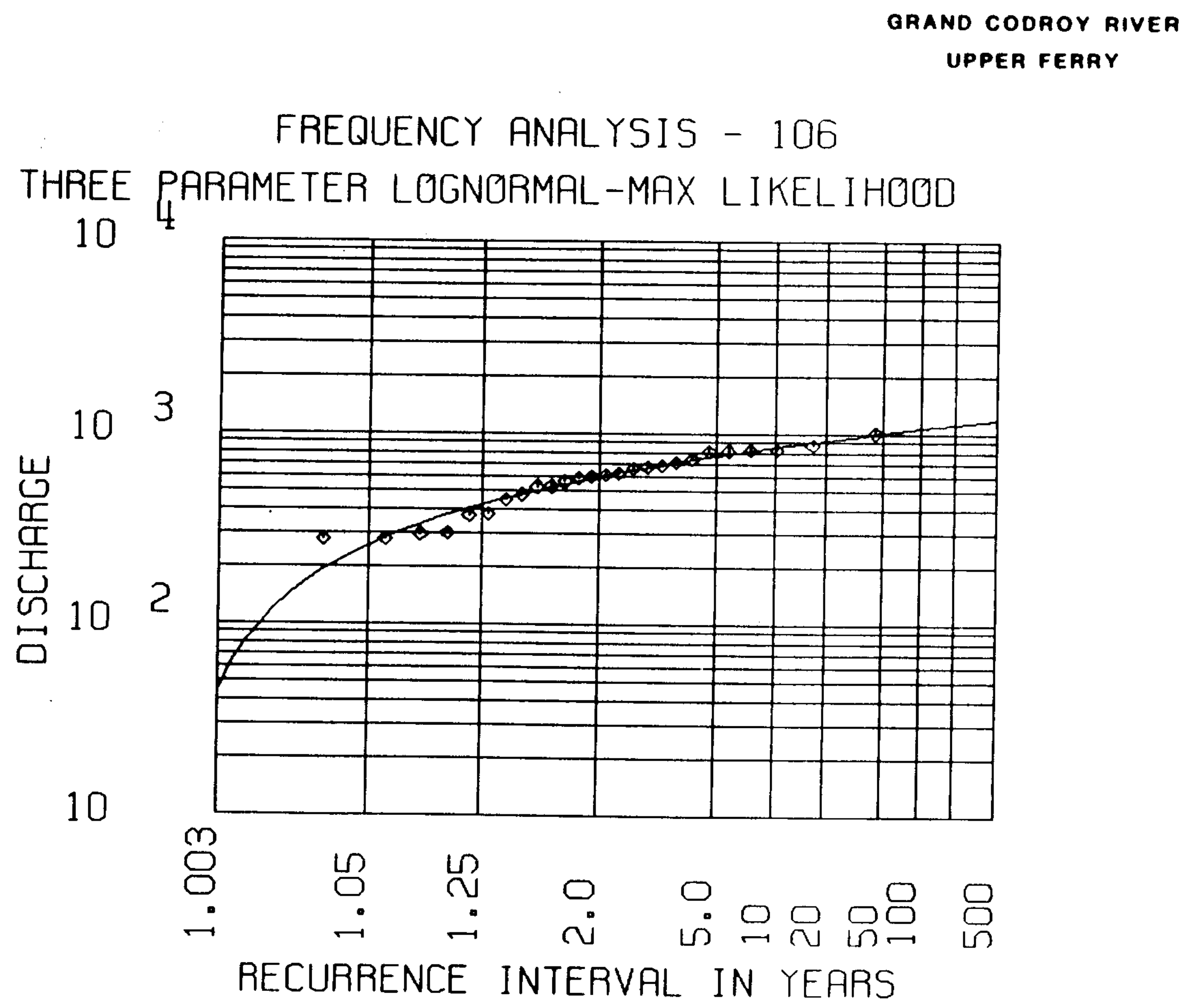

GRAND CODROY RIVER
UPPER FERRY
FREQUENCY ANALYSIS - 106
THREE PARAMETER LOGNORMAL-MAX LIKELIHOOD
10^4
10^3
10^2
10
DISCHARGE
1.003
1.05
1.25
2.0
5.0
10
20
50
100
500
RECURRENCE INTERVAL IN YEARS

FIGURE 5-8

TABLE 5-6

FLOOD FREQUENCY ANALYSIS CODROY VALLEY AREA

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

Location	Generalized Extreme Value	3PLN Distrib.	Log Pearson Type III	Wakeby Distrib.
North Branch	468	471	469	423
South Branch	394	400	395	392
Confluence N & S Branch	842	851	843	790
Upper Ferry	916	929	918	923

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

Location	Generalized Extreme Value	3PLN Distrib.	Log Pearson Type III	Wakeby Distrib.
North Branch	526	541	525	488
South Branch	435	456	433	511
Confluence N & S Branch	952	990	949	1010
Upper Ferry	1020	1070	1020	1160

in Table 5-6 (a). These values are generally slightly lower or higher than those developed by frequency analysis using 26 years of modelling data. The frequency analysis results are extremely close to the results projected from the regional analysis, however, and confirm the appropriateness of the flood frequency analysis using the simulated flood flows.

It is also appropriate to note the very wide range in the Regional Analysis (1984) estimates of flood flow for the South Branch. This range results from the unusual physical characteristics of the branch as compared to other watersheds used in developing the regional analysis:

- The area controlled by lakes and swamps (ACLS) in the South Branch is marginally below the limits of use for estimating flood flows in the Southern Region (i.e. 683 or 947 m^3/s may be inaccurate/too high).

- The mean annual runoff (MAR about 2250 mm) in the South Branch is slightly higher than is recommended for use in whole - island estimates of flood flow. Hence, 143 or 176 m^3/s may also be inaccurate/too low.

In all, the simulated flood flow frequencies for the South Branch (i.e. 3PLN) bridge these constraints by providing flow estimates which lie in the middle of the range of the regional estimates, and which are generated by 26 years of local precipitation data on this watershed.

One previous hydrologic study has also made estimates of flood flows on the Grand Codroy River. This work (Fenco-Newfoundland, 1979), which was part of engineering investigations which followed the collapse of the Grand Bay Bridge, employed a regional flood frequency analysis developed in the late 1960's (Poulin, 1971).

The approach is now well outdated and has been replaced by the recent regional analysis for the Province (1984). However, the results were based on available watershed characteristics and data from that time, and give the following flood flows at the Upper Ferry bridge ("previous") as compared to lower, more recent estimates ("current") from the 1984 regional analysis:

TABLE 5-6(a)

FLOOD ANALYSIS CODROY VALLEY AREA

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

Location	Generalized Extreme Value	3PLN Distrib.	Log Pearson Type III	Wakeby Distrib.	Regional Analysis (1984)	(1989)
North Branch	468	471	469	423	411	512
South Branch	394	400	395	392	143-683	386
Confluence N & S Branch	842	851	843	790	902	---
Upper Ferry	916	929	918	923	971	---

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

Location	Generalized Extreme Value	3PLN Distrib.	Log Pearson Type III	Wakeby Distrib.	Regional Analysis (1984)	(1989)
North Branch	526	541	525	488	536	611
South Branch	435	456	433	511	176-947	473
Confluence N & S Branch	952	990	949	1010	1192	---
Upper Ferry	1020	1070	1020	1160	1265	---

- regional analysis not applicable for this drainage area

	Daily Flow (Previous)	Peak Instantaneous Estimates (Previous)	(Current)
1:20 year	906 m^3/s	1130 m^3/s	971 m^3/s
1:100 year	1290 m^3/s	1614 m^3/s	1265 m^3/s

As noted above, the newest regional approach is considered to give more accurate flood values than these previous estimates (the recent Regional estimates are 14-22% lower than the earlier ones for this watershed area).

The last comparison was developed by simulating rainfall runoff from the 1:20 and 1:100 year storms over the Codroy Valley area. Rainfall data was taken from the intensity-duration-frequency record at Stephenville Airport (Environment Canada, 1987), and the rainfall was distributed over a 12-hour period using the AES Time Distribution developed by Hogg (1982). The 12-hour duration closely corresponds with the time of peak concentration for the watershed and, the hourly distribution and rainfall totals are as follows:

RAINFALL TOTALS (Stephenville)

12 hr	20 Year	98.1 mm
12 hr	100 Year	120.6 mm

12-HOUR STORM

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%

The rainfall totals at Stephenville were prorated to each subcatchment in the study to account for geographical differences (as discussed in section 5.5.3), and the antecedent moisture for each subcatchment was set at an average value (AMC II). The following Table summarizes the rainfall runoff results:

12-HOUR STORM RAINFALL RUNOFF CODROY VALLEY AREA

Location	RETURN PERIOD (YEARS) 1:20 Year Flow (m^3/s)	1:100 Year Flow (m^3/s)
North Branch	355	470
South Branch	451	578
N & S Confluence	804	1045
Upper Ferry	846	1098

These results provide an important addition to the frequency analyses conducted earlier (Table 5-6). Many of the values given earlier in Table 5-6 are derived from snowmelt with moderate rain, or long duration but low intensity rainfalls. Those combinations adequately describe the flood flow series for more frequent events, but slightly underestimate flood flows (in parts of the Codroy Valley) for less frequent events, such as the 100 year flow.

5.7.3 Selection of Design Flows

Table 5-7 summarizes the Regional, Frequency Analysis, and Storm Rainfall results for comparison. As noted above, frequency analysis results using the Three-Parameter Log Normal distribution (3PLN) provide the best fit to the data, match regional statistics, and are also a distribution which is consistent with that selected by the Canada-Newfoundland Flood Damage Reduction Program for general flood studies in Newfoundland. Hence, the frequency analysis results have been selected for determining open water flood conditions in the Codroy Valley Area for most locations at the 1:20 year level and at one location for the 1:100 year case.

However, the frequency analysis results (3PLN values) are slightly exceeded by flood flows which result from design rainfall storms. These storm flows exceed the frequency analysis results at several locations for the 1:100 year flow and at one location for the 1:20 year flood flow. The storm flows are not generally too much greater than the 3PLN frequency flows, but because those that are greater and equally probable, that have been selected for design purposes.

TABLE 5-7

SUMMARY - DESIGN FLOWS
CODROY VALLEY AREA

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

Location	Regional Analysis** (1984)	(1989)	3PLN Distrib.	Storm Rainfall	Selected Design Flow
North Branch	411	512	471	355	471
South Branch	±410	386	400	451	451
N & S Confluence	902	---	851	804	851
Upper Ferry	971	---	929	846	929

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

Location	Regional Analysis** (1984)	(1989)	3PLN Distrib.	Storm Rainfall	Selected Design Flow
North Branch	536	611	541	470	541
South Branch	±560	473	456	578	578
N & S Confluence	1192	---	990	1045	1045
Upper Ferry	1265	---	1070	1098	1098

-- regional analysis not suitable for this drainage area

** for general comparison purposes, only to confirm the appropriateness of the results of the other detailed analyses

In all, the right column of Table 5-7 summarizes the 1:20 year and 1:100 year design flood flows derived from the Three-Parameter Log Normal Distribution or design rainfall storms at all points along the Grand Codroy River. These values are used in the following section to derive 1:20 year and 1:100 year open water flood profiles.

6.0 HYDRAULIC ANALYSIS - DATA COLLECTION

6.1 General

The purpose of the hydraulic investigation was to derive the 1:20 and 1:100 year flood level profiles along the study area - considering open water flooding and the potential for ice-related problems. This initially involved data collection followed by analysis of open water flood levels and flood levels with ice.

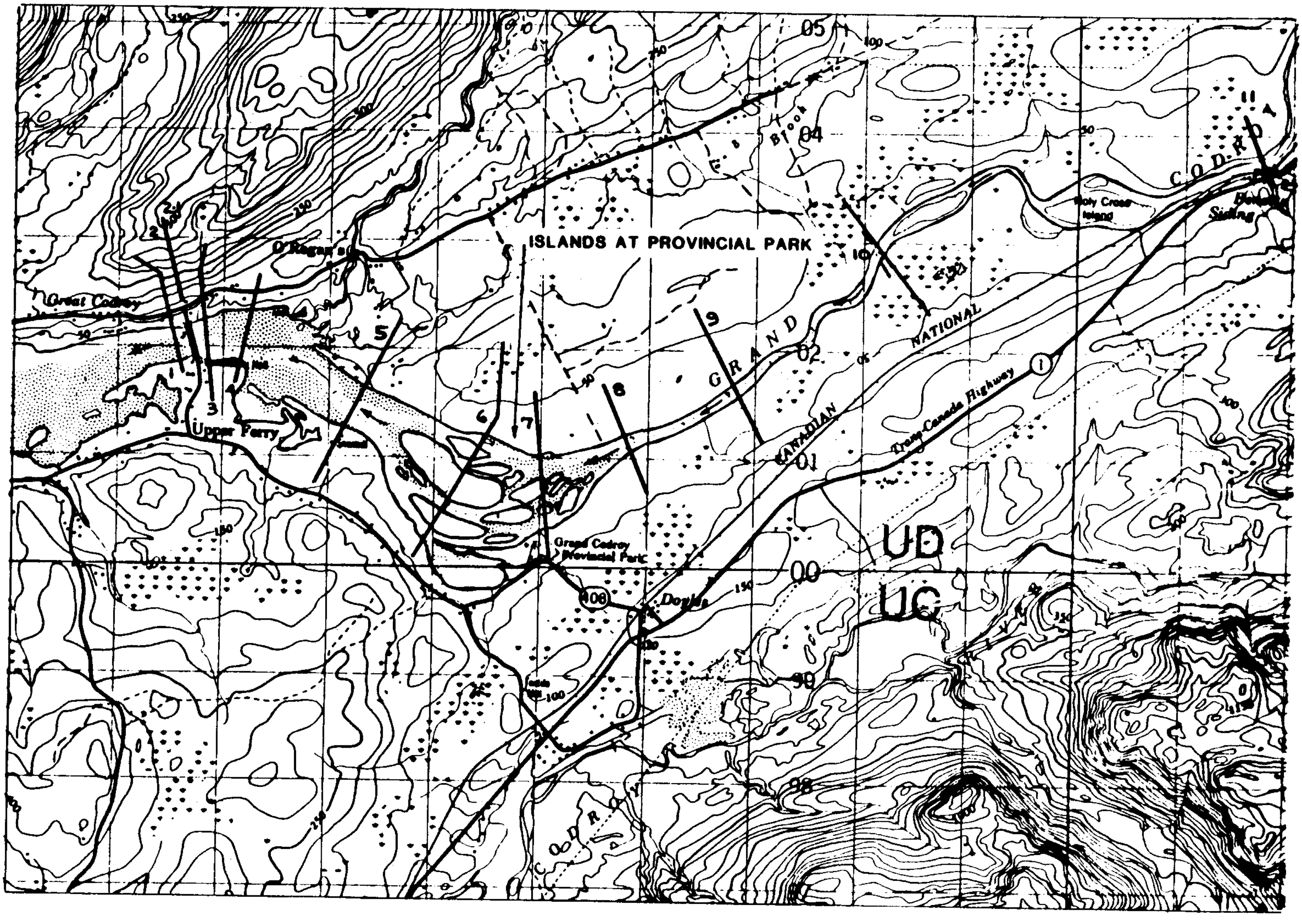
6.2 Channel Geometry

Both the open water and ice hydraulic analyses require river cross-sectional data obtained from field surveys. These were undertaken during the summer of 1988 and the surveys included measurements of representative channel cross-sections, structures across the watercourse, and photographs at each section. These data are provided in the Technical Appendix.

A total of 28 representative channel and floodplain and structure cross-sections were surveyed along the Grand Codroy River from Trans-Canada Highway Bridge near Coal Brook to the Grand Bay Bridge at Upper Ferry. These sections included all details of the three bridge structures because construction drawings were not available. The total distance surveyed along the river was about 29 km.

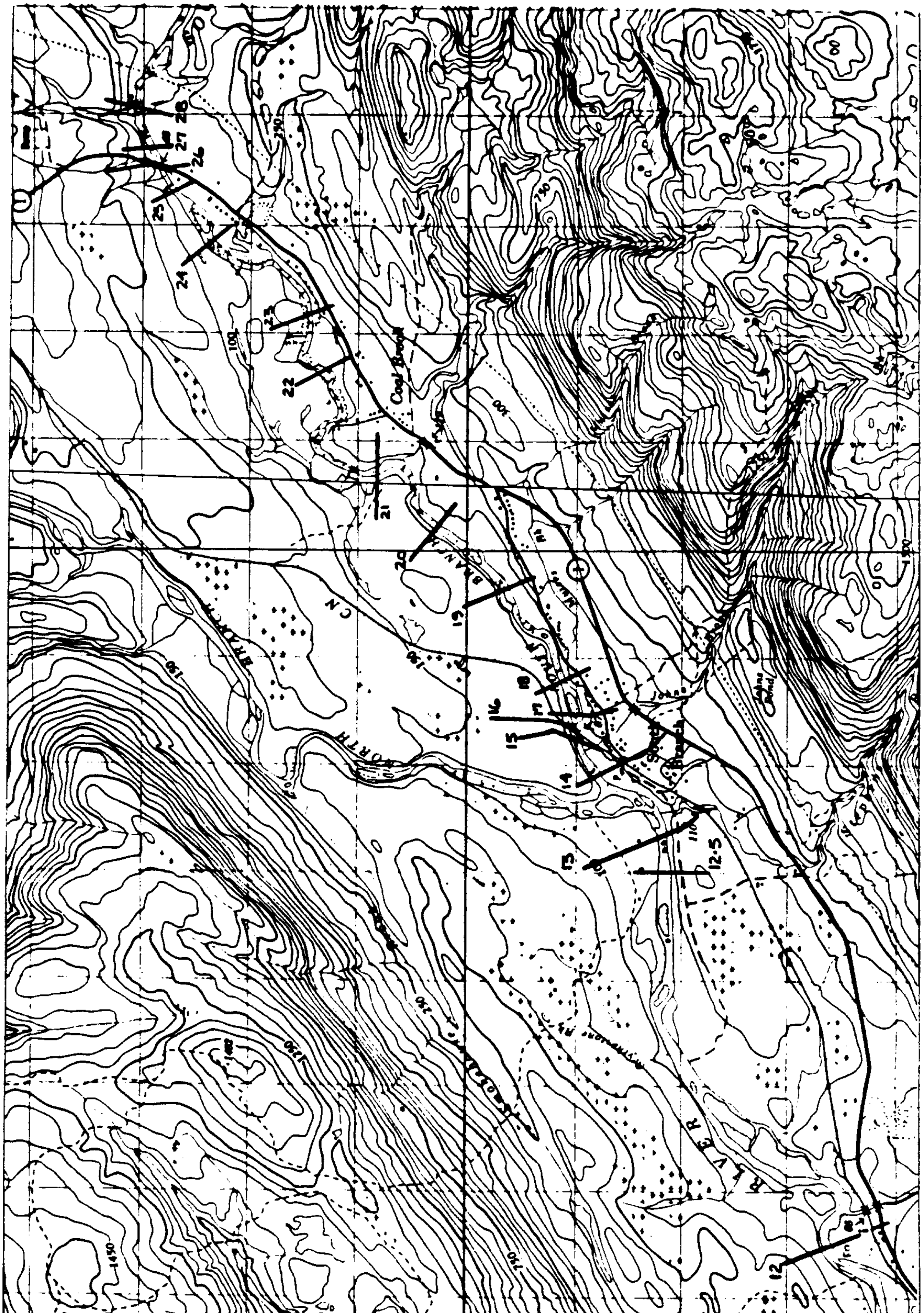
The cross-sections were surveyed above and below the waterline and beyond the top of bank, and each cross-section was referenced to geodetic datum. The approximate locations of the cross-sections are shown in Figures 6-1 and 6-2, and their precise locations are plotted on 1:2500 scale flood risk maps. The field cross-section information at some stations was supplemented with data from the 1:2500 scale topographic mapping (with 0.5 m contour intervals) 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.

Initially, the structures across the watercourse were coded for hydraulic modelling using the guidelines outlined in the HEC-2 Users Manual (HEC, 1982). For example, the skewness of the structures was taken into account in the coding where the structure is not perpendicular to the flow. Then, all the cross-sections were coded adopting the convention of looking downstream from



SURVEY SECTIONS IN LOWER REACH

FIGURE 6-1



SURVEY SECTIONS IN UPPER REACH

FIGURE 6-2

left to right bank, and all the sections are referenced to Geodetic Survey of Canada (GSCD) datum.

The Manning roughness coefficients for each cross-section were estimated in the field, from photographs taken of each section during the field work and from available mapping, but some of the values were subsequently adjusted during the calibration of the hydraulic model (plots and photographs of all cross-sections and bridge/culvert structures are presented in the Technical Appendices). The Manning's roughness coefficients for the left and right overbanks and channel are shown in the above-noted plots, as are any portions of the cross-sections supplemented with topographic mapping.

6.3 Starting Water Levels

The starting water level for hydraulic modelling of the Grand Codroy River is sea level at the river mouth. Water levels appropriate for this study have been recorded for many years at nearby Port aux Basques, but it is only since 1961 that maximum instantaneous values have been taken by the Marine Environmental Data Service (MEDS).

The average high tide level (average of large tides and mean tides) at the chart datum employed by MEDS is 1.80 m (Fisheries and Oceans, 1988). This datum at Port aux Basques is 0.869 m below the geodetic datum employed for this study. Hence, the average high tide level is 0.93 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.

The instantaneous water levels (no wave effect) at the river mouth 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 Codroy River Mouth	Instantaneous Level (MEDS datum)	Instantaneous Level (GSCD)
1:20 year	2.59	1.72 m
1:100 year	2.71	1.84 m

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

6.4 Historic Flood Levels

Part of field data collection involved interviews with local residents. These interviews were initiated to determine:

- historical high water levels
- information about the cause of flooding
- information about ice and open water flood processes.

Table 6-1 summarizes the flood level data collected during two separate series of interviews at the site. The following section outlines the findings from these surveys about ice related flood processes.

6.5 Ice-Related Flooding Processes

The history of flood-related problems in the Codroy Valley is dominated by occasions when break-up ice was a major contributor (Table 3-1). It also seems that the regular cycle of winter and spring flooding is respected by the riverside residents who wisely use the floodplain for agriculture purposes, such as pasture or haying, rather than for permanent dwellings.

Because of the absence of permanent homes along much of the course of the river (and those that are, are located above the limits of previous floods), there is little known about average and extreme flood conditions. Typical ice conditions and water levels, the frequency of flooding, the locations of ice jams, ice thickness and high water levels are not known at most locations. There

TABLE 6-1

OBSERVED ICE/FLOOD LEVELS

X-Section #	GSC Elevation (M)	Ice/Open Water	Description/Date of Occurrence
2	low/unknown	water/ice	January 1978
3	.895	Open Water	1982 spring melt level
4	1.380	Ice	Ice scar
7	1.241	Ice	Spring melt 1986
7	1.971	Open Water	January 1986
10	10.08	Ice	Ice scar (shove)
10-11	7.670	Open Water	September 1987
11	9.931	Open Water	High water level mark
13	24.43-25.43	Open Water	January 1986
13-14	24.43-25.43	Open Water	January 1986
13-14	24.284	Open Water	January 1986
13-14	24.608	Open Water	January 1986
13-14	23.651-24.00	Open Water	April 1982
14-15	25.098	Open Water	January 1986
16	25.50	Open Water	January 1986
18-19	25.5-26.0	Open Water	Annually
18	30.661-31.004	Open Water	October 1987
19	28.4	Open Water	April 1982/August 1982
19-20	29.93	Open Water	January 1986
20	29.9	Open Water	April 1982
20-21	30.50	Open Water	January 1986
21	32.20	Ice	January 1986
24	38.95	Ice	1948/50
24	39.50	Ice	1987
25	40.45	Water/Ice	1987
26-27	below 40.90	Water/Ice	1987
27	40.5	Water/Ice	February 1981
27	40.8	Ice	March 1988

is, however, some data from observation points at river crossings (e.g. TCH bridge near Coal Brook) and from some residents.

General

The annual sequence of break-up is reported to result in ice-related flooding problems which vary in their significance from year to year (Vasallo 1988 p.c.). The sequence can be characterized as progressive break-up, initiated by winter rainfall/snowmelt, moving from upstream to downstream.

Most often, the ice cover is raised and broken as it is carried downstream during brief, mid-winter runoff periods. These periods appear to generate insufficient flow to completely clear the river of ice and, thus, the ice from early break-ups often remains stranded along the river channel until later, larger flood flows.

It appears that the ice which is stranded in this way refreezes into a consolidated obstruction. This blocks the free passage of ice and melt water from subsequent rainfall/snowmelt occasions and is reported to cause flooding at the TCH bridge near Coal Brook. These upstream obstructions may benefit downstream locations, however, as the years which have such blockages result in a quiet melt of river ice at the mouth.

TCH Bridge-Coal Brook

This location provides a vantage point for observing the annual variability in ice conditions in the upper river system. Break-up seems not to have been a noteworthy problem until 1981, 1986 and 1988 when the ice accumulations at the bridge were reported to be a concern.

Records and photographs suggest that ice jam floods have been initiated at two locations - one just below the TCH bridge, and another about 100 m below that. As noted in 1986, however, a jam may also form about 1 km below the bridge, resist the flood forces for a while, and then break and surge down-stream to another blockage site.

In 1981, it is estimated that the ice accumulation at the bridge was about 3.6 m thick (12 feet). This is exceptionally thick and the blockage is usually thinner and relatively easy to disrupt by natural forces, or by blasting if required (Greer, p.c. 1988).

The extent of flooding immediately upstream of the TCH bridge near Coal Brook can only be estimated from ice scars on the trees along the river. These are generally in the range of about 2.1 m (7 feet) above low flow water levels - or slightly lower than the 2.8 m rise during 1988 ice jam conditions at the bridge (Vassalo, 1988).

South Branch Area

There is not much information about ice and flood conditions between the TCH bridge and South Branch, except for the valuable observations of a local resident, Mr. Gale, and the Newfoundland Department of Environment in 1986. It is clear from these observations that there were a series of temporary obstructions and releases as the ice moved downstream toward the community of South Branch.

At South Branch itself, there is a railroad crossing at a constriction and bend in the river. This appears to be a location where there could be frequent ice problems, but this was not confirmed by a local resident (Mr. Brake). It was his observation that there had only once been an ice problem at that location in the past 60 years. This was the winter of 1948-49, when there was a temporary lodgement at the bridge. The blockage may also have been triggered by the narrow span between the bridge piers but the bridge span has been widened since this occasion.

It was also noted that there have been ice blockages at the small bridge over John's Brook, which causes overbank flooding in the area between the rail bridge and road along the south bank.

Just upstream of this location, another observer (H. Sheppard) reported that the road at Muises Brook was overtopped by flood waters in 1986 and reached a level about 0.15 m (6 inches) from her home. It is suspected, however, that this flood originated from Muises Brook and not from the main river.

Grand Codroy Provincial Park

There are few observation of break-up flooding and ice effects in this section of the river because park occupancy is seasonal. However, the Park Warden recalls that ice and high flows raised water levels to about 2 m above normal levels in April-May of 1985. The river is quite wide at this location and this level of flooding suggests that there may have been an ice blockage on the bars and islands across this area.

Grand Codroy Bridge - Upper Ferry Area

We are fortunate that there are several keen observers of the river in this area. They report that there is an ice cover every year in the tidal pond/ estuary upstream of the bridge. This cover rises and falls with the tide and with changes in streamflow and usually melts/disintegrates in place. About once in 5 years, however, break-up ice from upstream moves progressively downstream as far as the bridge - particularly if the break-up is accompanied by heavy rainfall and unusually high flows. This sequence has not occurred in the most recent 5 years.

The usual break-up situation finds the progression of blockage and back-up followed by release and downstream flow, then re-blockage etc. progressing downstream to destroy the ice cover down to Grand Codroy Provincial Park. There is usually an ice blockage in the shallows at and on the islands at the park (shown in Figure 6-1, cross-section #7). The difference between the "typical" year and other years is what happens next.

In the typical year, the break-up front only reaches the islands and progresses no further downstream. Often (as noted above), this may happen more than once each winter as a result of mid-winter rainfall/snowmelt occasions. It is the experience of local residents that each occasion benefits the downstream area by sweeping the ice from the upstream reaches before it becomes too resistant to hydraulic forces. This sets things up for easy conveyance of subsequent flood flows through the ever widening reach below the islands at the park.

Ice thickness in the area of Upper Ferry is usually about 0.14 m (5-6 inches), but occasionally grows to about 0.46 m (1.5 feet).

The 1978 problem was initiated by a large rain storm which raised water levels throughout the basin and initiated a general downstream ice motion (Gillis p.c., 1988). The progression of break-up began much the same as the typical case and by about 11-11:30 am, the broken ice from upstream had cleared its way downstream to a position about 1.2 km east of the bridge. This continued until about 1:30-2:00 pm when the broken ice was stopped by the downstream ice sheet about 6 m (20 feet) downstream of the bridge. This placed the bridge in the "toe" of the ice accumulation - the unpredictably thick and unstable downstream part of an ice accumulation. All that is known is that the ice pieces which composed this mass were about 0.13-0.15 m thick (5-6 inches) and that water levels were not unusually high.

What is somewhat unusual (about once every 5 years) is that this January break-up was able to break its way through the ice cover at the islands (near the upstream provincial park) and reach the bridge.

At the old bridge, ice usually cleared along the north bank of the river before the south side became ice free. This was not the case in 1978 when, for unknown reasons, the south channel was open first and the north side was still ice covered (or partially blocked by broken ice). It was the only time in the past 40 years that one resident (Joe Martin) could recall this sequence of break-up at the bridge. This blockage, or high flows which were present, resulted in high velocity flow through the open area, formation of a large eddy below the bridge, and the ultimate collapse of one section of the structure at about 4:45 pm that day. The structural integrity of the bridge was broken by the loss of the first span, and other sections failed in a "domino effect" over the following six hours (Martin p.c. 1988).

Another resident (Mrs Martin) reported that the much of the original bridge (circa 1926) had been founded on piles, but that the first two abutments on the south side had their foundations on the bottom overburden - rather than on piles or rock.

6.6 Bottom Sediments

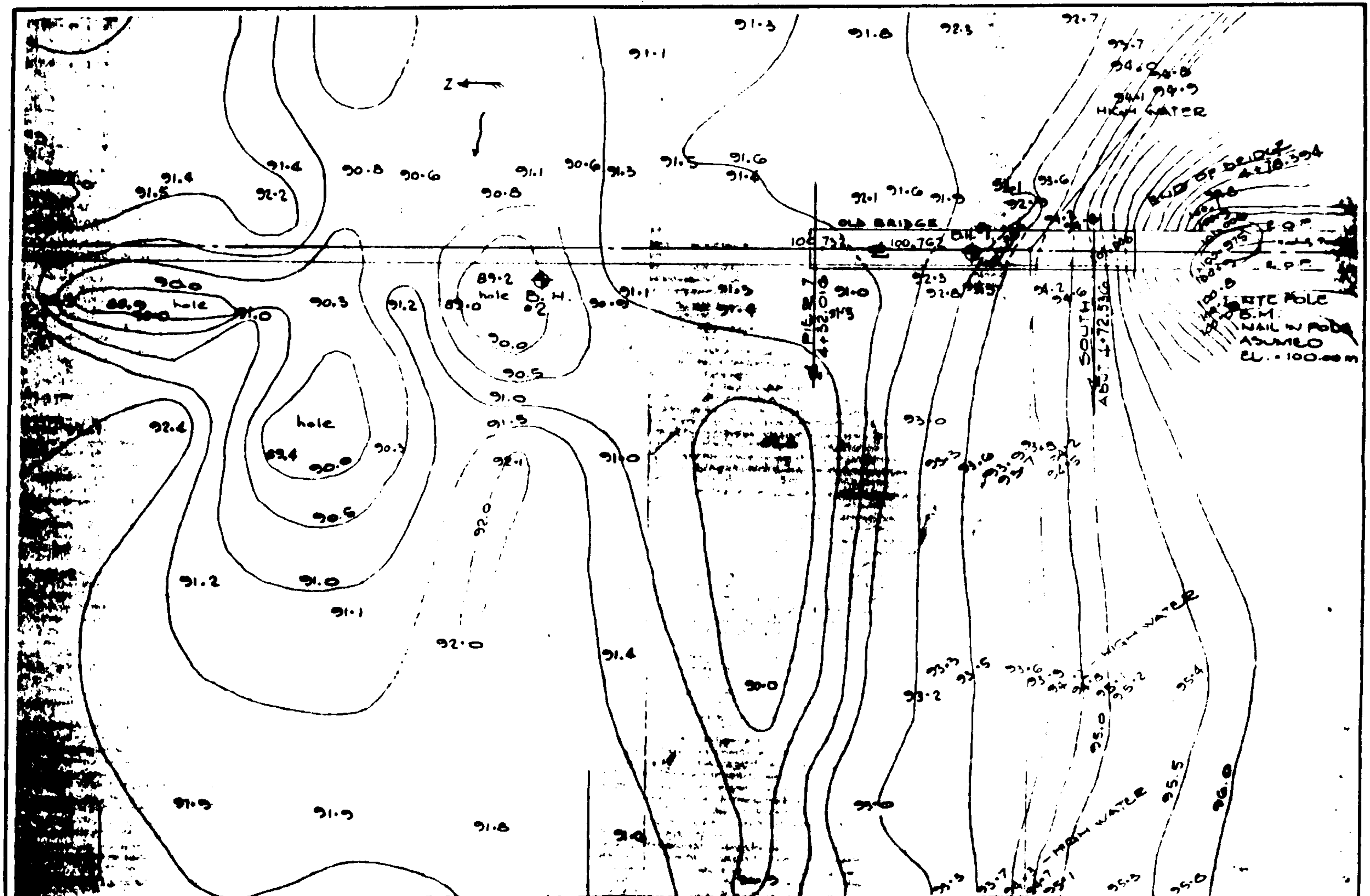
Information about the bottom sediments in the area of the Grand Codroy Bridge is contained in a report titled, "Subsoil Investigation - Proposed Bridge, Grand Codroy River" (Geotechnical

Associates Ltd., 1978). These investigations indicate that the soil consists of silty sand and sandy till which are very susceptible to scour (the median diameter (D50) of the bed material is reported to be 0.45 mm). This material is not normally affected by small tidal currents, but erosion of such medium sands is a factor for consideration during flood flows having velocities greater than about 0.35 m/s (Chow, 1959).

The geotechnical report further indicates that such conditions have occurred in the historic past. There is evidence in bore hole samples of scour and redeposit to a depth of about 9 m below the river bottom at certain locations (to elevation about -10.5 m GSCD), and the report projects that the depth of scour "under adverse conditions" would be about 6 m (-7.5 m GSCD).
Bottom contours and soundings (Fenco-Newfoundland, 1979) support this possibility (Figure 6-3). The Figure shows relatively uniform depths of about 1.1 m below low tide except in the area of the former bridge and just downstream. There, more than a year following the disaster, are three depressions which are about 4 m below the low tide level (to -5 m GSCD). These features may come and go from year to year or be relatively permanent, but, as shown in Figure 6-4, our survey crew also found three depressions with bottom elevations between -4 and -5 m GSCD.

6.7 Tidal Prism

Another aspect of hydraulic interest is the prism of predominantly fresh water which is held in the river during rising tides, and released with the ebb. The volume contained in this prism/wedge above the Grand Bay Bridge has been planimetered from the bridge to Section 9 (Figure 6-1), which is the upstream limit of the influence of normal tides. The mean tide range in this area is 1.1 m giving a normal value of approximately 750,000 m^3 which must leave the area during ebb tide (i.e. from high tide to mean sea level). This typically takes place over a three hour period for an outflow rate of 70 m^3/s. For purposes of this study, the ice processes and flood analyses will include this flow rate in sensitivity and analyses.



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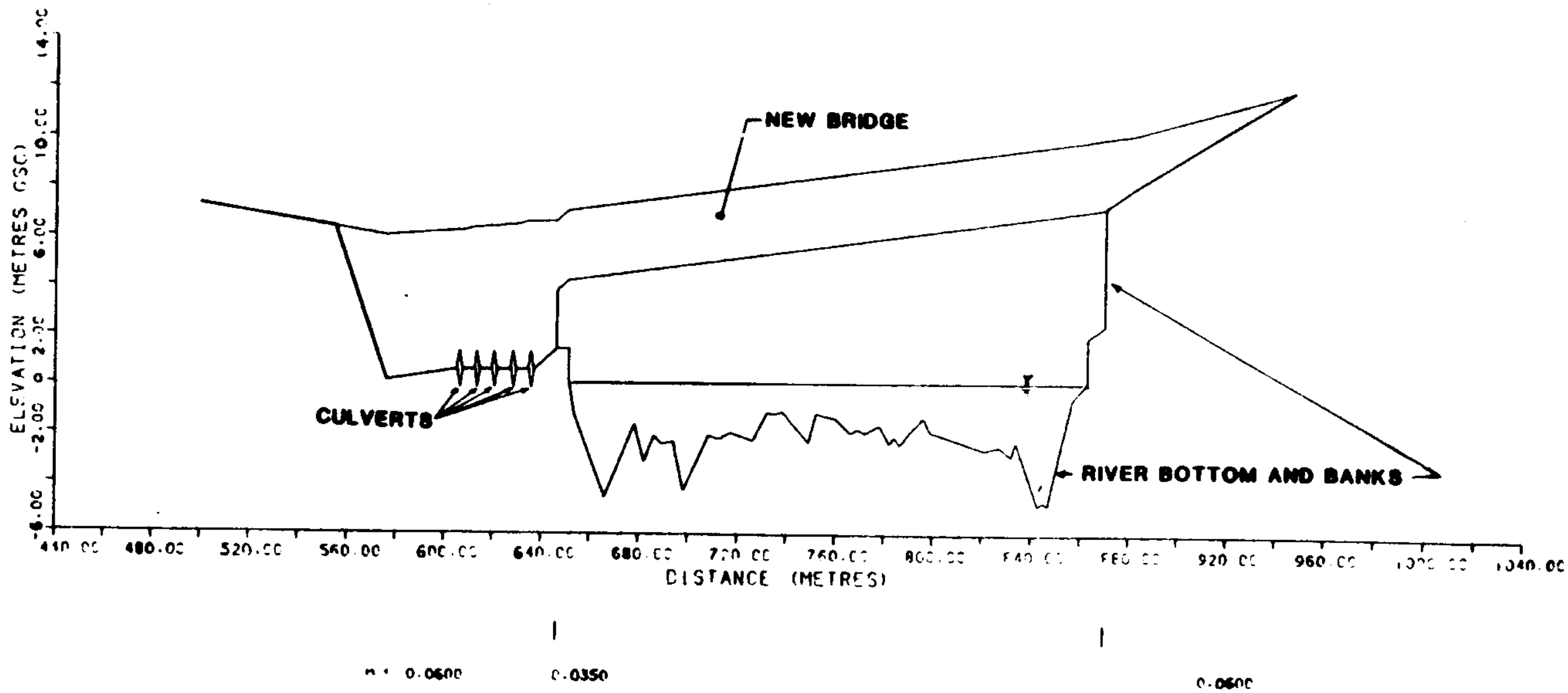
NOTE
· DIMENSIONS AND ELEVATIONS ARE IN METRES.

FIGURE 6-3

CODROY RIVER X-SECTIONS HEC-2 MODEL
CROSS SECTION NUMBER 2.1



GRAND CODROY BRIDGE

VIEW LOOKING DOWNSTREAM

FIGURE 6-4

7.0 HYDRAULIC ANALYSIS - MODELLING

7.1 Introduction

The data presented in Section 6.0 provided information to project ice related flood levels and allowed for development of a mathematical model to give backwater flood levels during open water periods. These flood levels are examined first.

The HEC-2 (HEC, 1982) computer model was used to carry out these investigations. 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 (e.g., the Waterford River and Exploits River), is well-documented, is parameter efficient for calibration, and is flexible in use.

The HEC-2 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 floodplain. 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.

7.2 Model Set-Up

A total of 32 cross-sections were used to model the Grand Codroy River. Six are bridge-related sections and 26 are river sections between the mouth and the upstream study limit. Approximate locations of these sections were shown earlier in Figures 6-1 and 6-2 and their exact locations are shown on the flood profile mapping sheets.

7.3 Calibration and Verification of the HEC-2 Model

The HEC-2 model was calibrated and validated with observed water levels and flow data in order to be confident in the computed water surface profiles. This information was obtained from several sources:

(i) Water level monitoring program undertaken for this study during the October-November 1988 period.

The program involved the Water Survey of Canada (Corner Brook Office) and a local observer who took regular measurements of water levels at the South Branch railroad bridge and the TCH bridge. These observations took place during relatively low flows and are summarized in Table 7-1.

Information was also gathered from local observers who had made note of water levels during other high flow periods. These included:

(ii) January 1986, when open water flood levels were known at cross-section 7 and at several locations near cross-sections 13, 14, 19 and 20 (Table 6-1).

(iii) April 1982, when open water flood levels were pointed out to our surveyors at cross-sections 3, 14, 19 and 20.

The latter two sets of observations were associated with relatively high flows and were selected for initial calibration and verification of channel and overbank roughness. The 1988 monitoring was reserved to verify channel roughness alone.

7.3.1 Model Calibration

The calibration process began with the open water observation from:

- January 1986.

TABLE 7-1

GRAND CODROY RIVER WATER LEVEL MONITORING 1988

Date and Time		TCH Bridge Level		CNR South Branch Bridge	
Oct. 5	7:30 a.m.	9'-4"	7:58 a.m.	20'-2"	normal level
Oct. 6	8:30 a.m.	10'-9"	8:50 a.m.	21'-7"	normal level
Nov. 3	7:30 a.m.	10'-2"	7:50 a.m.	21'-10"	normal level
	3:30 p.m.	11'-11"	3:50 p.m.	23'-8"	normal level
Nov. 18	7:40 a.m.	8'-10"	8:00 a.m.	18'-5"	above normal
	11:15 a.m.	8'-6"	10:50 a.m.	18'-7"	above normal
	3:30 p.m.	8'-6"	2:20 p.m.	19'-10"	above normal
Nov. 29	8:00 a.m.	6'-8"	-	-	above normal
	11:15 a.m.	7'-0"	-	-	above normal
	3:30 p.m.	8'-6"	-	-	above normal
Nov. 30	7:45 a.m.	11'-1"	8:00 a.m.	21'-10"	normal level
Nov. 3	4-5 p.m.*	11.32 ft[a] (3.452 m) 23.711 m[c]		24.50 ft[b] (7.468 m) 38.890 m[c]	

(a) distance below reference elevation 27.163 m (GSCD) on Rail Bridge

(b) distance below reference elevation 46.358 m (GSCD) on TCH Bridge

(c) water surface elevation (GSCD)

* discharge of 51.2 m^3/s at CNR Rail Bridge in South Branch

Levels were known at a number of locations along the river during that year (Table 6-1). The locations included both the lower and upper sections of the study area (sections 7, 13-14 and 19-20), and the flood involved both the channel and overbank areas. Starting water levels were known from tide tables, and flood flows were taken from the QUALHYMO simulation for this period.

The only parameter which was modified during the calibration process was the roughness of the channel and overbank portions of the floodplain. Theoretical values, assigned during model set-up (and summarized in the Technical Appendix), proved relatively good estimates of the actual roughness of the river, but fine tuning was required to bring flood levels into good agreement.

Figure 7-1 plots the channel profile and water level profile resulting from the calibrations. As shown, the fit of the observed and simulated flood levels in the channel and overbanks is quite good and considered acceptable pending verification.

7.3.2 Model Verification

The HEC-2 verification was undertaken using two events:

- April 1982
- November 3, 1988

The first was selected because flood flows again involved both the channel and overbanks and, hence, would verify channel and overbank roughnesses established in the calibration process. The second was chosen to verify that the channel roughness alone was correct. The November 3, 1988 observations (Table 7-1) were during the highest levels and only flow estimate from the surveys in that year.

Figure 7-2 plots the profile for the April 1982 flood which involved the channel and overbank areas. This case again employed simulated (QUALHYMO) flows and good agreement is demonstrated between observed and computed water levels at cross-sections 3, 14, 19 and 20. In this case, it appears that the channel roughness might be slightly adjusted to achieve a better fit at the upper most point (section 19-20). However, because the flood flow itself was simulated and

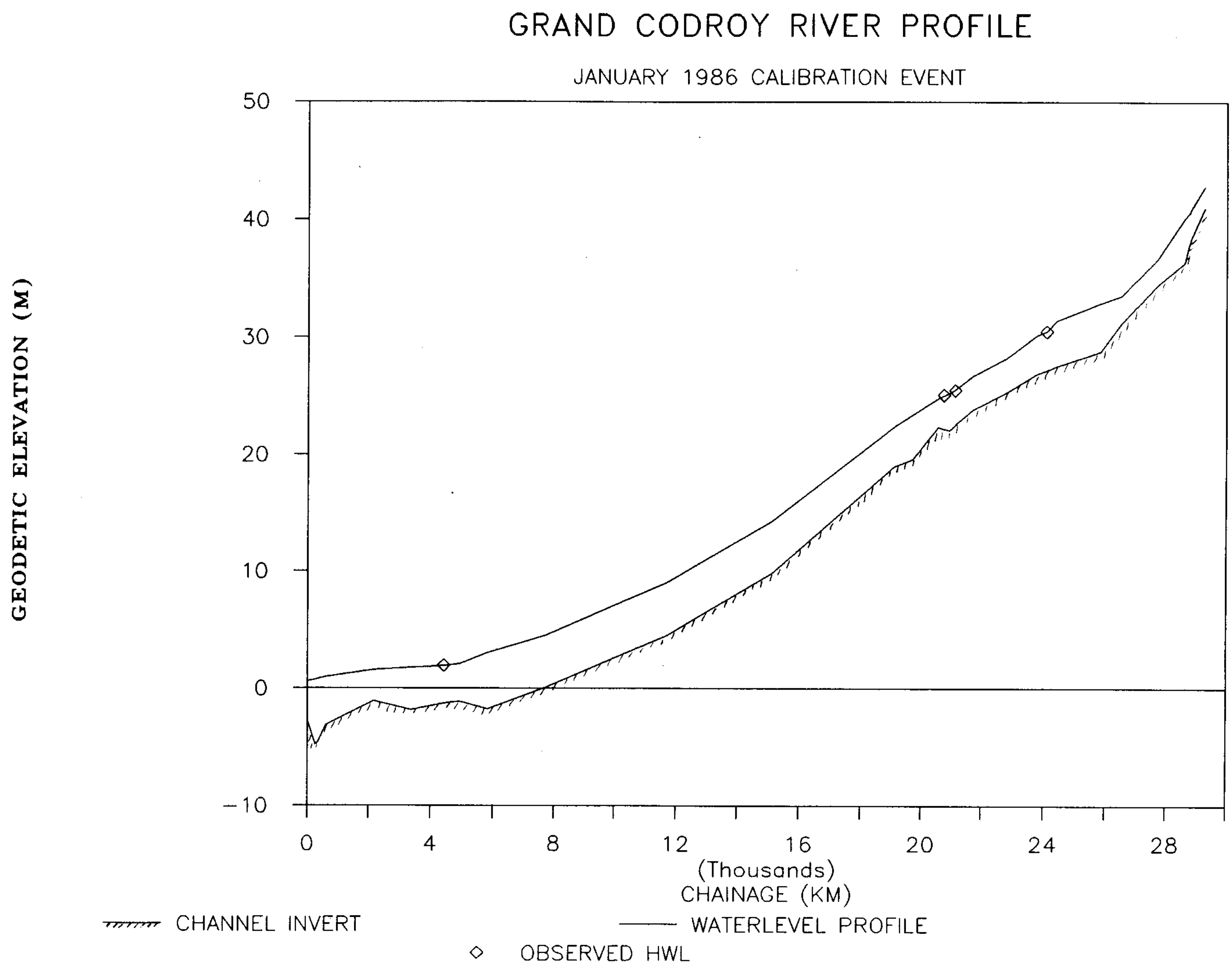

GRAND CODROY RIVER PROFILE
JANUARY 1986 CALIBRATION EVENT
GEODETIC ELEVATION (M)
50
40
30
20
10
0
-10
0
4
8
12
16
20
24
28
(Thousands)
CHAINAGE (KM)
CHANNEL INVERT
WATERLEVEL PROFILE
OBSERVED HWL

FIGURE 7-1

GRAND CODROY RIVER PROFILE

APRIL 1982 VERIFICATION EVENT

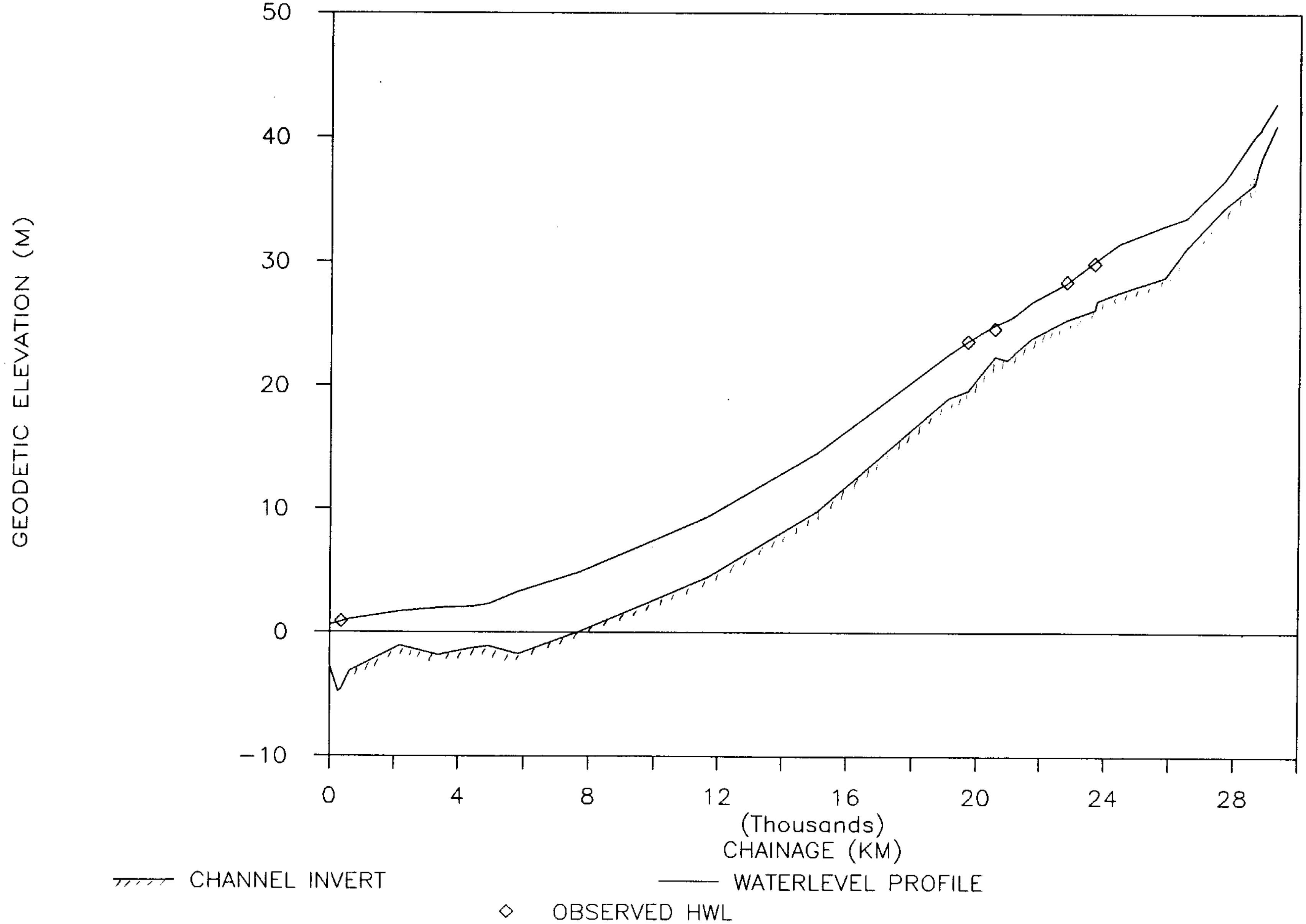


FIGURE 7-2

perhaps slightly high, it was decided to retain the theoretical channel roughness values in this river reach pending final verification results. Difficulties in establishing field estimates of flood levels are reflected at Section 14 where observed estimates are based on top-of-bank levels known to have been exceeded.

Figure 7-3 plots the final verification profile for the upper reach during 3 November 1988 - the only time when flows were accurately measured. The slight discrepancies apparent in the first verification result are no longer present with refined flow estimates (monitored), and the result confirms the accuracy of the selected channel roughness values.

Table 7-2 presents the numerical values of the calibration and verification simulations. It shows that the observed and computed levels are well matched, certainly within the limits of accuracy of the observation estimates, and achievable with roughness values which closely correspond with theoretical expectations (i.e. roughness values from 0.04 to 0.06 in the main channel and 0.05 to 0.1 in the overbanks). The particular roughness for each cross-section and the detailed geometry of each are listed in the Technical Appendix for ready reference.

In all, the calibrated and verified hydraulic backwater model is ready for use in determining flood levels along the Codroy River valley.

7.4 Sensitivity Analysis

Before delineating the flood risk area, a series of simulations were conducted with the calibrated and verified hydraulic model to determine the sensitivity of flood level projections to changes in certain parameters. Included on this list are variations in flood flow, roughness parameters, starting water level (sea level) and tidal flow. All sensitivity tests were conducted with the 20 and 100 year flow rates, and complete results are tabulated in Tables 7-3(a) and 7-3(b) which follow page 7-6.

Tidal Flows

It was noted earlier (in Section 6.7) that an outflow rate of approximately 70 m^3 could be present at Upper Ferry to combine with flood flows during the falling tide. Figures 7-4 and 7-5 show the

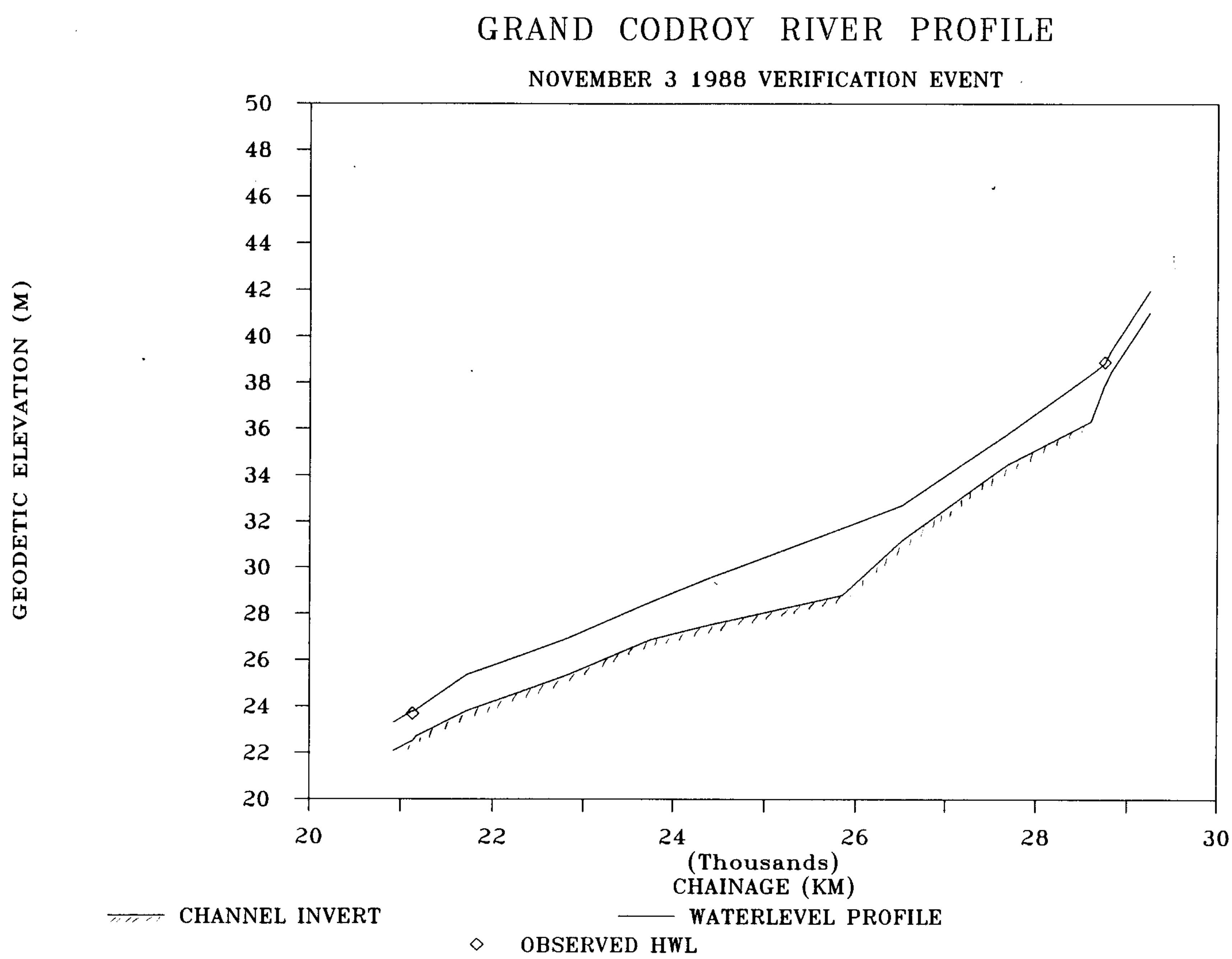

GRAND CODROY RIVER PROFILE
NOVEMBER 3 1988 VERIFICATION EVENT
GEODETIC ELEVATION (M)
50
48
46
44
42
40
38
36
34
32
30
28
26
24
22
20
20
22
24
26
28
30
(Thousands)
CHAINAGE (KM)
CHANNEL INVERT
WATERLEVEL PROFILE
OBSERVED HWL

FIGURE 7-3

TABLE 7-2

Backwater Model Calibration and Verification Events

EVENT	SECTION #	OBSERVED WATERLEVEL (M)	SIMULATED WATERLEVEL (M) *	DIFFERENCE (M)
	3	0.895	0.88	-0.015
APRIL	13	23.60	23.58	-0.02
1982	14	24.60 **	24.95	0.35
	19	28.40	28.30	-0.10
	19-20	29.90 **	30.05	0.15
	7	1.97	1.93	-0.04
JANUARY	14-15	25.10	25.04	-0.06
1986	16.1	25.50	25.57	0.07
	20-21	30.50 **	30.55	0.05
NOVEMBER	16.1	23.70	23.78	0.08
1988	26.1	38.90	38.86	-0.04

* employing flood flow estimates derived from hydrologic simulation (QUALHYMO)

** values aproximate as based on overbank elevation estimates

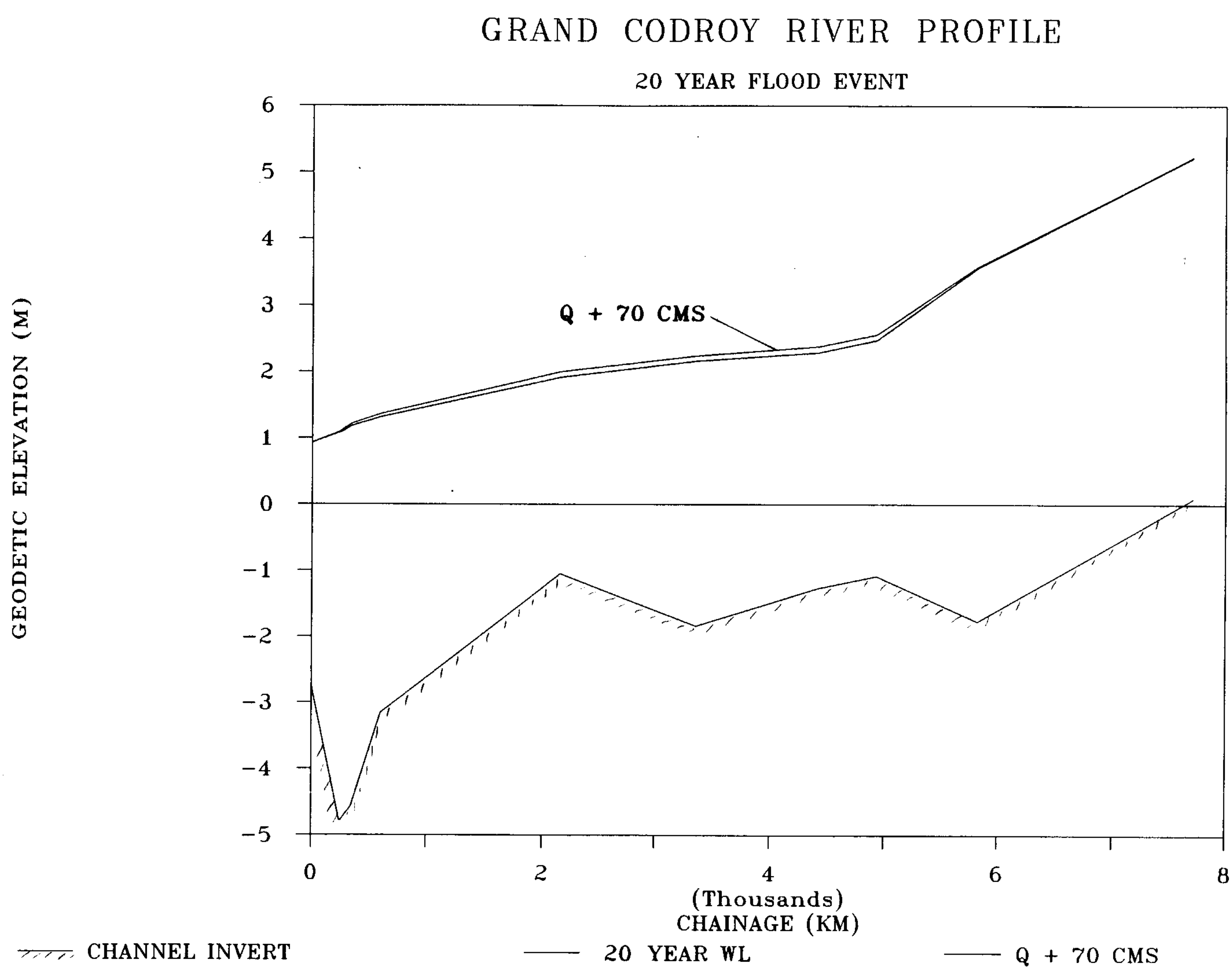

GRAND CODROY RIVER PROFILE
20 YEAR FLOOD EVENT
GEODETIC ELEVATION (M)
6
5
4
3
2
1
0
-1
-2
-3
-4
-5
Q + 70 CMS
0
2
4
6
8
(Thousands)
CHAINAGE (KM)
CHANNEL INVERT
20 YEAR WL
Q + 70 CMS

FIGURE 7-4

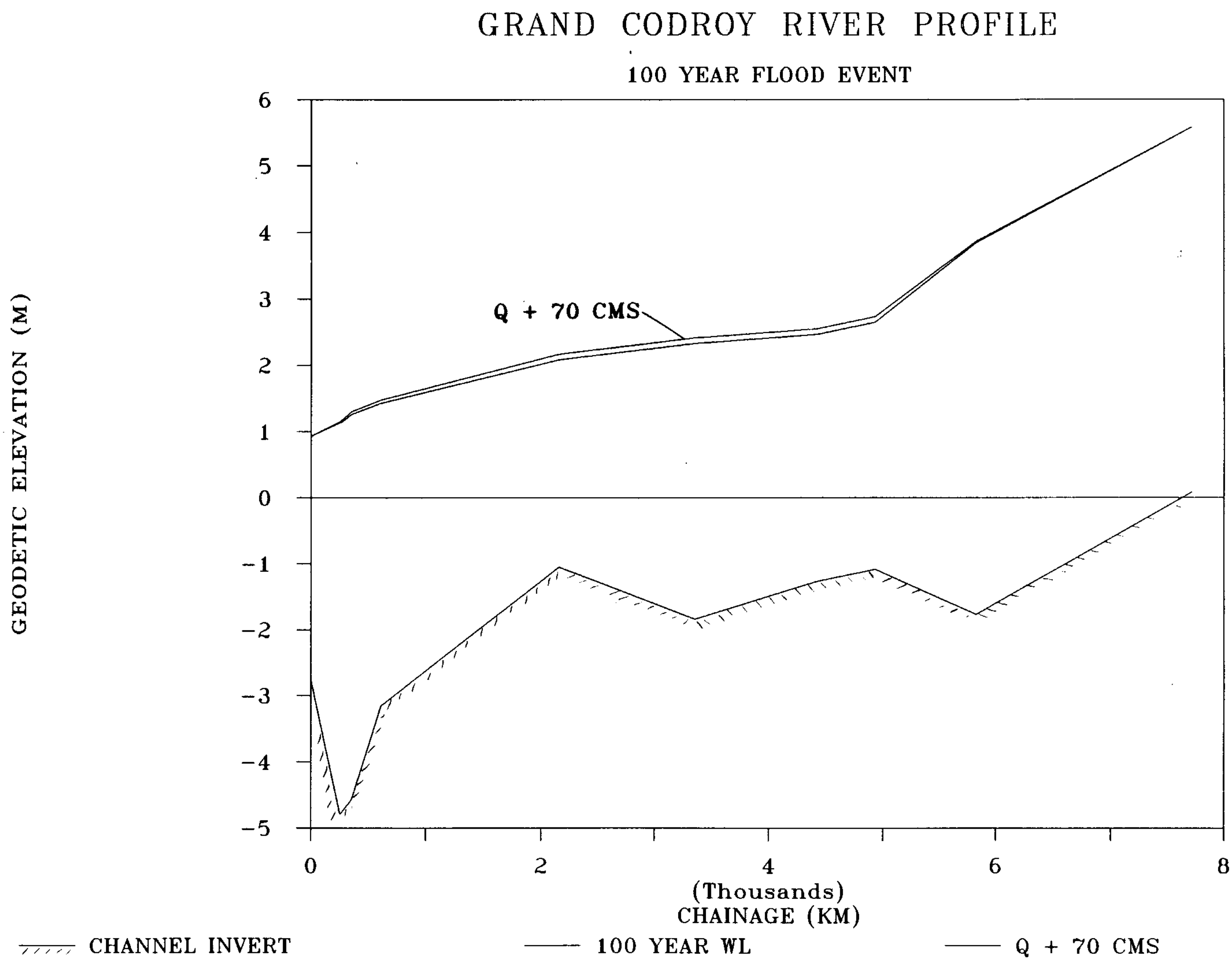

GRAND CODROY RIVER PROFILE
100 YEAR FLOOD EVENT
GEODETIC ELEVATION (M)
6
5
4
3
2
1
0
-1
-2
-3
-4
-5
Q + 70 CMS
0
2
4
6
8
(Thousands)
CHAINAGE (KM)
CHANNEL INVERT
100 YEAR WL
Q + 70 CMS

FIGURE 7-5

sensitivity of the 20 and 100 year levels to the addition of such a flow rate. There is an increase in level in both cases (about 0.08 to 0.1 m), but this influence is limited to the lower channel and does not extend significantly beyond the Provincial Park where the channel slope begins to steepen.

Although the flow rate increase of 70 m^3/s is relatively small, the level increase it creates and the probability (50%) that it would be present during flood flows, indicates that 70 m^3/s should be added to flood flows at the mouth.

Flood Flows

The previous calibration and verification was largely based on simulated flood flows from the QUALHYMO model. Although the backwater results provide another very positive confirmation about the accuracy of the hydrologic modelling, it is appropriate to assess the sensitivity of flood levels to variations in these flows.

Figures 7-6 and 7-7 illustrate that at ± 10% change in flood flow has little effect on flood elevations in the principal study areas - the upper reach, where it is steep and fast flowing, and the lower reach where the floodplain conveyance is large. The middle reach, between about 6 and 16 km from the mouth, is more sensitive to these flow variations, but only because there are very few cross-sections within this area (a "connecting" reach between the upper and lower areas of focused study).

Channel Roughness

It is normal to assess the sensitivity of water levels to channel roughness variations in the range of ± 10 or 15% from the calibrated/verified values. Herein, it was elected to evaluate the nature of a rather wider range ± 25% in these values.

Figures 7-8 and 7-9 (and Table 7-3 which follows) illustrate the effect of this very large range in roughness variation. As is the case with the plots of flow variation, even such major changes in roughness have very little effect in the upper study area (about ± 0.35 m change in water level) and in the lower study area at Upper Ferry. Again, however, the connecting reach which links the

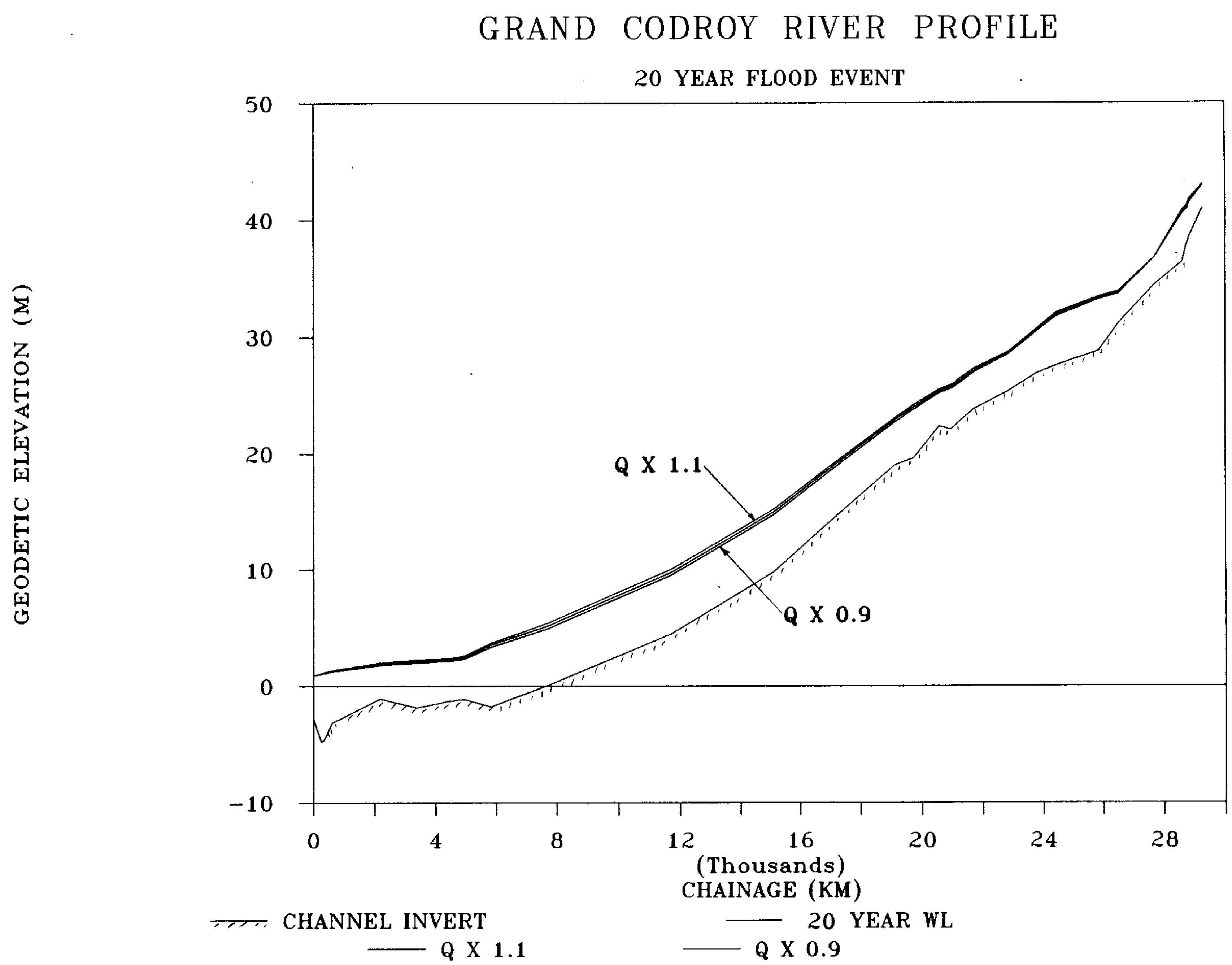
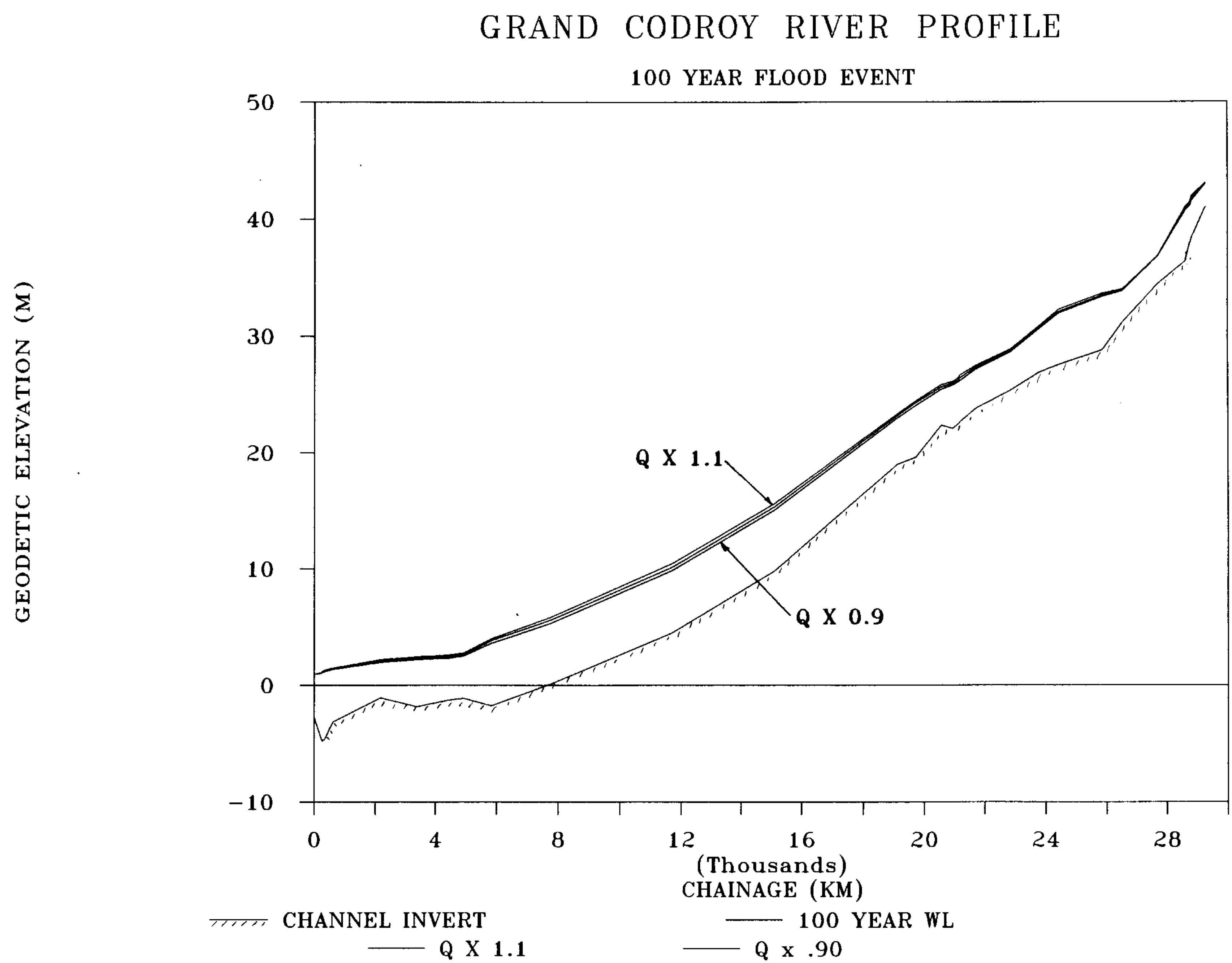

GRAND CODROY RIVER PROFILE
20 YEAR FLOOD EVENT
GEODETIC ELEVATION (M)
50
40
30
20
10
0
-10
Q X 1.1
Q X 0.9
0
4
8
12
16
20
24
28
(Thousands)
CHAINAGE (KM)
CHANNEL INVERT
20 YEAR WL
Q X 1.1
Q X 0.9

FIGURE 7-6



GRAND CODROY RIVER PROFILE
100 YEAR FLOOD EVENT
GEODETIC ELEVATION (M)
50
40
30
20
10
0
-10
Q X 1.1
Q X 0.9
0
4
8
12
16
20
24
28
(Thousands)
CHAINAGE (KM)
CHANNEL INVERT
100 YEAR WL
Q X 1.1
Q x .90

FIGURE 7-7

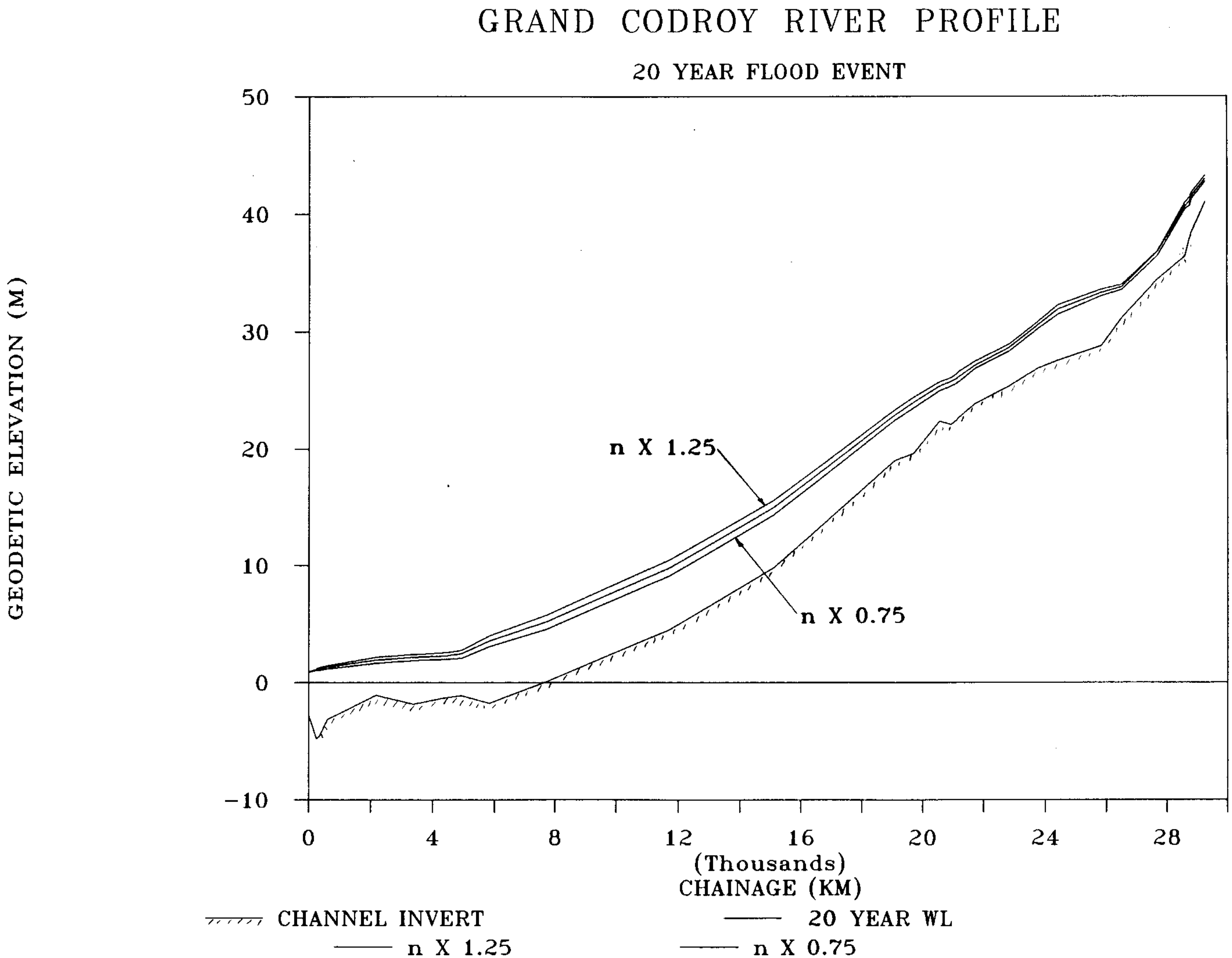

GRAND CODROY RIVER PROFILE
20 YEAR FLOOD EVENT
GEODETIC ELEVATION (M)
50
40
30
20
10
0
-10
0
4
8
12
16
20
24
28
(Thousands)
CHAINAGE (KM)
n X 1.25
n X 0.75
CHANNEL INVERT
20 YEAR WL
n X 1.25
n X 0.75

FIGURE 7-8

GRAND CODROY RIVER PROFILE

100 YEAR FLOOD EVENT

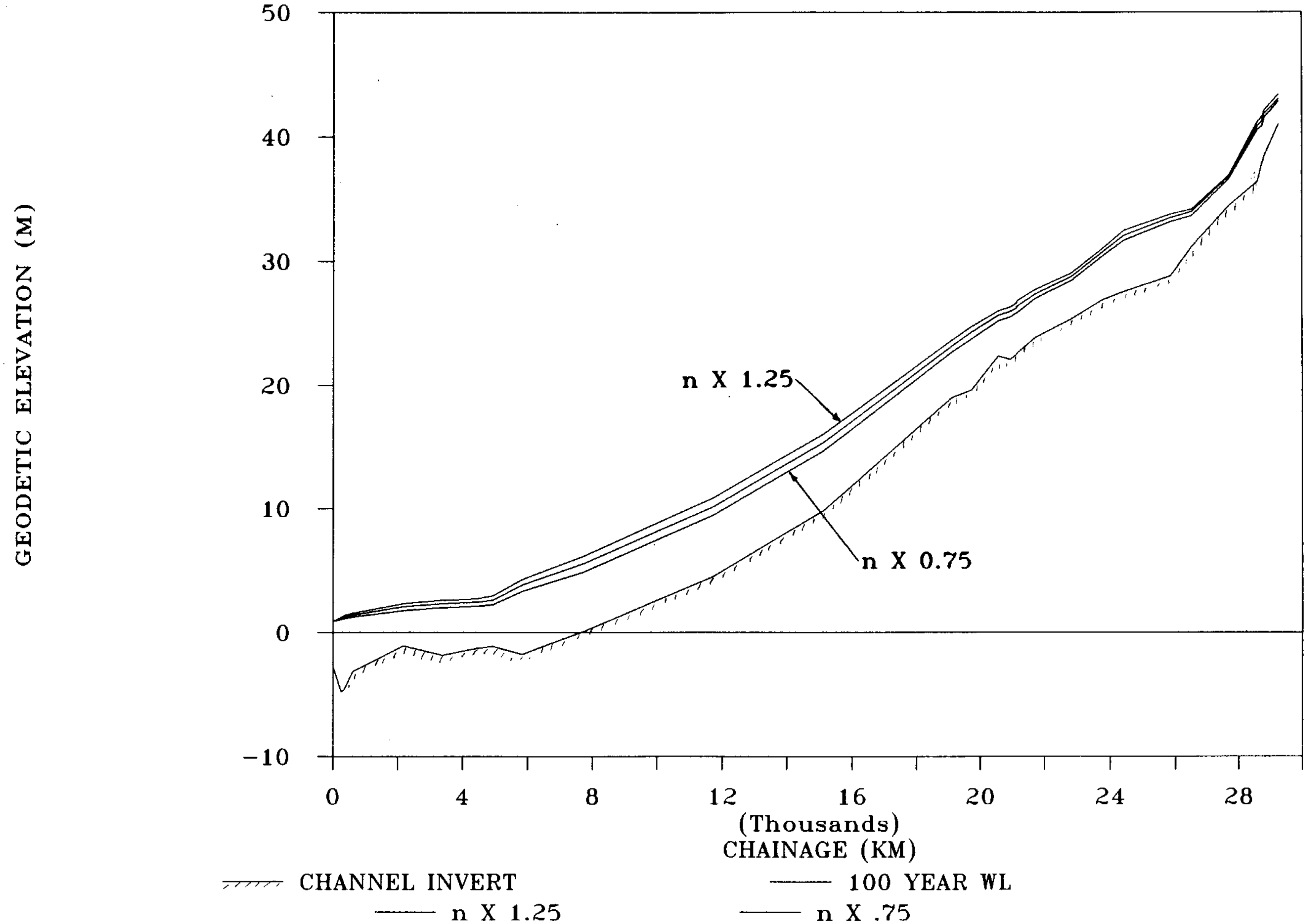


FIGURE 7-9

Starting Water Level

The final sensitivity test was designed to evaluate the effect of starting tidal water levels on the 20 and 100 year open water flood levels. Figures 7-10 and 7-11 show the effect of such changes in the area where their effect is confined - from the river mouth to the Provincial Park.

The lowest starting level is 0.00 m GSCD, the middle level is the mean high tide level (0.93 m) and the upper level is the 20 or 100 year instantaneous high water level in the harbour (1.72 and 1.84 m, respectively). The results illustrate that the flood flow levels are not very sensitive to the lower range of tidal levels during flood flows. Water levels in the lower reach quickly rise to those which are expected at mean high tide within about 2 km from the bridge at Upper Ferry, and the difference in level beyond that is not great.

The figures also show that in the exceedingly rare situation when a 20 or 100 year flow might combined with the peak flow from a 20 or 100 year flood flow (a 400 to 10,000 year probability) gives flood levels which are not above 2 m (GSCD) at the mouth, and little different from other levels about 6 km upstream.

In all, Figures 7-10 and 7-11 illustrate the merit of basing the flood risk mapping or river levels which begin at a mean high tide but include the possibility of high sea/harbour levels during less-than-major flood flows.

Conclusions

The effect of all the preceding sensitivity analyses are summarized in Table 7-3. The important conclusions of the analysis are that:

- It is appropriate to add a tidal prism (ebb tide) flow component to anticipated 20 and 100 year flood flows (Figures 7-4 and 7-5).

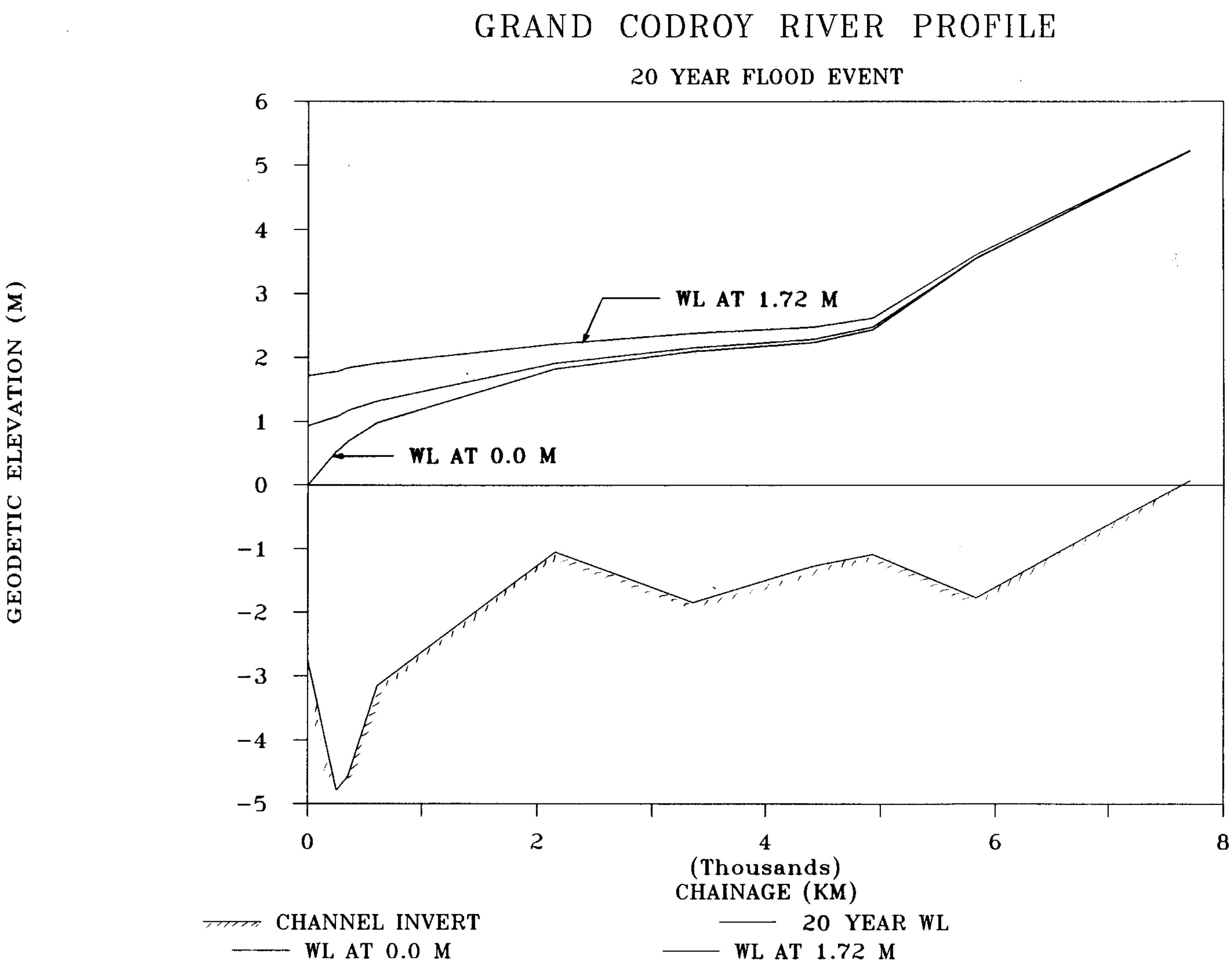

GRAND CODROY RIVER PROFILE
20 YEAR FLOOD EVENT
GEODETIC ELEVATION (M)
6
5
4
3
2
1
0
-1
-2
-3
-4
-5
WL AT 1.72 M
WL AT 0.0 M
0
2
4
6
8
(Thousands)
CHAINAGE (KM)
CHANNEL INVERT
WL AT 0.0 M
20 YEAR WL
WL AT 1.72 M

FIGURE 7-10

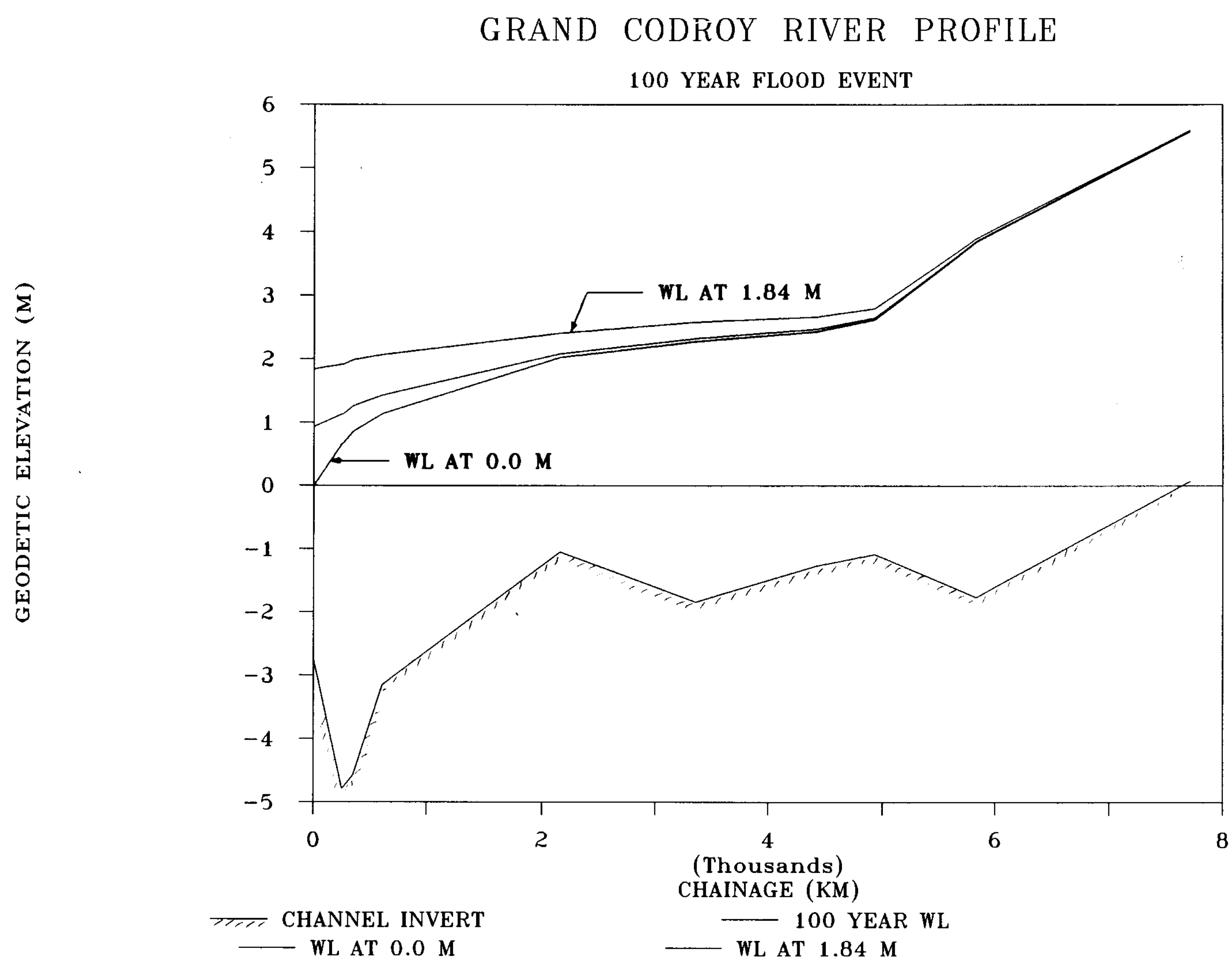

GRAND CODROY RIVER PROFILE
100 YEAR FLOOD EVENT
GEODETIC ELEVATION (M)
6
5
4
3
2
1
0
-1
-2
-3
-4
-5
WL AT 1.84 M
WL AT 0.0 M
0
2
4
6
8
(Thousands)
CHAINAGE (KM)
CHANNEL INVERT
WL AT 0.0 M
100 YEAR WL
WL AT 1.84 M

FIGURE 7-11

TABLE 7-3a : BACKWATER MODEL PARAMETER SENSITIVITY TABLE
100 YEAR STORM EVENT

SECTION #	Q (cms)	100 YEAR ELEVATION	STARTING WATER ELEVATION (SWE) SENSITIVITY: SWE =0.0 m	SWE =1.84 m	MANNINGS 'n' SENSITIVITY: 1.25n	0.75n	DISCHARGE (Q) SENSITIVITY: Q+70	1.1Q	0.9Q
1	1098	0.93	0.00	1.84	0.93	0.93	0.93	0.93	0.93
2	1098	1.14	0.69	1.91	1.24	1.04	1.16	1.18	1.10
2.1	1098	1.14	0.70	1.92	1.25	1.05	1.17	1.18	1.10
3	1098	1.27	0.89	2.00	1.4	1.15	1.31	1.34	1.21
4	1085	1.45	1.18	2.08	1.62	1.29	1.50	1.53	1.37
5	1085	2.13	2.06	2.47	2.41	1.82	2.20	2.25	2.00
6	1085	2.38	2.29	2.61	2.66	2.07	2.46	2.50	2.24
7	1085	2.51	2.47	2.71	2.8	2.19	2.59	2.64	2.38
8	1085	2.69	2.63	2.84	3.03	2.30	2.77	2.82	2.56
9	1085	3.91	3.91	3.96	4.31	3.40	3.94	4.11	3.70
10	1085	5.66	5.66	5.67	6.12	4.98	5.66	5.91	5.40
11	1071	10.30	10.30	10.30	10.8	9.55	10.30	10.59	10.00
12	1071	15.50	15.50	15.50	16.01	14.79	15.50	15.77	15.21
12.5	1045	23.25	23.25	23.25	23.6	22.76	23.25	23.45	23.05
13	1045	24.35	24.35	24.35	24.75	23.81	24.35	24.55	24.15
14	578	25.81	25.81	25.81	26.2	25.38	25.81	25.99	25.61
15	578	26.20	26.20	26.20	26.63	25.76	26.20	26.36	26.03
16	578	26.56	26.56	26.56	27	26.10	26.56	26.73	26.39
16.1	578	26.77	26.77	26.77	27.34	26.21	26.77	27.01	26.53
17	578	26.92	26.92	26.92	27.4	26.30	26.92	27.19	26.64
18	578	27.77	27.77	27.77	28.14	27.36	27.77	27.98	27.57
19	578	29.07	29.07	29.07	29.39	28.69	29.07	29.20	28.93
20	578	31.03	31.03	31.03	31.26	30.70	31.03	31.15	30.90
21	578	32.52	32.52	32.52	32.86	32.04	32.52	32.68	32.33
22	578	33.83	33.83	33.83	34.13	33.46	33.83	33.97	33.67
23	578	34.21	34.21	34.21	34.48	33.90	34.21	34.33	34.08
24	578	36.87	36.87	36.87	36.98	36.84	36.87	36.93	36.86
25	578	41.28	41.28	41.28	41.6	40.77	41.28	41.42	41.09
26	562	41.64	41.64	41.64	42	41.14	41.64	41.78	41.47
26.1	562	41.73	41.73	41.73	42.06	41.31	41.73	41.89	41.54
27	562	42.28	42.28	42.28	42.56	41.95	42.28	42.48	42.05
28	562	43.34	43.34	43.34	43.68	43.33	43.34	43.44	43.21

TABLE 7-3b : BACKWATER MODEL PARAMETER SENSITIVITY TABLE
20 YEAR STORM EVENT

			STARTING WATER ELEVATION (SWE) SENSITIVITY		MANNINGS 'n' SENSITIVITY		DISCHARGE (Q) SENSITIVITY		
SECTION #	Q (cms)	20 YEAR ELEVATION	SWE =0.0 m	SWE =1.72 m	1.25n	0.75n	Q+70	1.1Q	0.9Q
1	929	0.93	0.00	1.72	0.93	0.93	0.93	0.93	0.93
2	929	1.08	0.53	1.78	1.16	1.01	1.10	1.11	1.05
2.1	929	1.08	0.54	1.78	1.17	1.02	1.11	1.11	1.06
3	929	1.18	0.69	1.84	1.28	1.09	1.22	1.23	1.13
4	908	1.32	0.98	1.91	1.46	1.20	1.37	1.39	1.26
5	908	1.92	1.83	2.22	2.18	1.65	2.00	2.03	1.81
6	908	2.16	2.10	2.39	2.41	1.88	2.24	2.27	2.04
7	908	2.29	2.24	2.48	2.56	1.99	2.38	2.40	2.17
8	908	2.48	2.44	2.62	2.78	2.11	2.56	2.59	2.35
9	908	3.56	3.55	3.61	3.93	3.09	3.59	3.75	3.37
10	908	5.22	5.22	5.23	5.65	4.59	5.23	5.45	4.98
11	886	9.79	9.79	9.79	10.24	9.11	9.79	10.05	9.52
12	886	14.99	14.99	14.99	15.46	14.36	14.99	15.24	14.74
12.5	851	22.88	22.88	22.88	23.18	22.44	22.88	23.04	22.69
13	851	23.97	23.97	23.97	24.32	23.47	23.97	24.14	23.78
14	451	25.41	25.37	25.37	25.77	25.01	25.41	25.58	25.23
15	451	25.84	25.74	25.74	26.23	25.45	25.84	25.98	25.68
16	451	26.18	26.05	26.05	26.59	25.76	26.18	26.33	26.02
16.1	451	26.26	26.07	26.07	26.71	25.81	26.26	26.45	26.05
17	451	26.35	26.15	26.15	26.82	25.87	26.35	26.55	26.13
18	451	27.33	27.15	27.15	27.65	26.99	27.33	27.49	27.16
19	451	28.76	28.63	28.63	29.04	28.43	28.76	28.87	28.65
20	451	30.72	30.57	30.57	30.95	30.40	30.72	30.84	30.59
21	451	32.10	31.92	31.92	32.42	31.67	32.10	32.25	31.94
22	451	33.48	33.33	33.33	33.76	33.16	33.48	33.61	33.34
23	451	33.93	33.81	33.81	34.16	33.67	33.93	34.03	33.83
24	451	36.82	36.79	36.79	36.91	36.61	36.82	36.85	36.79
25	451	40.85	40.65	40.65	41.15	40.50	40.85	41.01	40.67
26	438	41.25	41.06	41.06	41.58	40.84	41.25	41.39	41.09
26.1	438	41.31	41.12	41.12	41.62	40.95	41.31	41.47	41.14
27	438	41.77	41.55	41.55	42.05	41.46	41.77	41.96	41.57
28	438	43.00	42.96	42.96	43.38	42.84	43.00	43.04	42.97

- It will eventually be of value to refine the geometry of the river by the addition of extra cross-sections and water level monitoring in the 6 to 16 km reach upstream of the river mouth.

- It is appropriate to base the 20 and 100 year flood levels on a starting level of the mean high tide (0.93 m GSCD) but also include the effect of high sea/harbour levels in delineating flood levels.

7.5 Sensitivity to Ice-Related Flooding Conditions

The second part of the hydraulic analysis required an assessment of break-up ice jams/blockages on flood levels. The proposed approach to obtain realistic estimates of the effect of ice blockages involved the following steps:

- collection of reliable data on water levels, ice jam locations and extent of the problem;

- estimating the degree of channel blockages (which reduce the flow conveyance area) from each ice jam using simulated flow data;

- simulating backwater levels with assumed and adjusted blockages until historical flood levels are matched;

- assessing the sensitivity of flood levels to ice jams/blockages.

Section 6.5 describes the ice-related flooding processes in the Codroy River basin and provides some estimates and measurements of ice effects along the river. Unfortunately, there is little other technical data than Tables 6-1 and 3-1 to describe the ice formations which have caused previous flooding, because so much of the river is remote from residential observers.

Although it initially appeared that there were six ice jam sites which could be simulated, only three had sufficient information on ice effects to reliably simulate backwater conditions during spring flow periods. These three top the following list of sites:

1) Trans Canada Highway Crossing at the North Branch
2) North Branch between Coal Brook and the TCH
3) Coal Brook area
4) Grand Codroy River upstream of the Provincial Park (section 10)
5) the Grand Codroy Provincial Park
6) Upper Ferry at the bridge

Trans Canada Highway Bridge - North Branch

Data from this bridge are reasonably good for the ice jams of March 1988 and February 1981. On these occasions, flood levels were estimated to be 40.8 m GSCD at the bridge and 40.5 m GSCD just upstream, respectively. However, flow estimates are only available for 1981 (QUALHYMO) and, hence, 1981 was selected for simulation.

Blockage simulations focused on cross-sections 25, which is just below the bridge and the recognized site of ice jam initiation. The blockage simulations began by considering only 50% of the channel conveyance was not blocked by ice at that time, and subsequent iterations considered 40% and 30% of the channel was open. The results are as follows for section 27:

Percent of Channel Open	Flood Elevation Simulated at Section 27 (GSCD)
50%	40.25 m
40%	40.44 m
30%	40.74 m

These simulations indicate that only about 35% of the channel was open in 1981 and perhaps less than 30% was open in 1988. What is also important is that the flood elevations approach the 20 year level with this type of blockage, but that the streamflow is only in the 2 year range.

North Branch Between Coal Brook and the TCH

Similar analysis for this site was based on observed levels and ice conditions in April 1987 at cross-sections 24 to 27. Iterations with various ice blockages give the following for comparison with the observed:

Percent of Channel Open	Flood Elevations Simulated at Sections 24	25	26
15%	38.01	41.00	41.75
10%	38.58	41.53	42.45
5%	40.49	40.55	40.56
Observed	39.50	40.45	40.90 (est)

These simulations indicated that only about 7% of the channel conveyance was open and unblocked in this year, which again had a flow rate (133 m^3/s) well below the 2 year flood flow. The observed flood level again approached the 20 year open water level and at section 24 was well above the 100 year level.

Coal Brook Area

The same ice blockage analysis was conducted in the area of cross-sections 20 and 21, where a level of 32.2 m GSCD was identified as the flood elevation associated with ice jams in mid-January 1986. Iterations with a range of channel blockages give the following results:

Percent of Channel Open	Flood Elevation Simulated at Section 21	22	23	Observed
50%	31.87	33.26	33.60	
30%	32.31	33.56	33.75	
20%	32.64	33.95	34.04	
Observed	32.20	---	---	

In this case, a blockage of about 35% of the conveyance appears to have been caused by ice. Flood flows in this year at section 21 (283 m^3/s) were in the 2 to 5 year range, but resulting flood levels are up in the 20 to 100 year open water range.

Grand Codroy Provincial Park

Wardens at the park, who rarely visit the park site (cross-section 7) in the spring, observed floating ice in early April 1986 at elevation 1.24 m GSCD. Analysis of various blockages quickly determined that this flood level was not associated with an ice obstruction, but simply the level of the river with open channel flow (282 m^3/s) conveying ice toward the river mouth.

Grand Codroy River Upstream of the Provincial Park

A local resident at cross-section 10 showed our field people a photograph of an ice accumulation which took place on March 11, 1986. It is difficult to establish from the photograph and ice scars elevations in the area, whether or not the ice level (± 10.08 m GSCD) was the same as the water level. It appears that there was definitely an ice obstruction between section 9 and 10, but the photograph shows that a considerable volume of ice was likely pushed up the sloping banks to that level rather than being carried there by flood waters.

It also proved impossible to simulate ice blockages at this site, because the date corresponded with a very low flow period - insufficient to support an ice accumulation to the observed/reported level. It may well be that this accumulation was caused by the floodwave associated with the release of an unknown upstream ice jam, but this will never be known with certainty.

Upper Ferry at the Bridge

This location does not usually experience ice-related problems, and the local ice often melts in place or is carried to the open sea without significant jamming or elevated backwater levels. However, ice was present during part of the 1978 flood which led to the collapse of the Grand Codroy Bridge.

The ice conditions which were present during the 1978 disaster were not unusual in terms of thickness, and nothing uncommon was noted about the resulting backwater level (elevation perhaps 1.0 m GSCD). What was noted as unusual was that an ice cover and accumulation were present on the north side of the river channel, but the depth, extent and duration of the accumulation were not a focus of particular attention. The accumulation also appeared to affect the formation of an eddy, but this might equally well have been caused by high flows eroding the river bottom materials.

In all, the absence of quantitative data about ice and water level conditions make it impossible to successfully reconstruct the extent of blockage which was present during 1978.

However, it is clear than an ice cover can and does form in the lower river and has remained in place during flows which are in the range of the 20 year value (e.g. 1978) but not higher. This cover impedes the progression of breakup ice, and it is considered realistic to assess backwater conditions near Upper Ferry with such a cover in place.

The "Ice Option" of the HEC-2 model was employed for this assessment. The parameters for the simulation included:

- 0.93 m — starting water level (average high tide)
- 0.46 m — ice accumulation thickness in the main channel and overbanks (as observed maximum)
- 0.04 — Mannings "n" value for ice accumulations
- 0.80 — specific gravity of ice containing some air voids

The results of the modelling are included in the Technical Appendix, but in summary give the following:

River Cross Section	Open Water Level (m)* 20 year	100 year	Ice Effect Level (m) 20 Year Flow
1.0	1.72	1.84	0.93
2.0	1.72	1.84	1.61
2.1	1.72	1.84	1.62
3.0	1.72	1.84	1.71
4.0	1.72	1.84	1.96
5.0	2.00	2.20	2.83

*includes effect of 20 and 100 year tide levels and outflow of tidal prism.

The simulation indicates that such an ice accumulation in the Upper Ferry region would elevate flood levels into the 1 in 100 year open water range at about the time that the ice clears the area.

7.5.1 Ice Related Sensitivity Conclusions

The findings above suggest that ice jams on the Codroy River upstream of the Provincial Park cause significant blockage of the low flow channel - certainly sufficient blockage to involve the overbanks in flood flow conveyance. This is not an unusual condition and is perhaps the normal condition in many rivers which have relatively wide flood plains (Calkins, 1983; Gerard and Calkins, 1984; MacLaren Plansearch, 1989). In essence, the downstream component of forces acting on such jams do not initially develop sufficiently to sweep the blockage away, because the force is relieved when the water reaches the bankflow spill stage. As levels rise, or the ice jams weaken by erosion or thermal effects, however, the ice jams release and are swept downstream.

It is also noted in the above reports that historical flood levels developed by jams are generally at about the maximum level which can be reached by ice accumulations at such locations.

In all, it is evident from local observations and simulations for the river upstream of the Provincial Park, that:

- ice jam blockages have previously contributed to spring flooding by raising water levels at times when flood flows are not particularly high;

- flood elevations from most historical ice jams upstream of the river mouth are raised to about the 20 year open water level before they release, but in one case (Coal Brook) the ice level was at the 100 year open water level;

- best available information indicates that historical high water levels developed by ice jams are adequately accounted for in the floodway and flood risk area defined by the 20 and 100 year open water flows, respectively.

The ice analysis findings in the previous section for the river reach <u>near Upper Ferry</u> are somewhat theoretical because of the absence of good field data. It is possible to simulate the river reach with the HEC-2 (Ice Option) model and the simulated conditions - which appear realistic from the limited data - indicate that ice effect flood levels may reach or slightly exceed 100 year open water levels. In all, however:

- ice related flood levels in the Upper Ferry area have not been observed to exceed the 100 year open water level, even during the 1978 flood;

- ice effect levels (from modelling) appear to be realistically accounted for in the flood risk area defined by open water flows.

7.6 <u>Hydraulic Analysis Conclusions</u>

The results of the open water analysis, and analyses of sensitivity of levels to the effect of tides and ice jams and other factors, leads to the following conclusions for defining and delineating the flood risk area:

1) It is appropriate to define flood prone areas using 20 and 100 year return period open water flood flows.

2) A discharge of 70 m^3/s should be added to these flows at the mouth (sections 1 to 7) to account for the tidal prism discharge during ebb tide.

3) The starting water level for defining flood levels in the river should be the mean high tide (0.93 m GSCD), but the instantaneous 20 and 100 year sea levels (1.72 m and 1.84 m) should be employed when delineating the flood hazard at the river mouth.

8.0 FLOOD LEVEL PROFILES

As indicated previously, the purpose of this study is to derive water surface profiles for the 1:20 and 1:100 year return period floods. The conclusions of the hydraulic analysis (Section 7.6) were used to define these levels.

8.1 Flood Level Delineation

Initially, the complete calibrated/verified HEC-2 model was run with a starting level of 0.93 m and the 1:20 and 1:100 year peak flows appropriate for each location. This provided water level data at all cross sections in the model, and a summary printout of the results for both flood conditions is presented in the Appendix. Figure 8-1 plots the profiles and Table 8.1 lists the flood levels at each sections. These levels include the effect of tidal discharge at the river mouth.

Next, the effect of 20 and 100 year sea levels were taken into account. These levels are higher than the river levels at Sections 1 to 4 inclusive, and, hence, elevation 1.72 m (20 year level at river mouth) and elevation 1.84 m (100 year level) replace the river flow level derived above. This completed the analysis at the mouth by showing the combined possibility of flood from high flows on the river or high levels at the mouth.

Last, the flood levels from Table 8.1 were plotted on the 1:2500 scale mapping. Typical cross section locations in the model are plotted in Figure 8-2, which covers a small portion of one of the floodplain mapping sheets. 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 along the channel 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.

Figure 8-2 shows the Trans-Canada Highway crossing about 3.5 km northeast of Coal Brook. Shown here are three structures which are in the flood prone area - a house and garage on the upstream side of the bridge and a cottage on the downstream side. The flood depth at the house would be about 0.5 m during a 1:20 year return period flood and 1.0 m during the 1:100 year flood. Depths at the garage are approximately 0.2 and 0.7 m for the 1:20 and 1:100 year floods,

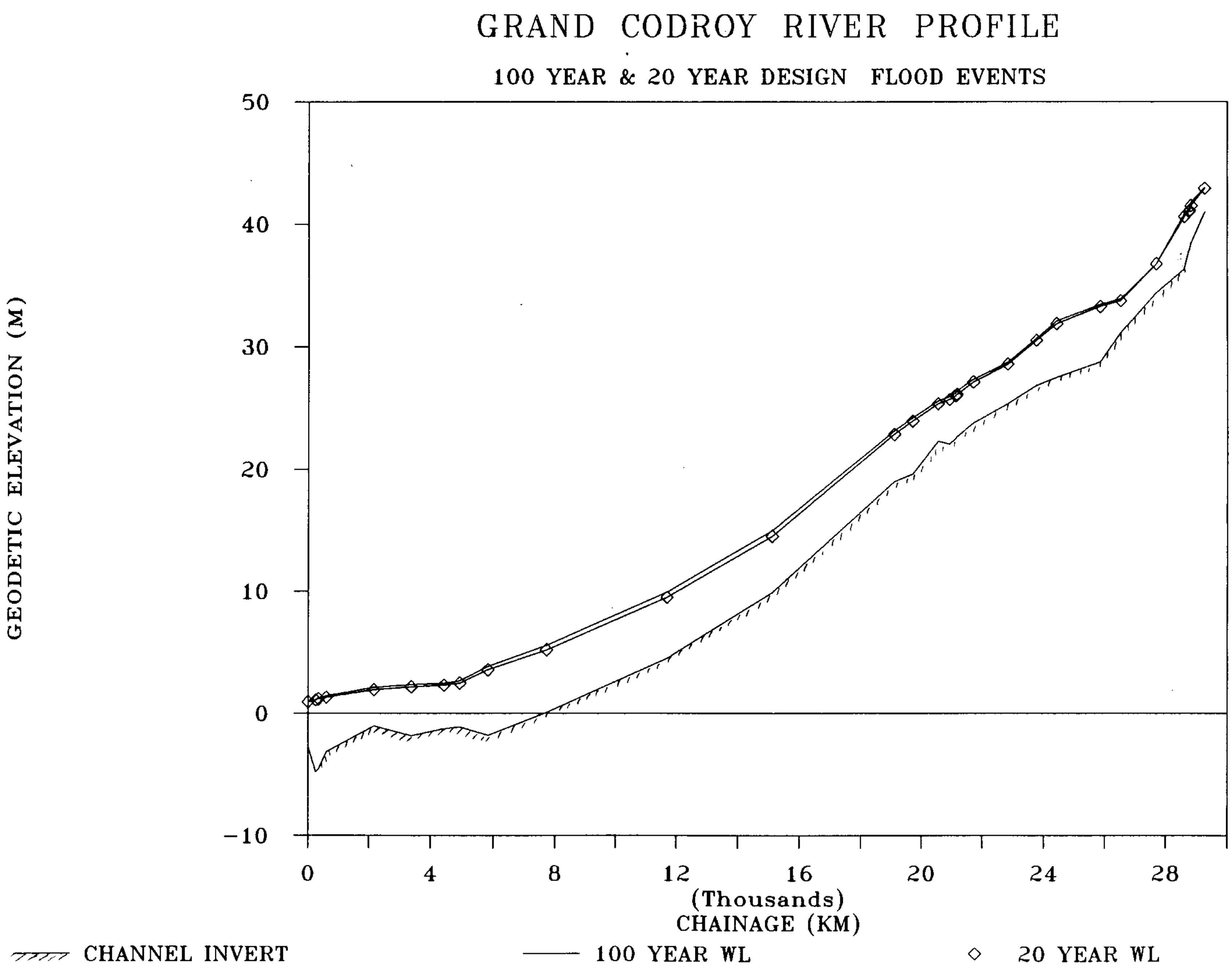

GRAND CODROY RIVER PROFILE
100 YEAR & 20 YEAR DESIGN FLOOD EVENTS
GEODETIC ELEVATION (M)
50
40
30
20
10
0
-10
0
4
8
12
16
20
24
28
(Thousands)
CHAINAGE (KM)
CHANNEL INVERT
100 YEAR WL
20 YEAR WL

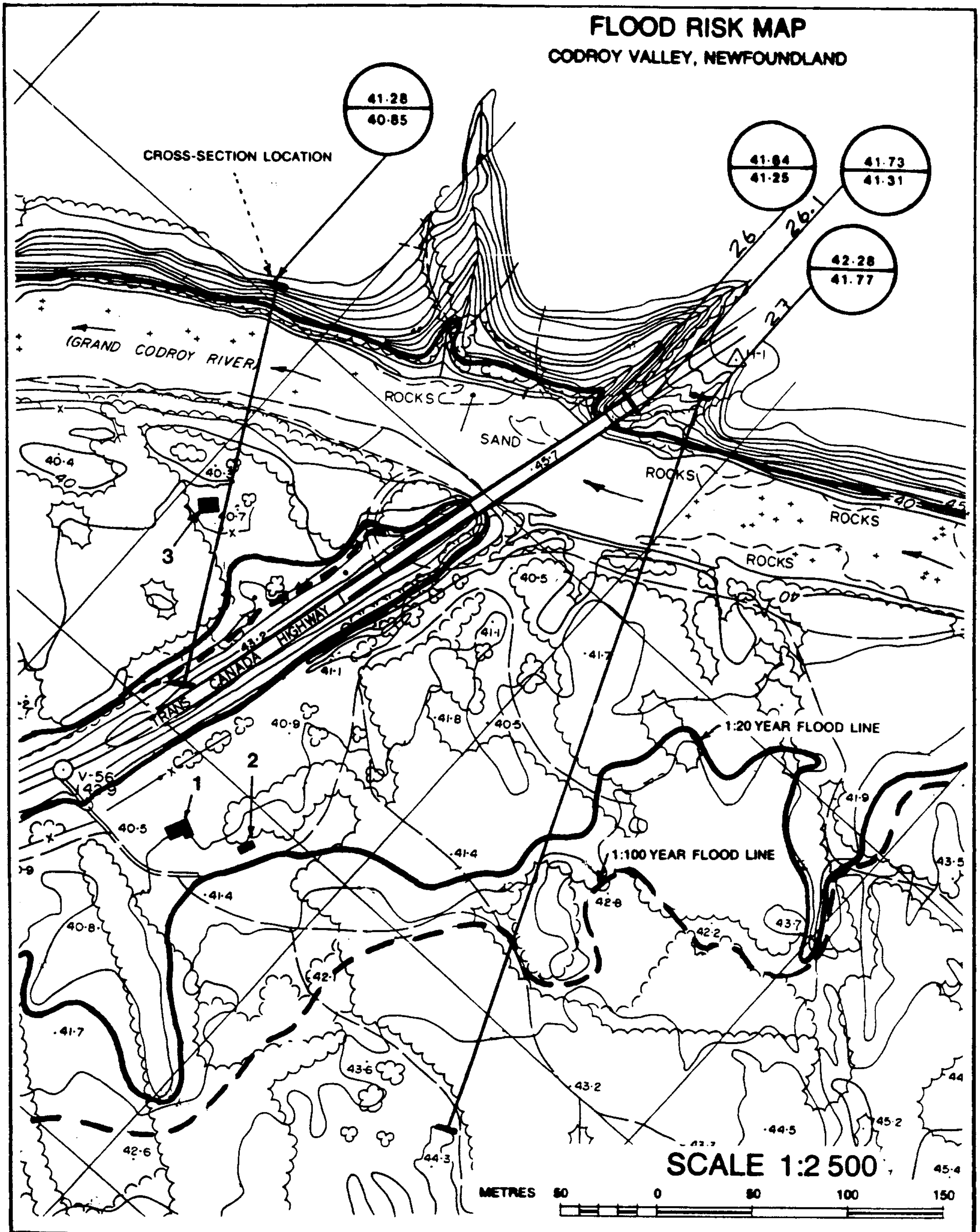
FIGURE 8-1

TABLE 8.1

CODROY VALLEY AREA

100 YEAR AND 20 YEAR DESIGN FLOOD ELEVATIONS

SECTION #	100 YEAR Q (CMS)	100 YEAR WATER LEVEL (M)	20 YEAR Q (CMS)	20 YEAR WATER LEVEL (M)
1	1098	1.84	929	1.72
2	1098	1.84	929	1.72
2.1	1098	1.84	929	1.72
3	1098	1.84	929	1.72
4	1085	1.84	908	1.72
5	1085	2.13	908	1.92
6	1085	2.38	908	2.16
7	1085	2.51	908	2.29
8	1085	2.69	908	2.48
9	1085	3.91	908	3.56
10	1085	5.66	908	5.22
11	1071	10.30	886	9.79
12	1071	15.50	886	14.99
12.5	1045	23.25	851	22.88
13	1045	24.35	851	23.97
14	578	25.81	451	25.41
15	578	26.20	451	25.84
16	578	26.56	451	26.18
16.1	578	26.77	451	26.26
17	578	26.92	451	26.35
18	578	27.77	451	27.33
19	578	29.07	451	28.76
20	578	31.03	451	30.72
21	578	32.52	451	32.10
22	578	33.83	451	33.48
23	578	34.21	451	33.93
24	578	36.87	451	36.82
25	578	41.28	451	40.85
26	562	41.64	438	41.25
26.1	562	41.73	438	41.31
27	562	42.28	438	41.77
28	562	43.34	438	43.00


FLOOD RISK MAP
CODROY VALLEY, NEWFOUNDLAND
CROSS-SECTION LOCATION
41·28
40·85
41·64
41·25
41·73
41·31
42·28
41·77
GRAND CODROY RIVER
ROCKS
SAND
ROCKS
ROCKS
ROCKS
TRANS CANADA HIGHWAY
1:20 YEAR FLOOD LINE
1:100 YEAR FLOOD LINE
V-56
SCALE 1:2 500
METRES 50 0 50 100 150
FIGURE 8-2

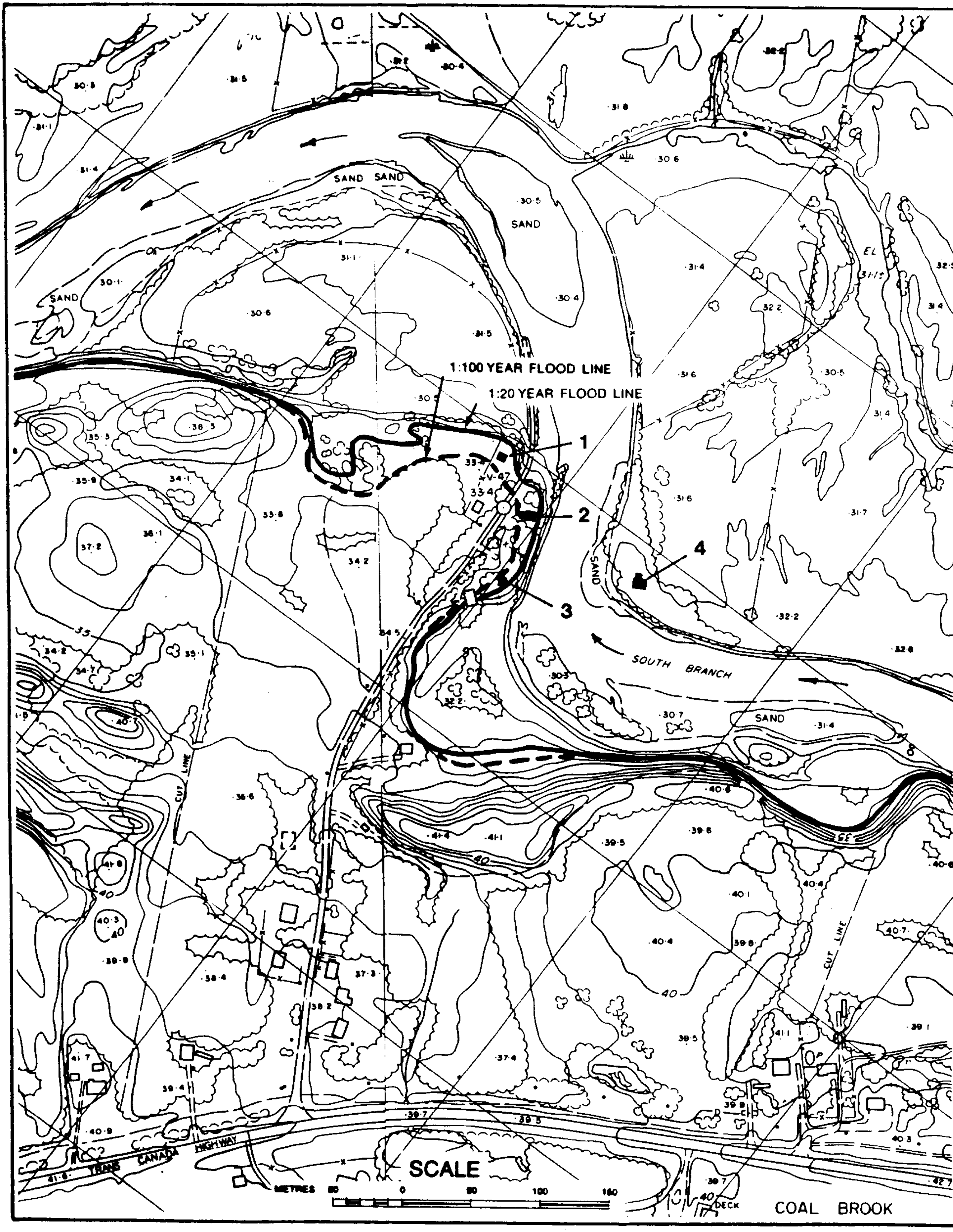
respectively. The cottage just downstream of the bridge would be surrounded by flood depths of 0.3 and 0.8 m during the 1:20 and 1:100 year floods.

Figure 8-3 is a photo-reduced copy of a portion of the flood risk map in the Coal Brook area. This location has four structures which are in the flood risk area defined by the 1:100 year flood level. Three of these (two of which are sheds) are located on the river bank along the small road which runs north from Coal Brook. Flood depths at each structure are about 0.1 m during the 1:100 year flood. The fourth building is a barn, on the river bank across from the other three. Flood depth at the barn rises from approximately 0.2 m for the 1:20 year flood to about 1.6 m during the 1:100 year flood.

Outside of these two areas, no other locations along the Codroy River contain flood-prone structures. In summary:

Figure	Structure	Comment
8-2	- house - garage - cottage	. all in the floodway where flood depths and/or velocities could lead to flood damage or risk to life
8-3	- three structures (2 sheds)	. on fringe of flood risk area where flood damage is possible but limited
	- barn	. in floodway where damage to structure and contents is potentially high

It should be noted that Figures 8-2 and 8-3 are but examples of portions of the flood risk maps prepared for the project. A total of 13 sheets have been prepared and the interested reader may order copies through the Department of Environment and Lands, St. John's.



1:100 YEAR FLOOD LINE
1:20 YEAR FLOOD LINE
1
2
3
4
SAND
SOUTH BRANCH
CUT LINE
TRANS CANADA HIGHWAY
SCALE
METRES 50 0 50 100 150
DECK
COAL BROOK

FIGURE 8-3

9.0 FLOOD DAMAGE REDUCTION OPTIONS

The last stage in this Flood Damage Reduction study involved preliminary identification of flood damage reduction alternatives which could be employed along the river. Noteworthy in considering these alternatives is that the floodplain mapping identifies a "floodway" and a "flood fringe" area.

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 valley area lying between the floodway and the outer limit of the area which is flooded on an average of once in 100 years (or which would be flooded by a repeat of an historical flood). 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 desirable 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 there is very limited damage 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.

9.1 Flood Damage Prevention

There are currently seven structures within the flood-prone area of the Grand Codroy River. There are, however, many other flood-prone areas which might otherwise be considered as desirable river-side locations for cottages or farm buildings.

9.1.1 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 desirable and acceptable to use this area for agriculture or open recreational purposes (i.e., as it is now).

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 farm buildings in the floodway fringe where flood-proofing measures, such as a berm, can substantially reduce the amount of future damage during a flood situation.

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 implementing floodplain regulations is low and the flood damage reduction benefits for future development is high.

9.1.2 Flood Proofing

Flood proofing includes 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 include:

- installing permanent or temporary closures at low level openings in structures;
- raising structures on fill, columns or piers;
- constructing 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. Flood proofing by permanent closure is usually limited to large industrial/commercial 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 for structures when permanent closure is difficult or impossible. As with permanent closure, no human intervention or flood warning is required to make the flood proofing effective.

Construction of floodwalls or berms around flood prone structures is most effective for structures with sufficient space surrounding them to put these works in place, and where soil permeability is low or flood peaks rise and fall quickly. This form of damage protection also reduces the potential for debris/ice damage to the building itself. Its principal limitation is that special consideration must be given to building access if the berm of floodwall is higher than about 1 m.

9.2 Recommended Option

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

1) It is recommended that the flood elevations advanced herein be adopted so that developable areas which are prone to flooding can be zoned in the near future for special flood risk restrictions and design consideration.

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

   a) Flood proofing by elevation on fill or by low berms for the three structures (numbered 1, 2 and 3) in the flood fringe near Coal Brook (Figure 8-3). Flood depths are projected to be low, and these buildings appear to be small/light enough for elevation. Alternately, a low berm (about 0.2 m high) could be put in place around these structures.

      These buildings might also be relocated outside of the flood fringe (a move of 10-20 m from their present location) if such has a cost-benefit advantage over the above options.

   b) Flood proofing by elevation at the barn within the floodway at Coal Brook (structure number 4 in Figure 8-3). This could be prohibitively expensive, however, considering the type and seasonal use of the structure.

      A less comprehensive but potentially valuable alternative for reducing flood damages would involve construction of a crescent shaped berm on the upstream side of the barn. This berm would not keep flood waters out of the building, but would substantially reduce the water speed and protect the barn from the impact of ice or other debris.

   c) Elevation on fill (about 1 m) of the cottage near the TCH Bridge near Coal Brook (structure number 3, Figure 8-2). This cottage is in the floodway at an area where there is considerable ice within the spring flood flow. Hence, elevation on columns/piers would not be secure against ice impact.

The preferable option would be to relocate (and flood proof) this building in the flood fringe about 50 m closer to the highway.

d) Flood proof the house and garage on the east side of the TCH near the bridge (structures numbered 1 and 2 in Figure 8-2). Both of these structures are in the floodway, but in an area where velocities are low. Hence, elevation on piles, columns or piers (or fill) should be possible. In addition, it would be necessary to add fill at access/egress points at this house to ensure that residents could leave the flood risk area in safety.

In conclusion, there are damage reduction options which may be applied to reduce future and existing problems in the Codroy Valley area. 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.

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CODROY VALLEY STUDY AREA
DRAINAGE BOUNDARIES

LEGEND

4 AREA IDENTIFICATION NUMBER

— — DRAINAGE BOUNDARIES

Roads:

hard surface, all weather	dual highway	more than 2 lanes
hard surface, all weather	2 lanes	less than 2 lanes
loose or stabilized surface, all weather	2 lanes or more	less than 2 lanes
loose surface, dry weather		
cart track		
trail or portage		

Routes:

pavée, toute saison	2 chaussées séparées	plus de 2 voies
pavée, toute saison	2 voies	moins de 2 voies
gravier aggloméré, toute saison	2 voies ou plus	moins de 2 voies
de gravier période sèche		
de terre		
sentier ou portage		

Scale 1:250,000 Échelle

Miles 5 0 5 10

Kilometres 5 0 5 10 15 20

CONTOUR INTERVAL 100 FEET
Elevations in Feet above Mean Sea Level
North American Datum 1927
Transverse Mercator Projection

Copies may be obtained from the Map Distribution Office,
Department of Energy, Mines and Resources, Ottawa.

FIGURE 4-1