Flood Risk Mapping Study
of
Portugal Cove, St. Philips, and Outer Cove

FINAL REPORT
Volume I

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Prepared for:

Department of Environment
Water Resources Division
Confederation Complex
4th Floor West Block
St.John's, Newfoundland
A1B 4J6

Prepared By:

CORETEC Incorporated
Suite E100, Prince Charles Building
120 Torbay Road
St. John's, Newfoundland
A1A 2G8

In Association With:

Davis Engineering & Associates Limited
112 Main Street
P.O. Box 40
Port Blandford, Newfoundland
A0C 2G0

CORETEC
Incorporated
Consulting and Research Engineers
Environmental and Information Technology

March 28, 1996

Mr. Robert Picco, P.Eng.
Head, Flood Studies Unit
Water Resources Management Division
Department of Environment
4th. Floor, Confederation Building West Block
St. John's, NF A1B 4J6

Re: Flood Risk Mapping Study of Portugal Cove, St. Philips, and Outer Cove

Dear Mr. Picco:

We are pleased to submit the final report on the above referenced study. Enclosed you will find the original plus fifty (50) copies of Volume I - Main Report, and ten (10) copies of Volume II - Appendices. Also included please find the electronic version of the report including text, data files, and applicable technical drawings.

Thank you for giving us the opportunity to work on this interesting and challenging study.

Please do not hesitate to call if you have any questions.

Yours truly,

Mona El-Tahan

Mona Shahwan El-Tahan, M.Eng., P.Eng.
President & CEO

Suite E100, Prince Charles Building, 120 Torbay Road
St. John's, Newfoundland, Canada A1A 2G8
Telephone: (709) 739-7770 Facsimile: (709) 739-7780 email: coretec@voyager.newcomm.net

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ACKNOWLEDGEMENTS

The Study Team is particularly appreciative of the assistance and guidance provided by Mr. Bob Picco, Head, Flood Studies Unit, Mr. Stephen Dyke, and Mr. Martin Goebel of the Department of Environment. The team also acknowledges the advice provided by Dr. Leonard Lye of Memorial University of Newfoundland.

Members of Study Team:

CORETEC Incorporated

Mona Shahwan El-Tahan, M. Eng., P.Eng.
Momen Wishahy, PhD., P.Eng.
Lisa Alexander, B. Eng.

Davis Engineering & Associates Limited

H. Laban Davis, P.Eng.,
Elaine D. McArthur, B.Eng.
Craig Hobbs
Alan Davis

1.0 INTRODUCTION

1.1 General

Prior to 1990, agreements between the Province of Newfoundland and the Government of Canada allowed for the identification and delineation of flood prone areas within the Province with the objective of reducing flood damages on flood plains. Sixteen flood prone areas in the Province were mapped and the flood risk areas were delineated and designated as areas where only certain conforming development should take place under this Flood Damage Reduction Program. However, at the termination of the Flood Risk Mapping and Studies Agreement in 1990, fifteen areas made up of over forty communities had experienced flooding problems but were not included in the program. In July 1993, a new agreement to help protect and conserve Newfoundland's water resources was signed which allows for flood risk mapping and studies to be carried out.

Under this new agreement, four flood prone areas within the towns of Portugal Cove - St. Philips and Logy Bay-Middle Cove-Outer Cove were identified as the focus of a study. In July 1995, CORETEC Incorporated, in association with Davis Engineering and Associates Limited, were commissioned by the Newfoundland Department of Environment to undertake a "Flood Risk Mapping Study of Portugal Cove, St. Philips and Outer Cove".

1.2 Objectives and Scope of Study

The following objectives of the study summarize the overall scope of the investigations:

[1] Conduct a thorough review of existing information for the purpose of understanding the flooding problem in the study area and the factors responsible for past floods.

[2] Coordinate a field program to collect data required to update the topographic mapping, to establish historical flood levels, and to calibrate the selected mathematical models.

[3] Conduct a hydrological investigation of the study area to determine the flows associated with 1:20 and 1:100 year recurrence interval floods by comparing streamflow record

analysis with flows obtained by modelling the physiographic features of the watersheds and using specified precipitation/snowmelt input.

[4] Using the flows obtained from the hydrology studies, perform a hydraulic analysis to determine the water surface profiles associated with the 1:20 and 1:100 year floods.

[5] Plot the 1:20 and 1:100 flood profiles for the areas on 1:2500 scale digital maps.

[6] Identify and evaluate appropriate remedial measures to alleviate any potential flood damage problems.

1.3 Study Area Description

Portugal Cove, St. Philips and Outer Cove are located on the Avalon Peninsula north of St. John's. The Town of Portugal Cove-St. Philips experiences flooding in two of its rivers: Main River and Broad Cove River. Ice jams are a signficant part of the problem in these areas. The Town of Logy Bay-Middle Cove-Outer Cove experiences flooding in two unnamed brooks, one located adjacent to the Collision Clinic in Outer Cove, and the other near Caddigan's Road in Logy Bay. In these locations the flooding has been attributed to undersized or collapsed culverts.

2.0 HISTORICAL REVIEW

2.1 Sources

To determine the flooding history of the study area, a number of information sources were investigated. Radio stations were contacted, however, they do not maintain archives and would only be able to provide information about very recent past events. Newspaper staff said that they do not maintain archives either, however, many libraries would have them on file. The main drawback was that it was necessary to know the dates of the events in question and for any known event dates, the details were available through the records of the Department of Environment. Interviews with local residents provided some qualitative details about past flooding events and subsequent damage, however, no photographs or quantitative information was available through these personal interviews. The primary sources of information, therefore, were the records of the provincial Department of Environment.

2.2 Recorded Events

2.2.1 Portugal Cove

In order to prevent recurring or anticipated periodic localized flooding, dredging and clearing for various sections of Main River in Portugal Cove have been approved and events documented by the Department of Environment. These renovations include events such as the removal of gravel and debris from a 25 m section of the river approved in July 1994; the removal of gravel buildup from a 30 m stream section approved in March 1987. Ice jams have also been found to cause flooding as was the case in February 1986 when an ice jam occurred at the bridge on Anglican Road (see Figure 1).

On April 11, 1986, a flood occurred on the Storey property in Portugal Cove as a result of an undersized driveway culvert on the river (see Figure 1 for location). The culvert was not capable

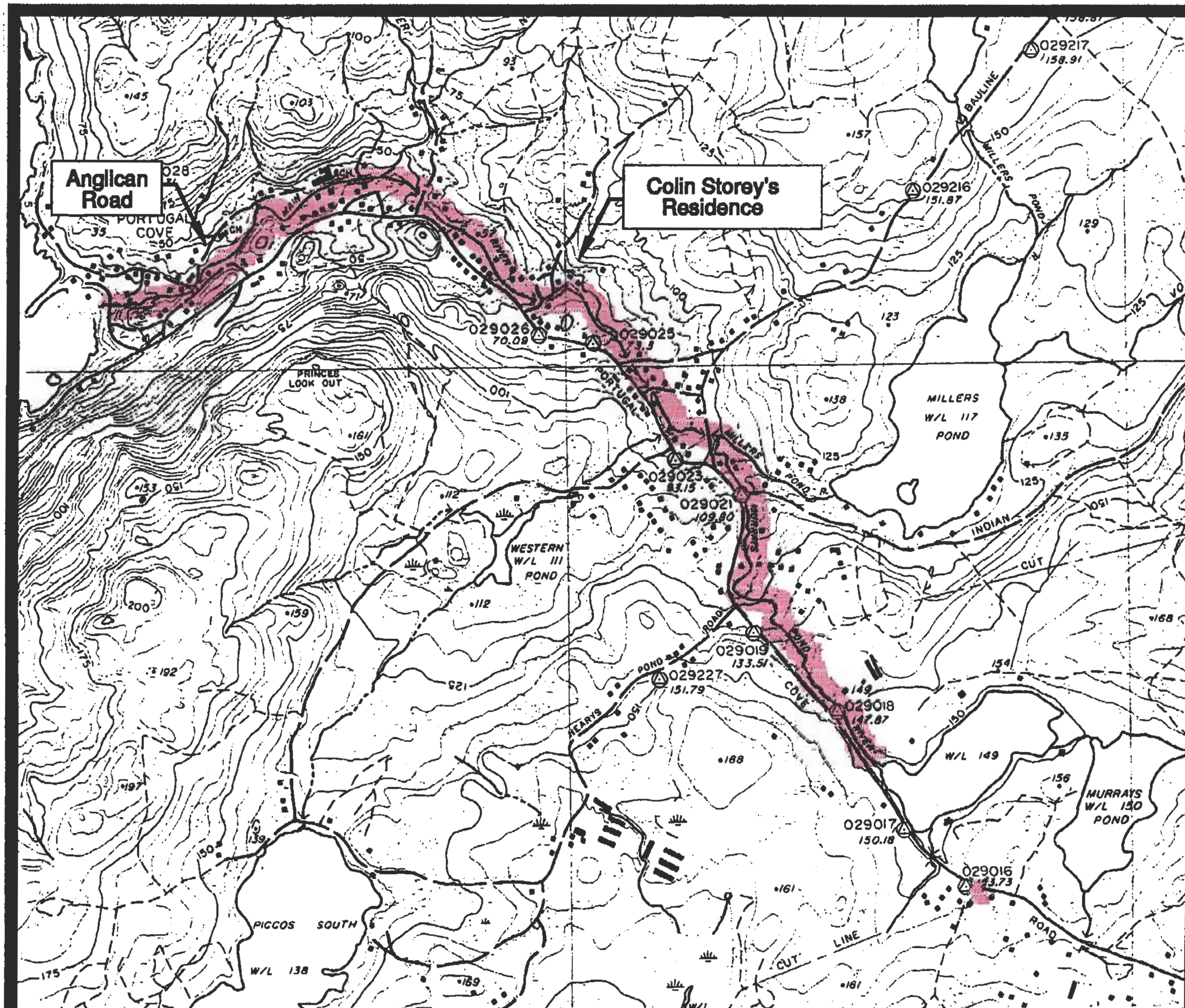


Figure 1

Murray's Pond River/Main River
Portugal Cove

River

Scale: N.T.S.

of conveying the stormwater flow and as a result the water overtopped the driveway which channelled the water onto adjacent properties belonging to Mr. Kean and Mr. Burry. A large quantity of gravel was deposited on this neighbouring property since the embankments contained no form of erosion control. After investigating, the Department of Environment recommended that the culverts be replaced with either a larger capacity culvert or a bridge to increase the flow capacity, environmental controls should be implemented to protect the embankments and the height of the approach should be adjusted to allow floodwater release over the top of the culvert away from adjacent properties. It was also recommended that the Town Council be informed of the problem and be discouraged from allowing any future development in the flood plain area. All construction and infilling should be a minimum of 15 m from the high water mark. The Department's records contain applications and approvals for replacement of the culvert with a bridge in both 1988 and 1994. A site investigation during the field work for this study indicated that the culvert has been replaced with a 5.1 m bridge.

The survey crew spoke with the residents of the house in the river upstream of Murray's Pond and they indicated no flooding incidents in the last twenty years.

2.2.2 St. Philips

A number of localized flooding events have occurred in St. Philips around Broad Cove River. The replacement of a culvert on an unnamed tributary of Broad Cove River in 1994 was done in an effort to prevent flooding.

In February 1986, another combination of ice jam and consequent accumulation of water in Broad Cove River caused flooding on the Hamelmann residence and surrounding farmland (see Figure 2). The flood waters approached the barn and posed a threat to livestock so permission was granted by the Department of Environment to use machinery to remove the ice jam. Due to

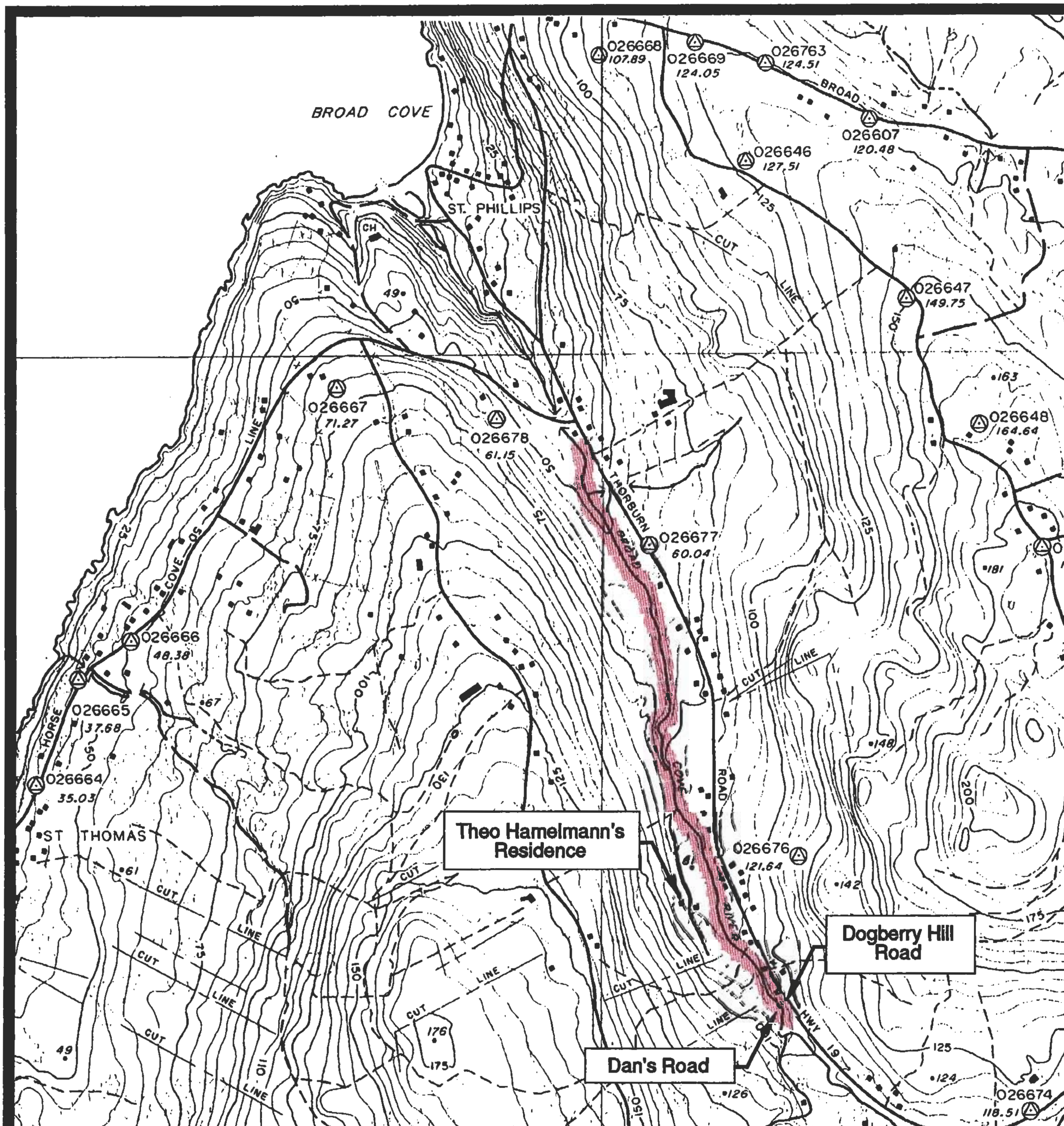


Figure 2

Broad Cove River
St. Philip's

River

Scale: N.T.S.

the location of the ice jam only a portion could be removed, however, this was sufficient to cause the flood water to subside. Conversations with the current residents and the landowners revealed that there have been no problems with flooding in the last several years.

Another ice jam at the Dogberry Hill Road bridge in 1989 resulted in flooding west and northwest of the bridge. The problem was rectified in only a few hours by clearing the ice jam with a backhoe. Only minor inconvenience and damage resulted from this incident.

A major flooding event occurred on Broad Cove River near where Dogberry Hill Road crosses the river (see Figure 2). The flooding occurred at the Cooper residence on Dan's Road in 1992. The flooding was due to an ice jam that had formed approximately 20 m upstream of the Dogberry Hill Road bridge to a height of 1.0 - 1.5 m. Water was accumulating behind the ice jam and backing up the river. As a result, approximately 75 mm of water accumulated in the sump located at the Cooper residence. It was recommended that a backhoe be used to remove the ice blockage and release the collected water. Discussions with an individual at the Cooper residence confirmed that a rise in the sump water level was a routine occurrence and that land-scaping had been carried out this year to help prevent flood water from overtopping the rear lawn. Conversations with persons at the Edwards residence and the Sharpe residence confirmed chronic problems with water flooding, however, no specific event times or details, or photographs were available.

2.2.3 Logy Bay

The Caddigan property (see Figure 3 for location) in Logy Bay was the site for several flooding events at a rate of one occurrence every two years. A personal interview with Mr. Caddigan revealed that his property had been flooded at least 7 times in a 15 year period. The flooding resulted from an undersized 900 mm diameter culvert on the unnamed stream that runs adjacent to the Caddigan residence. The addition of an additional 600 mm diameter culvert did not

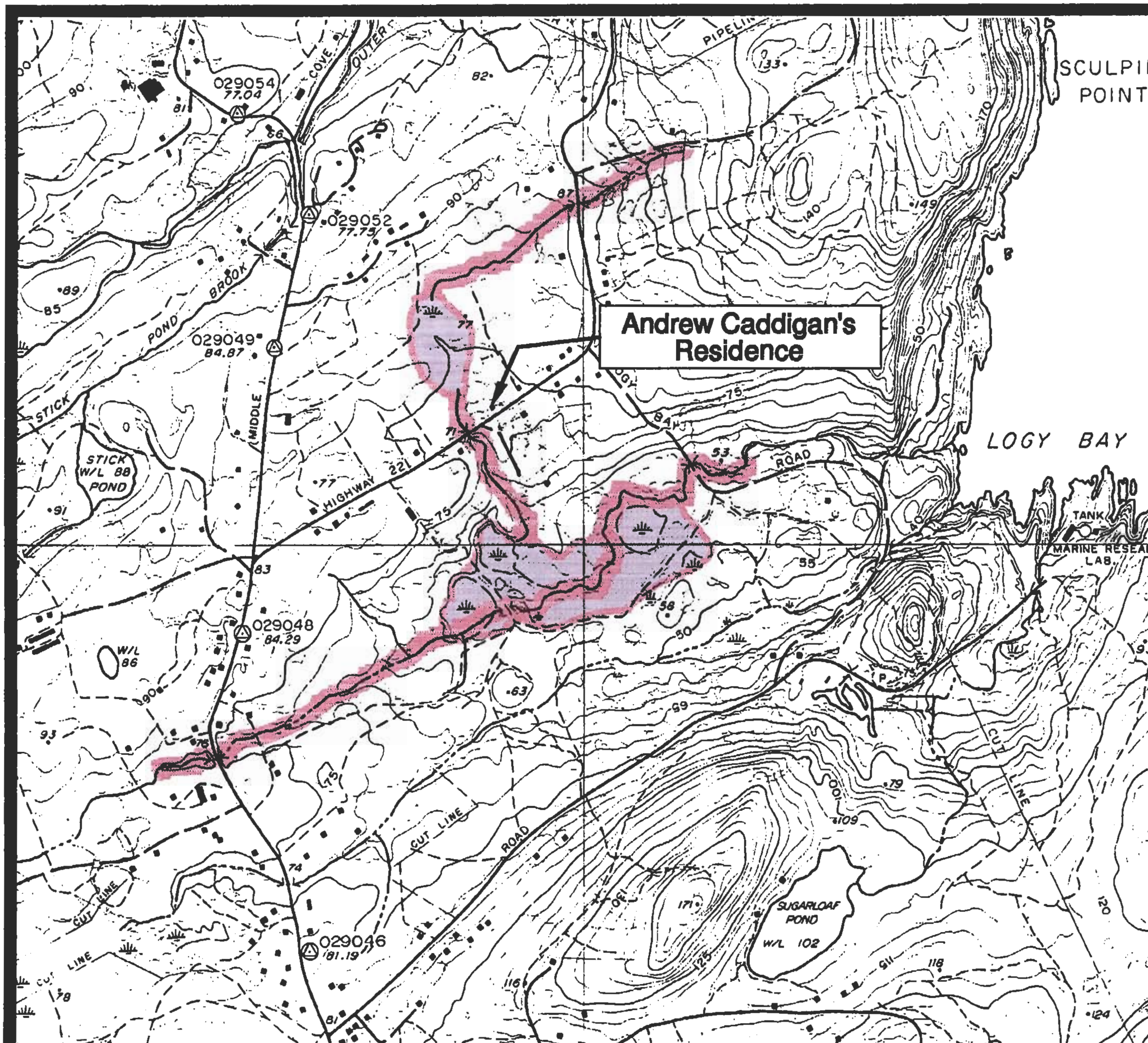


Figure 3

Unnamed River
Logy Bay

Marsh
River

Scale: N.T.S.

alleviate the problem, and in 1993 a hydrological review and analysis revealed that the culvert diameters were not sufficient for periods when they are obstructed with snow or ice. The basement elevation of the Caddigan residence is below the water levels anticipated due to a 25 year or 100 year flood, therefore elevated water levels in the river will decrease the groundwater flow gradient and the water table will subsequently elevate to such a point where it will exceed the basement floor. Approval was given in October 1994 to install a 2130 mm x 1400 mm pipe arch culvert to replace the two existing culverts. Subsequent to this installation, there have been no incidents of flooding on Mr. Caddigan's property.

The survey crew interviewed a number of residents in the flood plain area. The portion of the river that crosses the downstream end of Logy Bay Road has no history of flooding and has only had increases in water level of about 6" (150 mm) during runoff periods. Discussions with Mr. Caddigan indicated that his property periodically flooded to the elevation of his driveway. This water infiltrated into his weeping tile and progressed into his basement to a depth of about 18" (450 mm). This has not been a problem since the stream culverts were replaced in 1994. An interview with the downstream neighbour indicated that the river in that area consists mainly of bog in excess of 1 m deep and has extremely low flow and no discernable flow path.

2.2.4 Outer Cove

In January 1986, a blocked culvert at the Collision Clinic in Outer Cove (see Figure 4) caused a flood and washed out a section of Lower Road in Outer Cove. The culvert consisted of a number of open-ended oil drums placed end to end and buried. The partial collapse of a single oil drum caused a reduction in waterflow and approximately 1 week later a second oil drum apparently collapsed and blocked the flow of water. To relieve the backed up water, a temporary ditch was constructed across the Collision Clinic property. To prevent any further flooding, a 24" (610 mm f) culvert was installed to replace the temporary structure made up of oil drums. In May 1990 approval was given to replace the existing culvert which had partially collapsed with a new

24" (610 mm f) culvert. An interview with Mrs. Hickey, wife of the property owner, revealed that they have had no flooding problems since installing the new culvert, except in the case where the culvert was blocked by a plastic container that obstructed the flow.

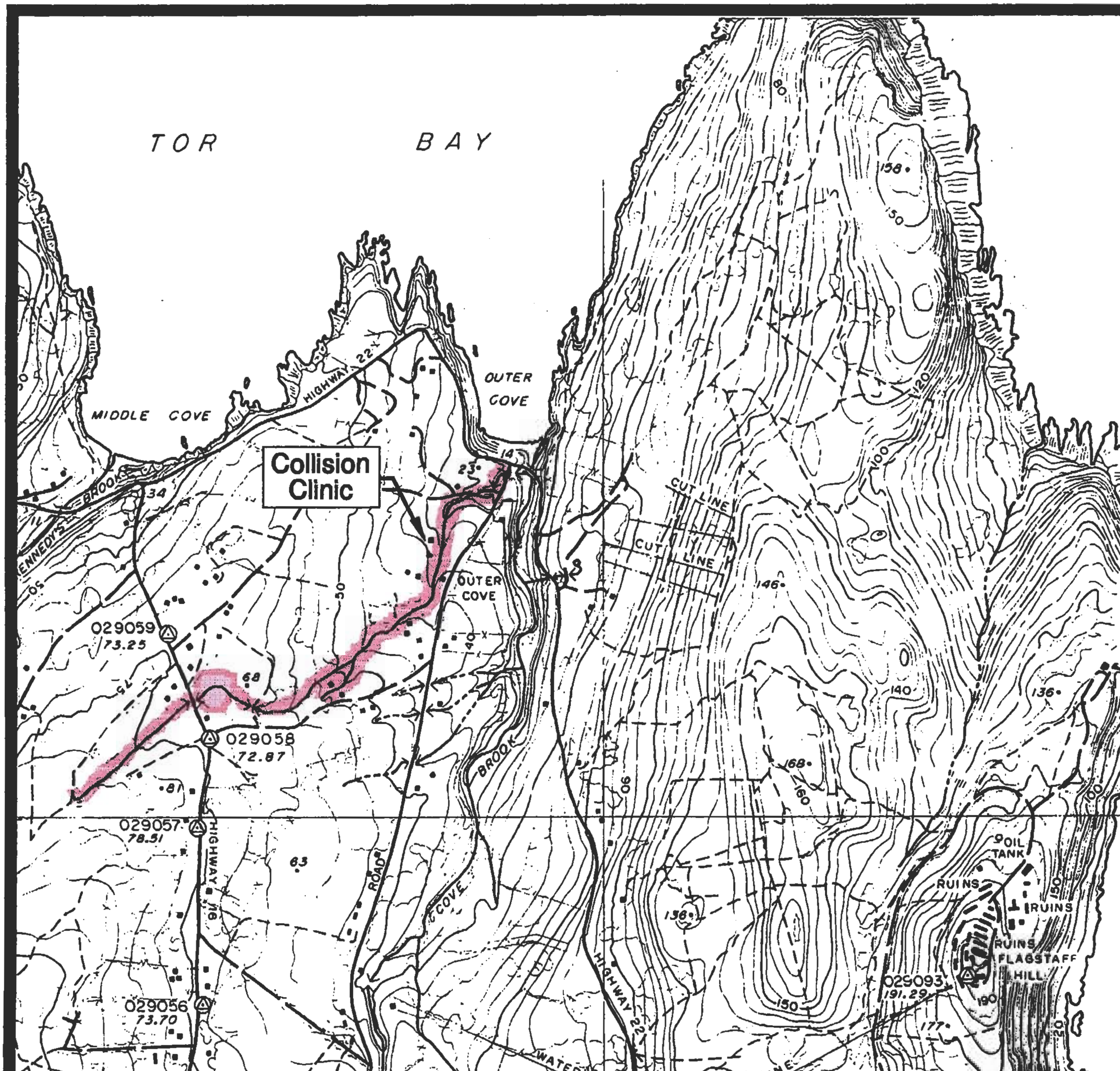


Figure 4

Unnamed River Outer Cove

Marsh
River

Scale: N.T.S.

3.0 FIELD PROGRAM

3.1 General

Field surveying commenced on 3 August 1995 with a crew consisting of a surveying technologist and a rodman. Horizontal controls were established with a Sokkisha Set 3 Total Station interfaced with a SDR Electronic Field Book. Resolving power of this system is 3 seconds. The same instrument was used to develop the stream sections and determine elevations and profiles for culverts, bridges and other manmade objects.

Temporary bench marks (TBM) were established along the survey routes using a Wild-Heerbrugg Level Model N2-70658. Levels were taken from known geodetic monuments and either checked at other geodetic monuments along the route or by a closed level set back to the starting point. The levels established at these TBM's were used to check and adjust when necessary the total station elevations when taking cross sections.

Surveying in some areas was very difficult. Dense vegetation hampered the survey and could not be cut and removed because it occurred primarily on private property. In addition local residents were not aware of the flood risk study being carried out and consequently the field crew was often interrupted by homeowners and residents with inquiries. A significant amount of time was required to converse with the public in an effort to inform the residents of the project and describe what was being done. In return, some residents were able to provide brief information about past flooding events.

In order to prevent any potential problems with the public an effort was made to inform residents through personal contacts when the survey crew would be present and what would be taking place. The scheduled four weeks to complete the four surveys was complicated by the attempt to obtain permission to survey on private property when homeowners were often absent for long periods while the crew was working.

For approximately half of the duration, primarily the first two weeks, of the field work the weather was clear and hot. This was followed by frequent days with heavy rain or drizzle which impeded the progress of the survey.

Due to the large number of artificial structures and features that had to be tied into the survey, for example in Portugal Cove there were 8 bridges and 23 culverts, many instrument setups were required on the control traverse. Because of the excessive number of setups required, the horizontal accuracy may have diminished somewhat due to error propagation.

The monuments in the study area were in poor condition and several were missing or destroyed. A significant amount of survey time was consumed in locating and accessing the remaining monuments.

The four (4) areas surveyed were Outer Cove, Logy Bay, Portugal Cove and St. Philips. Specific conditions related to surveying each area are discussed in the following sections.

3.2 Portugal Cove

The Portugal Cove survey stretched from the outlet at Murray's Pond River downstream 1,700 metres to a point near Churchill's Road Bridge on Main River (see Drawing No. 5-541-12). Murray's Pond River is a very small stream running through mainly private property. There are numerous small culverts and bridges, and a number of retaining walls on both sides of the stream. Vegetation, consisting of grass, weeds, shrubs and small trees, lines both embankments.

Also included in the survey was a portion of a stream located upstream from Murray's Pond as indicated in Figure 1. This consists of a small grassy area between two culverts where any flow passes around a house foundation, under Portugal Cove Road and eventually into Murray's Pond.

Below the confluence of Miller's Pond River, on the section called Main River, the volume and velocities of water increase significantly. There is a further increase below the confluence of

Western Pond Brook. Here, the river bottom is rocky, there is less vegetation, and the river bottom slope increases.

The whole of Main River is bounded by private property and, in some instances, fences are built to the river's edge. A number of property owners have constructed retaining walls along the river's embankment in order to stabilize them and/or increase the area of usable property.

Surveying cross sections along this river was difficult because of the many obstructions such as bridges, building and culverts. Two of the bridges are shown in Photographs 1 and 2. In addition the majority of the property surrounding the river is owned privately and the dense vegetation on the embankments could not be cut and removed so visibility was impeded.

Two of the required monuments in this area have been destroyed, namely monuments numbered 029022 and 029025. The remaining monuments numbered 029021 and 029019 are still existing, however, they are not intervisible and intermediate temporary control points were required to provide levelling control for this river.

3.3 St. Philips

Broad Cove River in St. Philips is wider than Main River in Portugal Cove and both embankments are covered with trees, bushes and other vegetation, as shown in Photographs 3 and 4. With private property stretching along the river, it was again decided that the cutting of trees should be avoided if at all possible. To minimize the requirements for cutting for alignment and cross sections, a traverse was surveyed along the riverbed. Cross sections were taken across the stream and up each embankment in locations that indicate slope and alignment changes, width changes or any known flooding locations. Along the lower reaches the riverbed slope increased, the height of embankments was greater and water velocities were higher, hence the likelihood of flooding was remote. No sections, therefore, were taken.

The roughness of the riverbed combined with algae made walking very difficult and the survey progressed at a slow pace as a result.

Sections were, as before, difficult to establish due to the fact that vegetation was present right to the river's edge. There were sections in the river that were difficult to walk and very difficult to survey due to high vertical banks up to 4.6 m.

Survey points had to be established in the river itself. The rocky nature of the riverbed made stake driving very difficult at times. Extreme caution had to be taken while transporting and setting up survey equipment to prevent maladjustments or damage. Control is very weak in this area as well.

3.4 Logy Bay

At Logy Bay a survey was carried out along two (2) streams (see Figure 3). One was a small stream originating east of Highway 22 which then crossed the highway and ran southwest for approximately 400 metres where it empties into a marshy area. There is no identifiable stream path through this area which lies in a north south direction and stretches approximately 500 metres. At the end of this marsh the stream emerges again. It then runs west of the Caddigan property, and crosses underneath Highway 22 again, as shown in Photographs 5 and 6. Beyond Highway 22, it runs in a southerly direction through pastureland for approximately 300 metres where it again empties into a low wet marshy area. There is no definable streambed in this area. Drainage from this marsh then empties into a slightly larger stream which runs in a generally northeast direction through a combination of wooded, pasture and marshy/boggy areas. The upper reaches of this stream crosses Highway 16 and then crosses Logy Bay Road before flowing into Logy Bay. The survey along this stream was from highway 16 downstream to a point approximately 100 metres beyond where the stream flows underneath Logy Bay Road. For approximately 300 metres upstream from Logy Bay Road, the stream runs through a marsh.

Surveying of these two streams was complicated by the lack of geodetic monuments. Many monuments were missing and line-of-site between the existing ones was obstructed by hydropoles. The survey crew had to establish the survey using resection procedures. Levelling using a spirit level had to be carried out over the entire survey and checked back to the starting geodetic.

3.5 Outer Cove

The survey at Outer Cove was along a very small stream running through a populated area (see Figure 4). Much of the upper reaches ran mainly through pastureland and immediately above the Collision Clinic, it ran through four lawns. Culverts are the principal means of conveying water underneath the lawns and driveways. The two most downstream culverts are shown in Photographs 7 and 8. The area surveyed extended from the outlet at Outer Cove for a distance of 1,700 metres upstream. Cross sections were taken at locations along the stream where changes in alignment, cross sectional area, or other features which might interfere with flow occurred. From the Collision Clinic downstream to the ocean, the slope increases rapidly as shown on the profile (see Drawing No. 5-541-06).

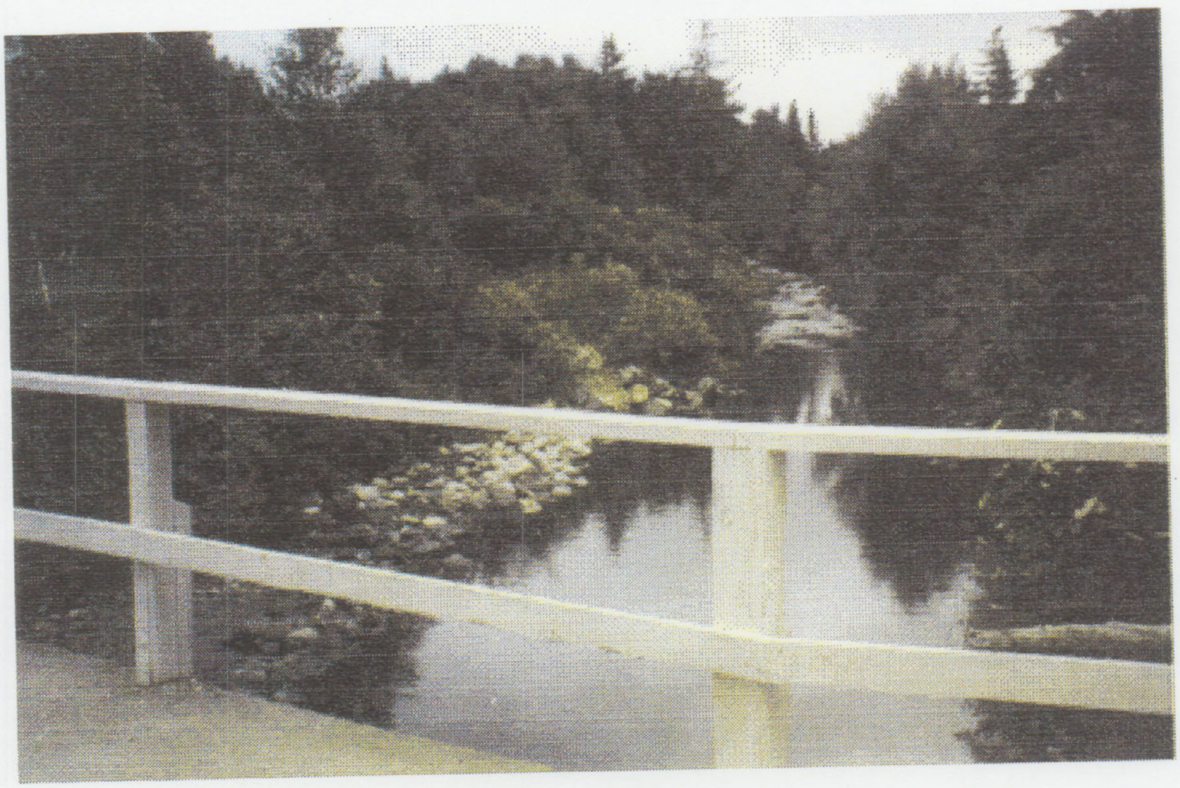
The survey crew found the stream difficult to identify in some areas. The stream only appeared as saturated ground, for example, in the low lying channel through the first 300 m. It disappeared into a flat marshy area east of Middle Cove Road and it is not until downstream of this that it takes on a defined shape similar to a road ditch.

Control was established at one end of the survey using two monuments on highway 16. Due to the imbalance of level control in this area, it was necessary to use a spirit level.

PHOTOGRAPH 1: Main River, Portugal Cove, in the background. Evidence of flooding denoted by brown patch in lawn in center of photo.

PHOTOGRAPH 2: New bridge built over old collapsed slab on Main River, Portugal Cove.

PHOTOGRAPH 3: Bridge over Broad Cove River, St. Philips, at Dogberry Hill Road.



PHOTOGRAPH 4: Bridge over Broad Cove River, St. Philips, on private residence downstream of Dogberry Hill Road.

PHOTOGRAPH 5: Andrew Caddigan's residence - location of flooding problems in Logy Bay. Note arch culvert below center of photo.

PHOTOGRAPH 6: Downstream view of arch culvert on Caddigan's Road, Logy Bay.

PHOTOGRAPH 7: Culvert through back lawn of residence above Collision Clinic, Outer Cove.

PHOTOGRAPH 8: Intake of culvert running under Collision Clinic parking lot, Outer Cove. Note smaller size of culvert compared with upstream culvert in photo 7.

4.0 HYDROLOGY STUDIES

4.1 Methodology

Two main approaches are used to determine the 1:20 and 1:100 year recurrence interval flood flows. The first approach is deterministic: the runoff from a specified input (1:20 and 1:100 year precipitation) is calculated using the characteristics of the drainage basins as parameters. Both the Rational and Soil Conservation Service (SCS) TR-55 Chart Methods are applied, which require estimation of a number of physiographic parameters including soil type, land use, and basin and channel slopes, in addition to precipitation data. The second approach is streamflow data analysis: the streamflows from nearby gauged basins are used to estimate the flow at the particular watershed of interest using single station and regional frequency analyses. The deterministic approach will be verified by applying the methodology (i.e. Rational and SCS methods) to gauged basins to determine if the estimated peak flows are similar to those estimated from frequency analysis of measured flow data. A combination of the deterministic and streamflow analysis approaches, known as the probabilistic rational method, will also be applied for comparison purposes. The results obtained using all of these methods are then compared to determine the most reasonable 20 and 100 year peak flows for use in the hydraulic analysis.

4.2 Watershed Characteristics

The watershed features relevant to this study include drainage area, hydraulic length, basin and channel slope, and soil type and land use, which are used in combination to estimate the SCS curve numbers. The procedures followed to quantify these parameters are explained, and the values obtained have been tabulated for the basins of interest in Portugal Cove, St. Philips, Logy Bay and Outer Cove.

The watershed features of two additional basins, Northeast Pond River near Portugal Cove, and North Pond Brook in Torbay, have been included in the tables, as these are nearby gauged basins. They will be used to verify the application of the deterministic approach, since the results obtained from the Rational and SCS methods can be compared with estimates of peak flows based on actual measured flow data.

In St. Philips, there were two main points of interest with a history of flooding problems, so two distinct drainage basins were considered. The larger of the two has an outlet at the bridge at the former Hamelmann residence, and will be referred to as "St. Philips I". The outlet for the other basin, referred to as "St. Philips II", is the bridge at Dogberry Hill Road, and it is contained within the former basin. The characteristics of both St. Philips basins were calculated. Additional parameters required for the application of the Department of Environment's Regional Flood Frequency Analysis equations include the drainage density, the area occupied by lakes and swamps, and area controlled by lakes and swamps. These are given for the five study basins - Portugal Cove, St. Philips (I and II), Logy Bay and Outer Cove. They were not required for the two gauged verification basins (Northeast Pond River and North Pond Brook), with the exception of the area occupied by lakes and swamps, which is required (expressed as a fraction) for the SCS TR-55 Chart Method.

4.2.1 Drainage Areas of Basins

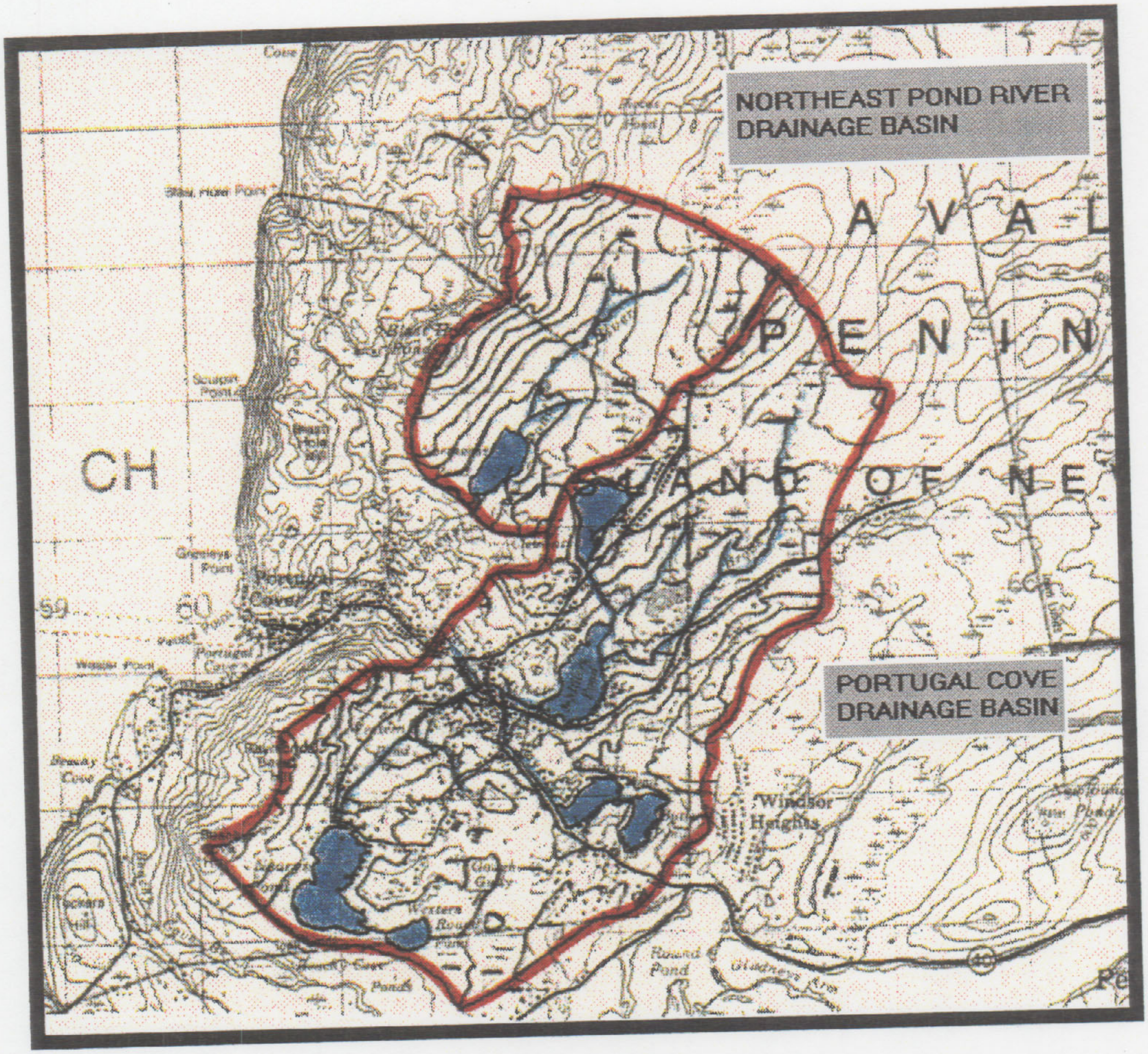
The drainage areas of the five study basins (Portugal Cove, St. Philips (I and II), Logy Bay and Outer Cove) were determined by outlining the drainage area on graph paper and manually calculating the area using 1:50,000 scale topographic mapping. Portugal Cove and Northeast Pond River are adjacent watersheds, and are shown together in Figure 5. Figure 6 shows the outlined drainage areas of both basins in St. Philips, and Figure 7 shows the drainage areas for the Logy Bay and Outer Cove basins. The North Pond Brook basin is reproduced in Figure 8 from information provided by the Surface Water Section of the Department of Environment's Water Resources Division. The drainage areas are summarized in Table 1.

Table 1: Watershed Drainage Areas

WATERSHED	DRAINAGE AREA (km^2)
Portugal Cove	10.97
St. Philips I	18.46
St. Philips II	17.54
Logy Bay	1.46
Outer Cove	0.82
Northeast Pond River*	3.63
North Pond Brook*	6.7

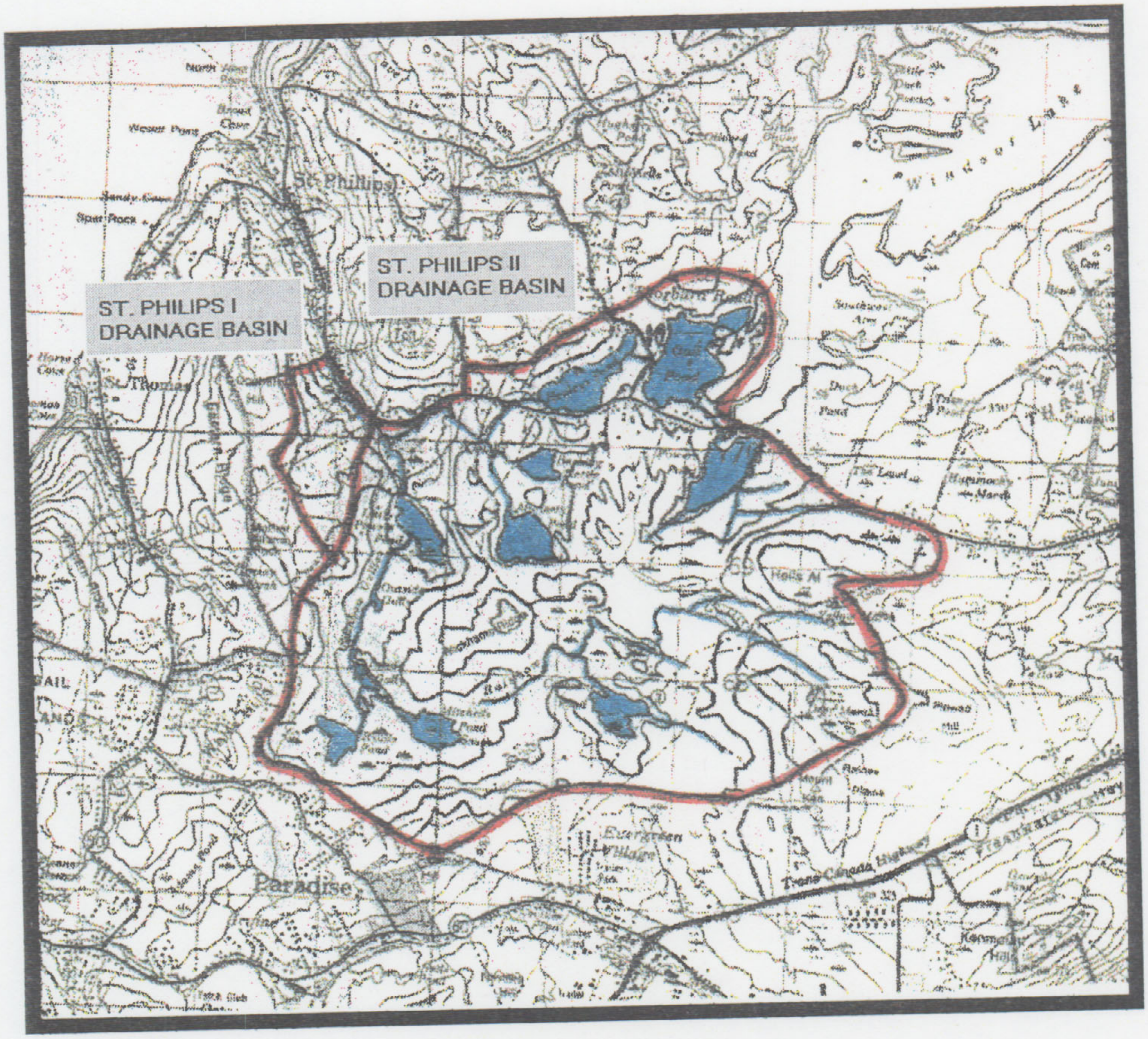
* Drainage areas were provided with the hydrometric station data.

Figure 5: Portugal Cove and Northeast Pond River Drainage Basins[1]



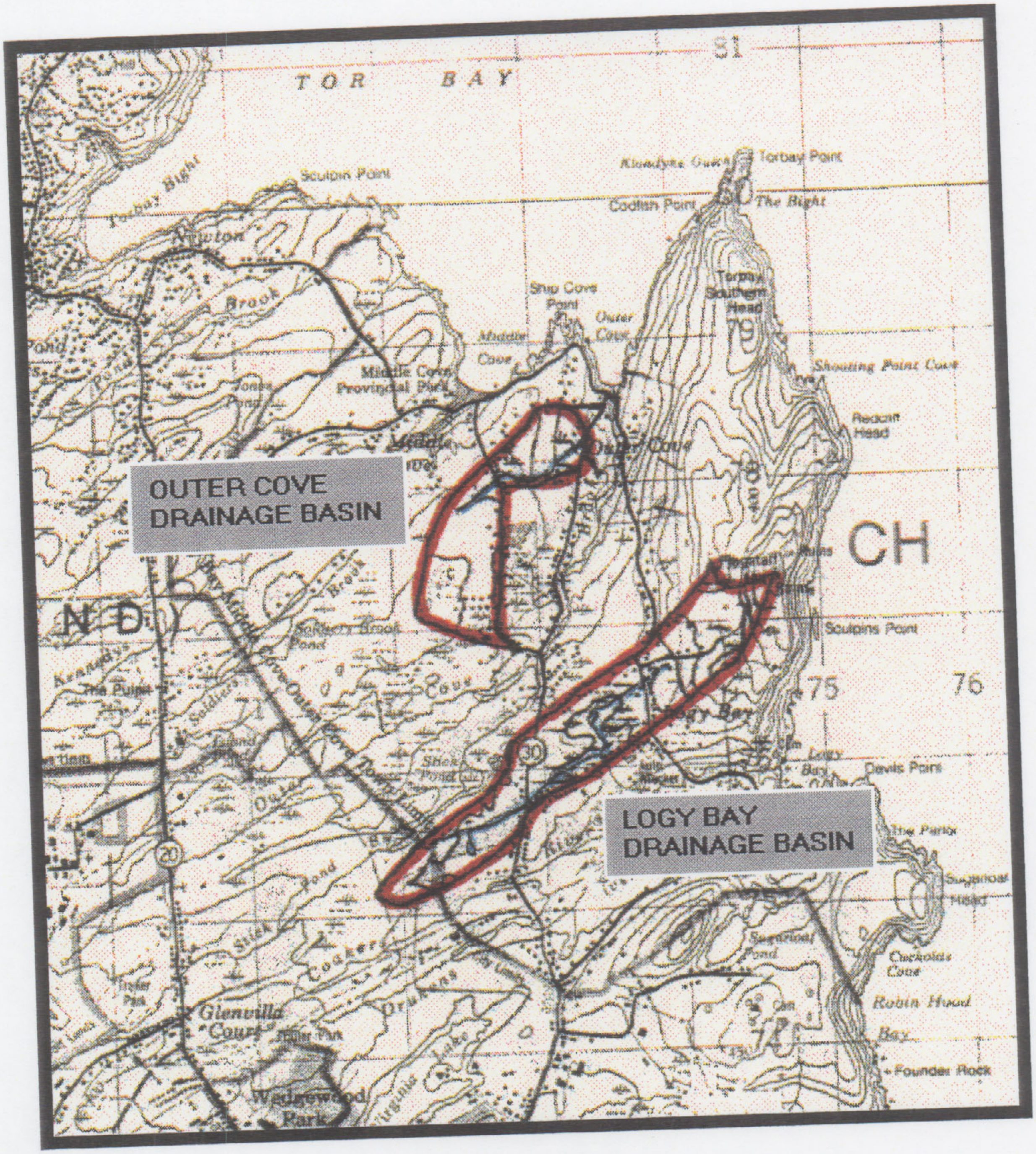
[1] **Scale: N.T.S.**

Figure 6: St. Philips Drainage Basins[1]



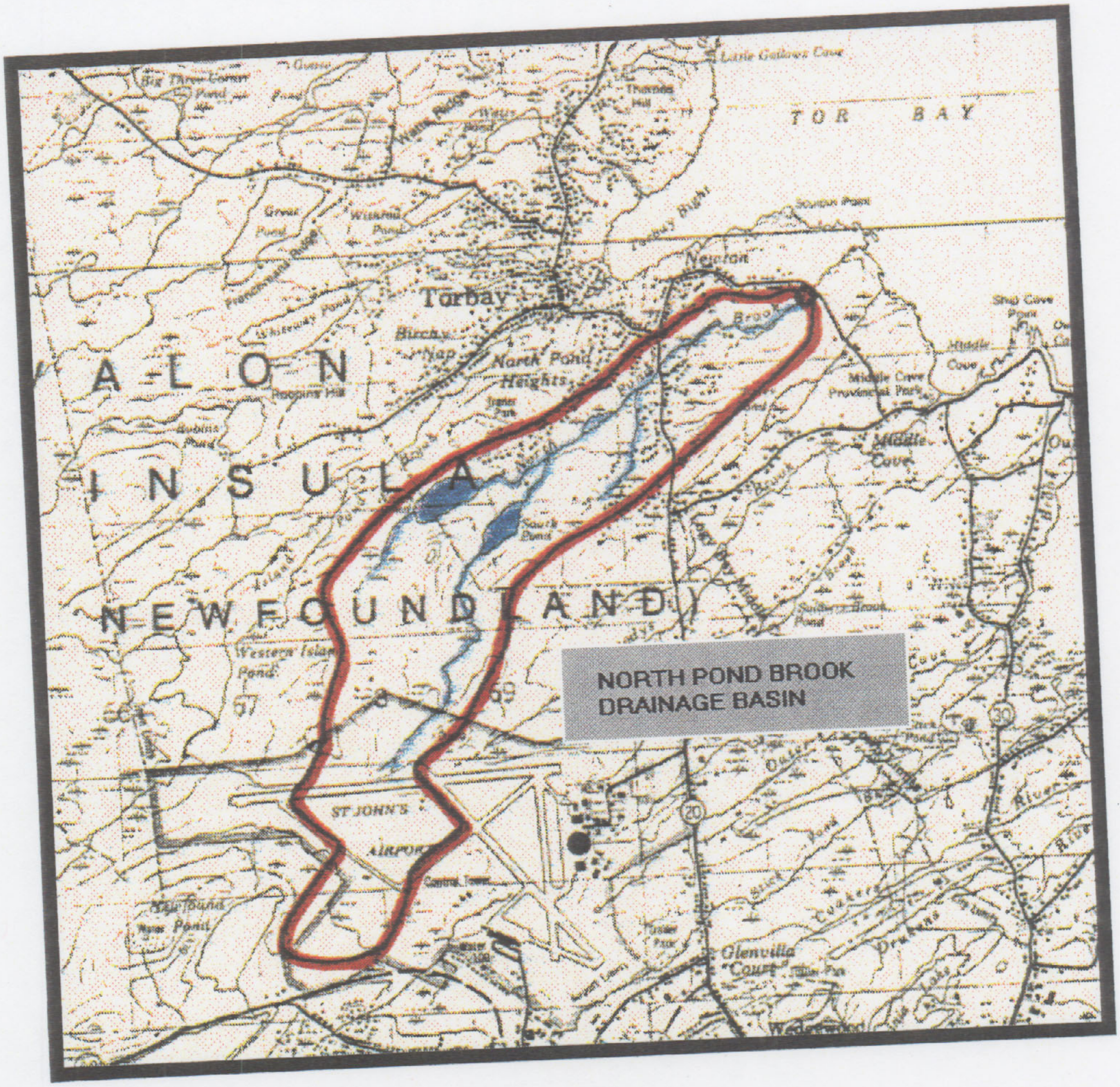
[1] **Scale: N.T.S.**

Figure 7: Logy Bay and Outer Cove Drainage Basins[1]



[1] **Scale: N.T.S.**

Figure 8: North Pond Brook Drainage Basin[1]



[1] **Scale: N.T.S.**

4.2.2 Hydraulic Length

The hydraulic length is the distance from the outlet, or point of design, to the hydrologically most remote point in the basin. The hydraulic lengths were determined manually by taking the longest route along the main channel to the basin divide. The lengths are given in Table 2:

Table 2: Watershed Hydraulic Lengths

Basin	Hydraulic Length (m)
Portugal Cove	5000
St. Philips I	8250
St. Philips II	7500
Logy Bay	2000
Outer Cove	1950
Northeast Pond River*	2630
North Pond Brook	7400

* Value obtained from RFFA

4.2.3 Basin and Channel Slope

Both the average basin slope and the main channel slope are required in the deterministic models. These parameters are calculated using methods outlined in the Roads and Transportation Association of Canada (RTAC) Drainage Manual.

The average basin slope is required to determine the time of concentration, and can be approximately determined as follows (RTAC, 1982):

[1] Place a transparent grid over a contoured map of the basin.
[2] Measure the total length, LV, of the vertical grid lines within the watershed.
[3] Measure the total length, LH, of the horizontal grid lines within the watershed.
[4] Count the number of times, XV, that the vertical grid lines are intersected by contour lines.
[5] Count the number of times, XH, that the horizontal grid lines are intersected by contour lines.
[6] Note the contour interval, h, of the map used.
[7] Calculate the vertical slope as the product of XV and h divided by LV.
[8] Calculate the horizontal slope as the product of XL and h divided by LH.
[9] Determine the average land slope by taking the average of the vertical and horizontal slopes.

This procedure was followed for each of the study basins and for the two verification basins. Instead of using a transparent grid, the contours were outlined on the same graph paper previously used for determining the drainage areas. The calculated average basin slopes are given in Table 3.

The main channel slope is the slope of the branch along which the time of concentration is required. This branch should include any lakes through which the main channel passes, and should extend to the upper limit of the effective watershed. The average slope method was used in this study. This method requires that the channel slope be determined between points 85% and 10% of the total main channel length , to avoid the distorting effects of the steep upper portion of a watershed and of possibly steep or flat lower portion. The main channel slopes were determined in this manner for each of the study basins and for North Pond Brook. The slope value for Northeast Pond River was obtained from the Regional Flood Frequency Analysis User's Guide (Water Resources Division (WRD), 1990). The main channel slopes are given in Table 3.

Table 3: Average Basin and Main Channel Slopes

Basin	Average Basin Slope (%)	Main Channel Slope (%)
Portugal Cove	4.74	2.03
St. Philips I	4.12	1.38
St. Philips II	4.02	1.35
Logy Bay	4.53	4.06
Outer Cove	3.00	3.33
Northeast Pond River	5.38	2.42*
North Pond Brook	4.21	1.87

* Value given in RFFA.

4.2.4 Soils and Land Use

The hydrologic response of a watershed is significantly affected by soil type and thickness, as these parameters are a major influence on infiltration rates. The classification of soils into the four SCS hydrologic soils groups is required for the SCS TR-55 Chart Method to determine peak flow. It is also required for the SCS Curve Number Method in order to determine the time of concentration for the Rational Method. The four hydrologic soil groups, A, B, C, and D, range from low to high runoff potential (from high to low infiltration rates). Soils with properties between those of each group are classed as AB, BC, and CD. Detailed descriptions of each of the main soil types are given in the Appendix A.

Two sources of soils information were collected. The Geological Survey of Canada provided a surficial geology map of the study area, a fourth edition Landform Classification map of St. John's updated by the Surveys and Mapping Branch of the Department of Energy, Mines and Resources from aerial photographs taken in 1966. This map showed that the predominant surficial geology consists of a combination of bedrock concealed by vegetation, bog and glacial till veneer (less than 2 m thick) with particle sizes ranging from boulder to silt/clay. The Broad Cove River valley of St. Philips consists of a glaciofluvial fan (fine grained sand to coarse grained cobbly gravel).

The second source of soil information was the Newfoundland Soil Survey, Report No. 3, Soils of the Avalon Peninsula (1981) obtained from Agriculture Canada. The soil types described in this document were selected for use in the study because the correspondence between these types and the SCS hydrologic soil groups had already been established in the Urban Hydrology Study of the Waterford River Basin Watershed Modelling Report, HYMO 1988 (WRD, 1988). The above-mentioned report was used as a guideline for the classification of soil types into hydrologic soil groups for use in the analysis.

The soil series present in each of the study areas was determined using a 1:100,000 scale map included in the Soil Survey Report which was enlarged to a 1:50,000 scale to allow for easier estimation of the soil type areas. The Cochrane series of soils were predominant throughout the drainage basins, followed by some Torbay, Organic, and Pouch Cove soils. The Bauline series was present in part of the Portugal Cove basin, and some Red Cove series soils were found in the Logy Bay basin. Descriptions of the soil series types, the soil classification symbol convention used in the Soil Survey, the soil combinations present in the study areas, and their corresponding hydrologic soil group classifications are given in Appendices B, C, D, and E, respectively. The distribution of the hydrologic soil groups within each watershed is given in Table 4.

Table 4: Distribution of Soil Groups within Watersheds as a Percentage of Drainage Area

Soil Group	Portugal Cove	St. Philips I	St. Philips II	Logy Bay	Outer Cove	Northeast Pond River	North Pond Brook
A	0.0	0.0	0.0	0.0	0.0	0.0	0.0
AB	5.0	0.0	0.0	0.0	0.0	31.4	0.0
B	23.6	2.9	3.1	0.0	0.0	0.0	0.0
BC	18.9	7.3	6.3	0.0	0.0	18.4	20.5
C	40.8	86.8	87.5	77.8	100.0	46.0	72.9
CD	3.0	0	0.0	13.0	0.0	0.0	2.6
D	8.7	3.0	3.1	9.2	0.0	4.2	4.0
Total	100.0	100.0	100.0	100.0	100.0	100.0	100.0

The runoff characteristics of a watershed are also determined to a great extent by land use. Land use information was estimated from 1:50,000 scale topographical maps. The land use categories used in this study include pasture/barren, residential (low density housing, impermeability 16 - 30%), forests, paved roads, gravel roads, lakes/ponds, and swamps. The distribution of land use within each watershed is summarized in Table 5. No others are present in significant quantities in the study areas.

Table 5: Land Use Distribution within Watersheds as a Percentage of Drainage Area

Land Use	Portugal Cove	St. Philips I	St. Philips II	Logy Bay	Outer Cove	Northeast Pond River	North Pond Brook
Pasture/ barren	14.1	1.6	1.4	6.9	49.8	4.0	17.0
Residential	10.8	2.2	1.8	7.5	9.1	0.0	5.9
Forests	54.1	77.3	76.9	69.4	29.8	74.7	59.7
Paved Roads	0.8	0.2	0.2	1.0	1.3	0	1.4
Gravel Roads	0.7	0.4	0.4	0.7	0.3	0.3	0.8
Lakes/Ponds	6.4	8.6	9.0	0	0	3.2	2.7
Swamps	13.1	9.7	10.3	14.5	9.7	17.8	12.5
TOTAL	100.0	100.0	100.0	100.0	100.0	100.0	100.0

In addition to soil type and land use, the level of soil moisture significantly affects both the volume and rate of runoff. Three antecedent moisture conditions (AMC's) are described by the Soil Conservation Service for use in runoff modelling and are labelled AMC I, II and III. The lowest runoff potential occurs in the case of AMC I, in which soils are dry but not to the wilting point. Average soil moisture conditions are classified as AMC II, and AMC III represents saturated soil conditions having the highest runoff potential, in which heavy rainfall, or light rainfall and low temperatures have occurred within the last five days. Average soil moisture conditions (AMC II) were assumed in this study, for reasons explained in the following section.

The curve number indicates the percentage of precipitation falling on the basin which contributes to direct runoff. It is estimated based on an evaluation of land use, soil type and thickness, and antecedent moisture conditions. The curve number is required for both the Rational (to determine the time of concentration), and the SCS TR-55 Chart Methods of determining peak flows. The curve numbers corresponding to each soil type for the land use classifications previously outlined are given in Table 6 for antecedent moisture condition II. These values were obtained from the RTAC Drainage Manual (RTAC, 1982) and the Watershed Modelling Report HYMO, 1988 (WRD, 1988). The original source in both cases was the Soil Conservation Service National

Engineering Handbook, Part 4, "Hydrology" from the U.S. Department of Agriculture. The use of a value of 100 for ponds/lakes and swampy areas implies that all precipitation on that area will contribute directly to runoff. This would only be the case if there is no storage - the ponds are at their highest level, and the swamps are completely saturated - a very conservative assumption. The Drainage Manual (RTAC, 1982) recommends that 100 be used in special cases only. However, the value of 100 for water bodies and swamps was used in the Urban Hydrology Study of the Waterford River Basin (WRD, 1988), and is also used in this study. Because this conservatism is already built into the curve number calculations, it was decided that average soil moisture conditions (AMC II) would be sufficiently representative of the study basins. Furthermore, the use of AMC III in combination with the use of 100 for water bodies and swamps results in unrealistically high curve number values (greater than 90).

Table 6: Curve Numbers for each Land Use and Soil Type

	SOIL GROUP						
Land Use	**A**	**AB**	**B**	**BC**	**C**	**CD**	**D**
Pasture/barren	39	50	61	68	74	77	80
Residential	61	73	76	80	84	86	87
Forests	25	40	55	63	70	74	77
Paved Roads	74	79	84	87	90	91	92
Gravel Roads	72	77	82	85	87	88	89
Lakes/ponds	100	100	100	100	100	100	100
Swamps	100	100	100	100	100	100	100

The curve numbers calculated for each basin are given in Table 7. The spreadsheets used to calculate the curve numbers for each individual basin show the breakdown of land use within each soil type. These are given in Appendix F.

Table 7: Curve Numbers for each Basin

Basin	Curve Number (CN)
Portugal Cove	74
St. Philips I	75
St. Philips II	76
Logy Bay	76
Outer Cove	76
Northeast Pond River	66
North Pond Brook	76

4.2.5 Other Relevant Characteristics

Additional parameters required for the application of the Regional Flood Frequency Analysis equations developed by the Water Resources Division of the provincial Department of Environment and Lands include the drainage density (the total length of all streams in the watershed divided by the drainage area), the area occupied by lakes/swamps, and the area controlled by lakes and swamps. The latter of these parameters is more complex to determine. A lake or swamp must have a surface area equal to at least one percent of the drainage area to the outlet of the lake or swamp for the runoff from the associated drainage area to be considered controlled. The values of these parameters are given in Table 8 for each basin.

Table 8: Additional Watershed Parameters Required for Regional Flood Frequency Analysis

Basin	Drainage Density DRD (1/km)	Area Occupied by Lakes/Swamps (km^2)	Area Controlled by Lakes/Swamps (km^2)
Portugal Cove	1.4	2.1	9.6
St. Philips I	1.1	3.4	16.2
St. Philips II	1.1	3.4	16.2
Logy Bay	2.2	0.2	1.5
Outer Cove	1.4	0.1	0.2
Northeast Pond River	1.0	0.8	3.63
North Pond Brook	Not Required	1.0	Not Required

4.3 Precipitation Data

The available rainfall intensity-duration-frequency data at the St. John's Airport location (compiled and prepared by the Hydrometeorology and Marine Division of the Atmospheric Environment Service (AES)), was used in this analysis. Data from 1949 to 1990 was included in their frequency analysis, which provided rainfall amounts (mm) and rates (mm/hr) for 5 minute, 10 minute, 15 minute, 30 minute, 1 hour, 2 hour, 6 hour, 12 hour, and 24 hour durations corresponding to return periods of 2, 5, 10, 25, 50, and 100 years. The key return periods for this study are 20 years and 100 years. The 20 year values are determined through linear interpolation between the 10 and 25 year values. This data is given in Appendix G. A factor ranging from 1.1 to 1.5 can be used to account for snow melt contribution. This factor is suggested based on our team experience. Since we have chosen a conservative estimate of flow (in section 4.7), we felt it was appropriate to use a snow melt factor in the lower end of the range. Therefore the rainfall amount will be multiplied by an estimated factor of 1.20 to take into account runoff due to snowmelt.

4.4 Deterministic Approach

4.4.1 Rational Method

The main parameters required for the Rational Method are the runoff coefficients (which depend on soil type and land usage), the drainage area, and the rainfall intensity, as expressed in the Rational formula (Equation 1) as follows:

Equation 1:

$$Q = 0.00278 \times C \times I \times A$$

where Q = peak flow in m^3/s

C = runoff coefficient

I = rainfall intensity in mm/h for a storm of duration equal to the time of concentration

A = effective area of drainage basin, hectares (ha)

Since the runoff coefficient is representative of the integrated effects of soil properties, ground cover, slope of the terrain, depression storage, storm rainfall and antecedent rainfall, considerable judgement is required to select a suitable runoff coefficient for use in the Rational method. If a watershed has variable soils or land uses, the overall runoff coefficient can be calculated by adding the individual products of the sub-areas and their respective runoff coefficients, and then dividing by the total watershed area.

The rural runoff coefficients were tabulated in the Drainage Manual for four different soil descriptions ranging from tight clayey soils (low infiltration) to coarse well drained sands and gravels (high infiltration). These descriptions were matched with their corresponding SCS hydrologic soil group (A for high infiltration, D for low infiltration), since these had already been mapped for the curve number calculation. The urban runoff coefficients (for residential paved and unpaved roads) were not associated with any particular soil condition in the references consulted, and were considered to be constant across all soil types in this study, as the percentages of urban areas were generally quite low. The runoff coefficients assumed in the study for each land use / soil group combination are given in Table 9.

Table 9: Runoff Coefficients (C) for each Land Use and Soil Group

Land Use	Soil Group						
	A	AB	B	BC	C	CD	D
Pasture/barren	0.15	0.20	0.25	0.30	0.35	0.40	0.45
Residential	0.33	0.33	0.33	0.33	0.33	0.33	0.33
Forests	-0.05	0.075	0.20	0.25	0.30	0.35	0.40
Paved Roads	0.83	0.83	0.83	0.83	0.83	0.83	0.83
Gravel Roads	0.70	0.70	0.70	0.70	0.70	0.70	0.70
Lakes/ponds	1	1	1	1	1	1	1
Swamps	1	1	1	1	1	1	1

The spreadsheets used to calculate the runoff coefficients are presented in Appendix F. For Portugal Cove and St. Philips I, a value of 0.43 was calculated. The runoff coefficient for St. Philips II was 0.44; for Outer Cove it was 0.40, and for Northeast Pond River it was 0.37. For Logy Bay and North Pond Brook, the runoff coefficient was 0.42.

The time of concentration is the time required for storm runoff to travel from the most remote point of the basin to the site in question. Since it is the most difficult parameter to estimate (RTAC, 1982), various methods will be applied and the results compared. The RTAC Drainage manual describes five methods of predicting time of concentration: The Airport Drainage Method, the SCS Upland Method, the SCS Curve Number Method, the Bransby-Williams Formula and the Channel Velocity Method.

The Airport Drainage Method was applied to all of the study basins and the two gauged verification basins. Although intended for estimating inlet times of airfield drainage systems, it is applicable to small rural basins and simple urban systems. The formula takes into account the runoff coefficient, slope and distance according to Equation 2.

Equation 2:

$$t_c = 3.26[(1.1 - C)\ L^{0.5} / S^{0.33}]$$

where: t_c = time of concentration (minutes)

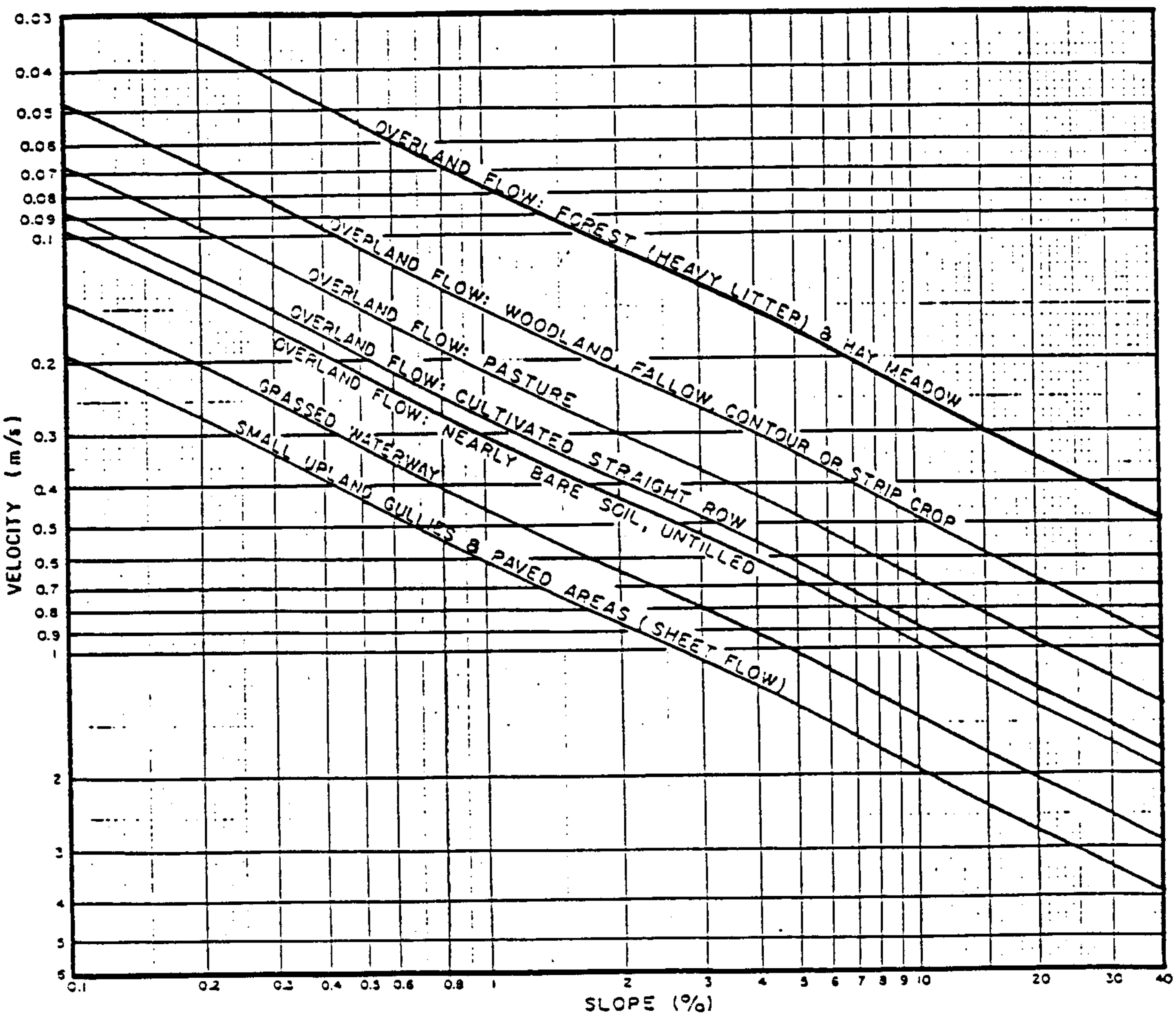
C = runoff coefficient

L = distance travelled (m)

S = slope of travel path (%)

The SCS Upland Method applies to overland flow and flow in gullies and grassed waterways. It requires basin slope and land usage as input parameters. The velocity of flow, and hence travel time, can then be read from a nomograph shown in Figure 9. This method was applicable only to the watersheds in Logy Bay and Outer Cove. In Logy Bay, the velocity for small upland gullies was used for the channel portion (1000 m), and the velocity for woodland overland flow was used for the remaining 1000 m to the basin divide. In Outer Cove, the velocity for small upland gullies was used for the first 1150 m (channel), and the velocity for pasture overland flow was used for the other 800 m to the basin divide.

Figure 9: SCS Upland Method for Estimating Time of Concentration



The SCS Curve number method of determining lag times takes into account soil type, land use, slope and travel distance, and can be used for a wide range of conditions. Furthermore, this method produces results which are more realistic than those given by other methods (RTAC, 1982). It was applied to all of the basins. The lag time, t_L, in hours, is given by Equation 3, and S in Equation 3 is given by Equation 4. The lag time, t_L is converted to the time of concentration, t_c, in hours, by multiplying by a factor of 1.7.

Equation 3:

$$t_L = \frac{L^{0.8}\ (0.039\ S_{CN} + 1)^{0.7}}{735\ Y^{0.5}}$$

Equation 4:

$$S_{CN} = 254\ (\frac{100}{CN} - 1)$$

and L = travel distance to head of basin, m

CN = curve number, should be between 50 and 98

Y = average slope of land in basin (%)

The fourth alternative is the Bransby-Williams formula which takes into account shape, length and slope but not soils and land use. The formula is given by Equation 5:

Equation 5:

$$t_c = \frac{0.605\ L}{S^{0.2}\ A^{0.1}}$$

where: t_c = time of concentration, hours

L = gross length of main channel to head of basin, km

S = net slope of main channel, %

A = watershed area, km^2

The Channel Velocity Method calculates time of concentration from the estimated velocity of flow over the length of the water. This overall velocity can be estimated from experience with similar watersheds, or from a summation of travel times derived from an application of Manning's formula to successive reaches of the channel (RTAC, 1982). The Channel Velocity Method was not employed directly in the study, but was used as a quick check to determine whether the velocities obtained from the times of concentration derived by other methods were reasonable values. This was done by dividing the main channel (hydraulic) length by the time of concentration to get an average watershed velocity.

The times of concentration for each of the watersheds using each method are given in the following table, using the required parameters from Table 9.

Table 10: Times of Concentration using Various Methods (hr)

Watershed	SCS Curve Number Method	Bransby - Williams Formula	Airport Drainage Method	SCS Upland Method
Portugal Cove	2.76	2.07	2.04	-
St. Philips I	4.30	3.50	2.97	-
St. Philips II	3.92	3.21	2.81	-
Logy Bay	1.28	0.88	1.04	1.16
Outer Cove	1.54	0.95	1.13	0.91
Northeast Pond River	1.93	1.16	1.52	-
North Pond Brook	3.79	3.27	2.59	-

The rainfall intensity is then obtained from the intensity - duration - frequency curve, using a duration equal to the calculated/estimated time of concentration, and the desired frequency (1:20 or 1:100 years). The procedure is described in Section 4.3. The peak flows from the Rational Method are summarized in Table 11. The detailed spreadsheets are in Appendix H.

Table 11: Peak Flows in m^3/s using Rational Method with Different Times of Concentration

Basin	SCS Curve Number t_c		Bransby - Williams t_c		Airport Drainage t_c		SCS Upland t_c	
	Q20	Q100	Q20	Q100	Q20	Q100	Q20	Q100
Portugal Cove	28.3	36.4	30.9	39.8	31.0	40.0	-	-
St. Philips I	38.3	48.8	43.2	55.3	46.4	59.6	-	-
St. Philips II	39.5	50.5	43.7	56.1	46.1	59.2	-	-
Logy Bay	5.2	6.5	6.3	8.0	5.5	7.0	5.3	6.8
Outer Cove	2.5	3.2	3.2	4.0	2.9	3.6	3.3	4.2
Northeast Pond River	9.8	12.6	12.5	15.9	11.3	14.3	-	-
North Pond Brook	14.7	18.8	15.8	20.3	17.3	22.2	-	-

4.4.2 Soil Conservation Service (SCS) TR-55 Chart Method

There are several SCS methods available for determining peak flow. These are based on the SCS rainfall-runoff relation, as given in Equation 6:

Equation 6:

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

where: Q = volume of runoff expressed as depth in inches
P = volume of precipitation expressed as depth in inches
I_a = initial abstraction expressed as depth in inches
S = the potential maximum retention expressed as depth in inches.

Initial abstraction is a function of land use, treatment, and condition; interception; infiltration; depression storage; and antecedent soil moisture. It is estimated as equal to 20% of S based on empirical analysis (McCuen, 1982). Replacing I_a with 0.2S in Equation 7 yields:

Equation 7:

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

in which S is estimated by Equation 8:

Equation 8:

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

where CN is the runoff curve number.

Determination of the runoff volume therefore requires as input the curve number, based on soil type, land usage and antecedent moisture conditions, and the precipitation depth for the desired return period for a given storm duration. The peak discharge can then be determined using a number of procedures. The TR-55 Chart Method, as outlined in McCuen's guide to hydrologic analysis using SCS methods (McCuen, 1982) was selected for use in this study. This method is based on a 24-hour storm volume and a type II storm distribution (a dimensionless rainfall distribution developed by the SCS for use in the United States). The required steps are outlined in the Computation Sheet shown in Figure 10.

Figure 10: Computation Sheet for TR-55 Chart Method

PROJECT ______________________ Computed By ______ Date ________
Checked By ______ Date ________

1. Required Input

A = ________ Acres : Drainage Area
T = ________ Years : Design Frequency (return period)
P = ________ Inches: Rainfall depth for 24-hour, T-year event
Y = ________ % : Average watershed slope
CN = ________ : Runoff Curve Number

2. Compute Volume of Runoff, Q

Q = ________ Inches: Use CN and P as input to Fig. 5

3. Watershed Shape Adjustment (Optional: if adjustment is <u>not</u> made, set EA = A)

HL = ________ feet : Hydraulic Length
EA = ________ Acres : Equivalent Drainage Area (use Fig. 10)
HF = ________ : HF = A/EA

4. Obtain Unit Peak Discharge, QU

QU = ________ cfs/inch Q : Use EA with Fig. 11 (Sheet 1, 2, and 3 for flat, moderate, and steep slopes, respectively)

5. Watershed Slope Interpolation Factor, SF (Optional: if adjustment is <u>not</u> made, set SF = 1.0)

SF = ________ : Use Y and EA with Table 7

6. Ponding and Swamp Storage Adjustment Factor, PF (Optional: if adjustment is <u>not</u> made, set PF = 1.0)

PPS = ________ % : % of Ponds and Swampy Area (Based on actual drainage area A)

Location in watershed (check one):
Design Point (6-a)___; Center or Spread out (6-b)___; Upper Reaches (6-c)___

PF = ________ : Use PPS and T with Table 6-a, 6-b, or 6-c.

7. Peak Discharge QP, Calculation with Adjustments

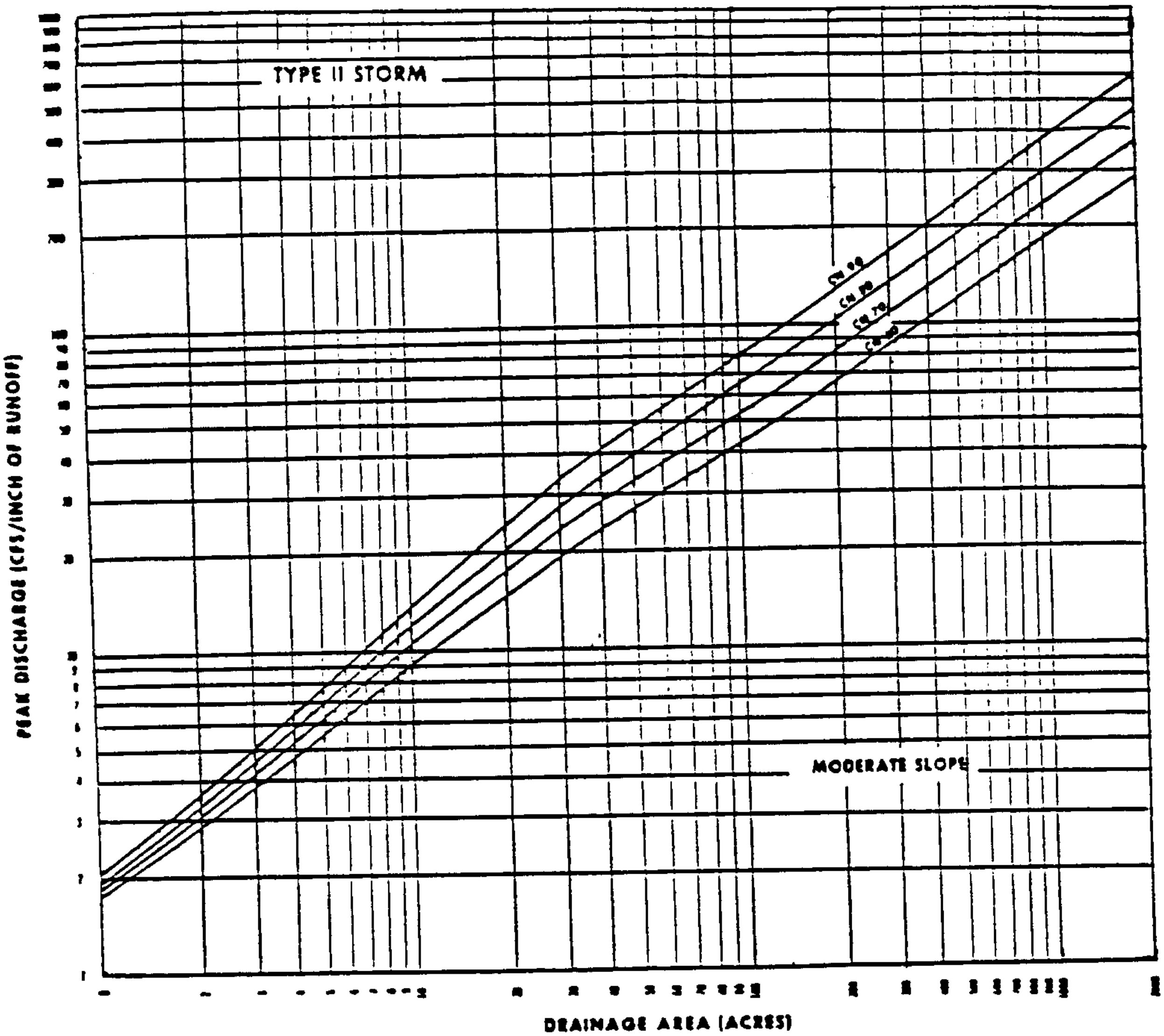
QP = QU x Q x HF x SF x PF
= ______ x ______ x ______ x ______ x ______
= ______ cfs

The rainfall depth, P, for 24-hour 20 and 100 year events were determined using the Atmospheric Environment Services rainfall intensity-duration-frequency data for the St. John's airport. The 24 hour rainfall amount for a 20 year return period was 93 mm, determined by interpolating between the values for the 10 and 25 year return periods. For the 100 year return period, the value of 113.6 mm was read directly from the table. Both of these values were multiplied by 1.2 (a 20% increase) to account for the additional water available for runoff due to snowmelt, and were then converted to the required units of inches (4.39 inches for a 20 year return period and 5.37 inches for a 100 year return period). The same P values were used for all of the drainage basins. The volume of runoff, Q, was then computed for each basin using CN and P in Equations 7 and 8. The Q values could have been determined from a graph provided in the guide, but using the equation was deemed to be more accurate than taking values from the graph. The optional watershed shape adjustment factor shown on the computation sheet (Figure 10) was not used.

The unit peak discharge, QU, had to be obtained graphically, since no equation was provided, using the drainage area A, the curve number CN, and the average watershed slope - flat (less than 2.5%), moderate (2.5% to 7.5%), or steep (greater than 7.5%). All of the basin slopes in the study fell within the moderate range. Figure 11 shows the graph used to obtain the peak discharge rates for small watersheds with moderate slope and a 24-hour type-II storm distribution.

A watershed slope interpolation factor, SF, was determined using the watershed slope Y and the drainage area A in a table in McCuen's guide, which is reproduced in Appendix H. Another adjustment is made to account for the effects of ponding and swamp storage. The percentage of ponds and swampy area is used in combination with the location of the ponds/swamps in the watershed (design point, center or spread out, or upper reaches) to come up with an adjustment factor which varies with storm frequency (return period). The location of the ponds/swamps for the study basins were all in the center or spread out. The table of adjustment factors is also given in Appendix I.

Figure 11: Unit Peak Rates of Discharge for Small Watersheds with Moderate Slope



The peak discharge QP is then calculated according to Equation 9:

Equation 9:

$$QP = QU \times Q \times SF \times PF \times 0.02832$$

where: QP = peak discharge (m^3/s)

QU = unit peak discharge (cfs/inch)

Q = volume of runoff (inches)

SF = slope interpolation factor

PF = ponding and swamp storage adjustment factor

0.02832 = factor to convert from cfs to m^3/s

Table 12 gives the 20 and 100 peak flows as calculated using the SCS TR-55 Chart method. The spreadsheet with all the parameter values is provided in Appendix J.

Table 12: Peak Flows Calculated Using the SCS TR-55 Chart Method

Basin	Q20 (m^3/s)	Q100 (m^3/s)
Portugal Cove	16.8	27.4
St. Philips I	23.8	38.3
St. Philips II	23.9	38.5
Logy Bay	4.8	7.6
Outer Cove	3.1	4.8
Northeast Pond River	5.1	8.9
North Pond Brook	13.0	20.5

4.5 Streamflow Data Analysis

4.5.1 Available Streamflow Data

Streamflow data from ten nearby basins was reviewed, and pertinent data are given in Table 13.

Table 13: Hydrometric Station Records for Study Area

Station	Location	Years of Record	Drainage Area (km^2)	Avg Qmax /Area ($m^3 / s / km^2$)
02ZM006	Northeast Pond River at Northeast Pond	24	3.63	0.89
02ZM007	Broad Cove Brook at St. Philips	14	17.5*	0.65
02ZM010	Waterford River at Mount Pearl	13	16.6	1.02
02ZM016	South River near Holyrood	11	17.3	0.71
02ZM017	Leary Brook at St. John's	11	15.3	1.13
02ZM018	Virginia River at Pleasantville	10	10.7	1.25
02ZM019	Virginia River at Cartwright Place	9	5.55	0.75
02ZM020	Leary Brook at Prince Philip Drive	8	17.8	1.13
02ZM021	South Brook at Pearl Town Road	8	17.8	1.13
	North Pond Brook	5	6.7	0.74

* our calculation

Most of the above basins were considered to be too urban in comparison with the study basins. Only three were selected for this study - Northeast Pond River at Northeast Pond, Broad Cove Brook in St. Philips, and North Pond Brook in Torbay. Northeast Pond River at Northeast Pond river flows into Main River, Portugal Cove, downstream of the point of interest. Data from this river will be the primary basis for estimating the flows in the Portugal Cove basin. As Broad Cove Brook in St. Philips is one of the study rivers, any measured flows are considered to be valuable data. Furthermore, the location of the gauge on Broad Cove Brook as given in the Water Survey of Canada data file (in terms of latitude and longitude coordinates) is very close to one of the main locations of interest for the flood study, at Dogberry Hill Road, so transposition of flow will not be required. The data files for Broad Cove Brook also indicate that the hydrometric station was a regulated station. A dam at the south end of Windsor Lake regulates the flow, since the overflow from the lake (a water supply) runs into the Broad Cove Brook system. This will be an important factor to consider when the peak flows from the streamflow analysis are compared with those obtained from the deterministic analysis. North Pond Brook in Torbay is the location of a provincial hydrometric station. Data from this station will be used to estimate the flows in Outer Cove and Logy Bay, as it is closest to these basins.

Streamflow records will be used in three alternative ways to calculate 20 and 100 year peak flows. The first method uses flows estimated from single station frequency analyses to estimate flows at nearby ungauged basins. The second uses the regional frequency analysis from the Department of Environment, while the third uses both single station and the regional frequency analysis.

4.5.2 Single Station Frequency Analysis

The results of a single station frequency analysis of a nearby river can be transferred to the desired drainage basin. The 20 year and 100 year peak flows for the gauged basin are estimated from measured flow data. These flows are then adjusted to the basin of interest by multiplying by the ratio of the drainage area of the basin of interest with the drainage area of the nearby gauged river.

A single station frequency analysis performed on Northeast Pond River as a part of the Water Resources Division's Regional Flood Frequency Analysis gave values of 5.2 and 6.6 m^3/s for the 20 and 100 year flows using the Generalized Extreme Value Distribution and 19 years of flow data (1970 - 1988). This analysis was updated using an Excel spreadsheet and regression analysis to reflect the additional years of flow data currently available (up to 1993). A single station frequency analysis was also carried out on Broad Cove Brook and North Pond Brook.

Of the three basins, only one, Northeast Pond River at Northeast Pond, has a sufficiently long record (24 years) for the 100 year flows to be confidently estimated. Limited flow data (1968 - 1981) is available for Broad Cove Brook in the St. Philips watershed. Since only 14 years of data are available, only the 20 year peak flows can be confidently estimated. Extrapolation of frequency curves beyond twice the period of records is not recommended (RTAC, 1982). However, since this is the only data available, the 100 year flows are estimated. The 5 year record at North Pond Brook is not sufficient to confidently estimate even the 20 year flows, but a single station frequency analysis was still carried out.

Three distributions (Gumbel, logGumbel, and lognormal) were tried using the Cunnane plotting position (approximately quantile-unbiased for a range of distributions (Stedinger et al, 1993)), given in Equation 10:

Equation 10:

$$p_i = \frac{i - 0.4}{n + 0.2}$$

where p_i = the probability of exceedance
i = the rank
n = the number of years of record

The best fitting distribution which produced the minimum error was chosen. The results of the frequency analysis are given in Table 14, and Figures 12 - 17 show the regression curves and output for the three stations. The application of these results to the study basins is summarized in Table 15.

Table 14: Single Station Frequency Analysis Results

River	Years of Data	Distribution	Q20 (m^3/s)	Q100 (m^3/s)
Northeast Pond River	24	logGumbel	5.4	6.9
Broad Cove Brook	14	lognormal	33. 0	49. 4
North Pond Brook	5	lognormal	263.5	807.5

The values obtained for Northeast Pond River are just slightly higher than those given in the Regional Flood Frequency Analysis report (5.2 and 6.6 m^3/s) for 5 less years of record. The flows estimated for Broad Cove Brook appear to be reasonable values considering the ratio of flow to drainage area (1.9, compared with 1.4 for Northeast Pond River using the 20 year flows). The values for North Pond Brook, however, are obviously grossly in error, as such high flows are impossible. Not only was a very short record used, but the range of flows within the five years was also quite large - from 2.2 to 7.9 m^3/s. It is therefore not that surprising that no reasonable 20 or 100 year flows were obtained from the North Pond Brook single station frequency analysis.

Table 14.1: Single Station Frequency Analysis Results (by WRD,DOE)

River	Years of Data	Distribution	Q20 (m^3/s)	Q100 (m^3/s)
Northeast Pond River	24	GEV	5.39	6.96
Northeast Pond River	24	3P Lognormal	5.6	7.42
Broad Cove Brook	14	GEV	18.4	23.0
Broad Cove Brook	14	3P Lognormal	18.4	23.9
North Pond Brook	5	GEV	8.58	10.2
North Pond Brook	5	3P Lognormal	11.1	16.7

The calculations in Table 14.1 are based on the generalized extreme value (GEV) and 3 parameters lognormal (3P lognormal) distributions as provided by the Water Resources Division of the Department of Environment. The confidence level in a 24 year record is much higher than those for 14 and 5 year records especially for estimating 20 year flows. Also the analysis of Northeast Pond River by various distributions presented in Table 14., 14.1 produce very similar results. This increases the confidence level in the estimation of that river. It is also noted that the 20 year flow at Northeast Pond River per unit area of 1.4 m^3/km^2 is more conservative in comparison to that of Broad Cove Brook of 1.1 m^3/Km^2 using 3P lognormal and GEV distributions. In addition, we have very limited confidence in the results produced based on 5 year record, regardless of the type of distribution. Northeast Pond River, rather than North Pond Brook, will therefore be used to estimate the peak flows in Logy Bay and Outer Cove.

Table 15: Peak Discharges in Study Basins based on Proration of SSFA Flows

Basin	Q20 (m^3/s)	Q100 (m^3/s)	Gauged Basin
Portugal Cove	18.6	23.6	Northeast Pond River
St. Philips I	26.2	33.1	Broad Cove Brook (regulated)
St. Philips II	23.7	29.7	Broad Cove Brook (regulated)
Logy Bay	3.7	4.6	Northeast Pond River
Outer Cove	9.7	16.5	Northeast Pond River

Figure 12: Regression Curve for Northeast Pond River

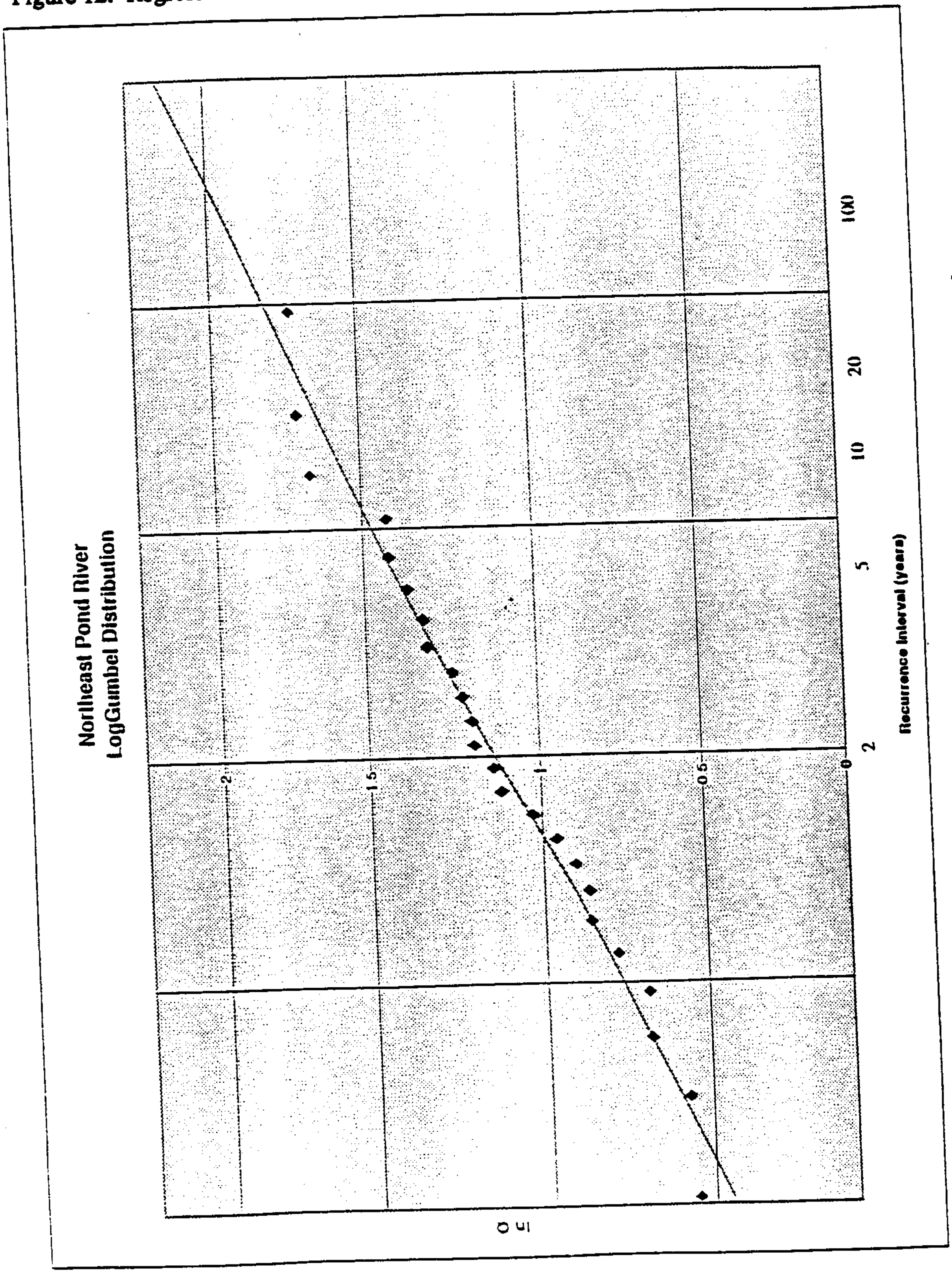


Figure 13: Regression Analysis Output for Northeast Pond River

Northeast Pond River
Regression Analysis
Lognormal Distribution

SUMMARY OUTPUT

Regression Statistics	
Multiple R	0.98944
R Square	0.97899
Adjusted R Square	0.97803
Standard Error	0.05039
Observations	24

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	1	2.60261	2.60261	1025.11509	5.98039E-20
Residual	22	0.05585	0.00254		
Total	23	2.65846			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*
Intercept	1.12369	0.01029	109.25368	1.37537E-31	1.10236	1.14502
X Variable 1	0.34576	0.01080	32.01742	5.98039E-20	0.32336	0.36815

Figure 14: Regression Curve for Broad Cove Brook

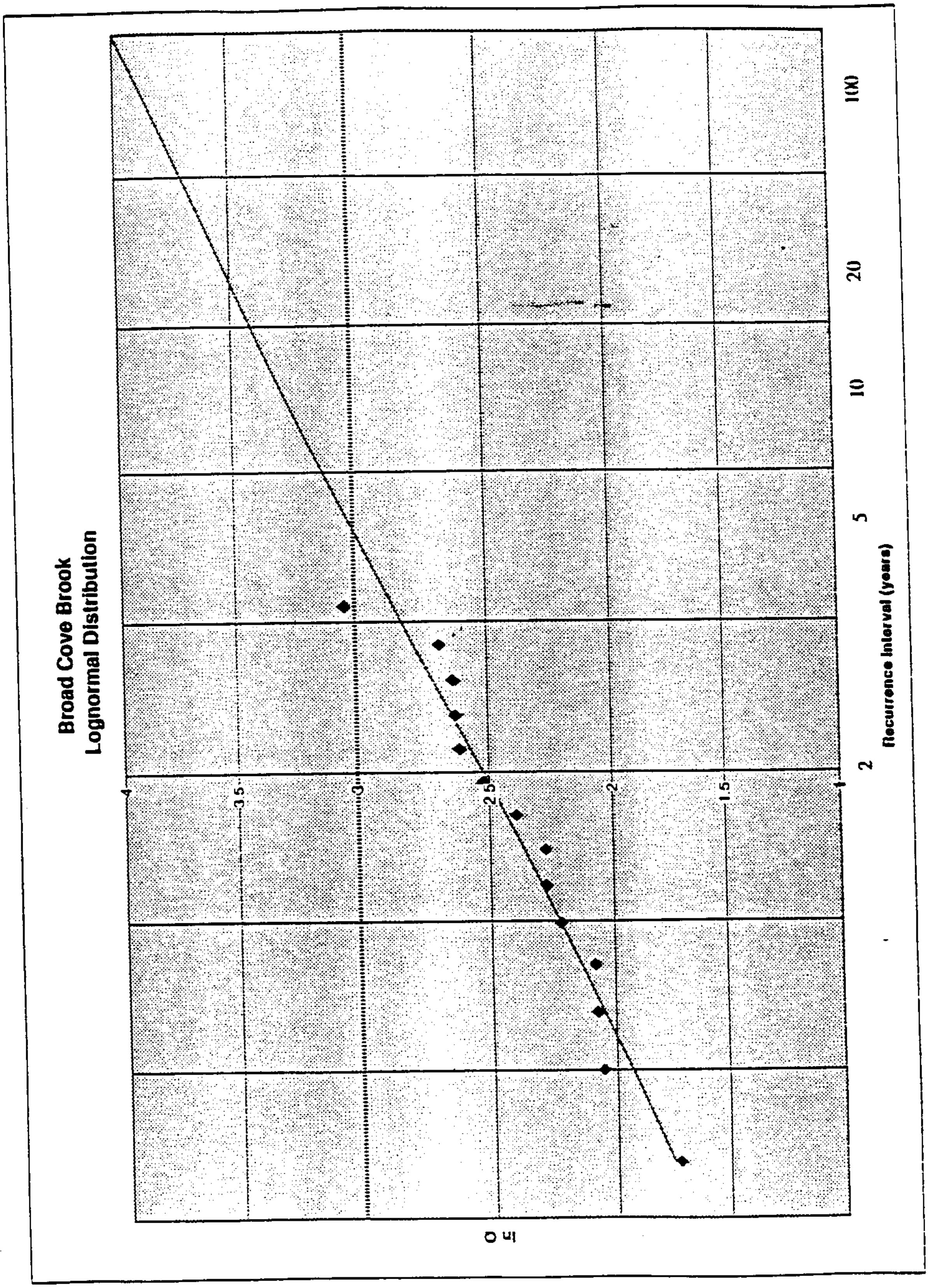


Figure 15: Regression Analysis Output for Broad Cove Brook

Broad Cove Brook
Regression Analysis
Lognormal Distribution

SUMMARY OUTPUT

Regression Statistics	
Multiple R	0.971775498
R Square	0.944347619
Adjusted R Square	0.93970992
Standard Error	0.082845593
Observations	14

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	1	1.39755275	1.39755275	203.6242	6.8681E-09
Residual	12	0.082360708	0.006863392		
Total	13	1.479913457			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*
Intercept	2.523458819	0.024482894	103.070284	4.66E-19	2.470115177	2.576802461
X Variable 1	0.59147185	0.041449509	14.26969507	6.87E-09	0.50116113	0.68178257

Figure 16: Regression Curve for North Pond Brook

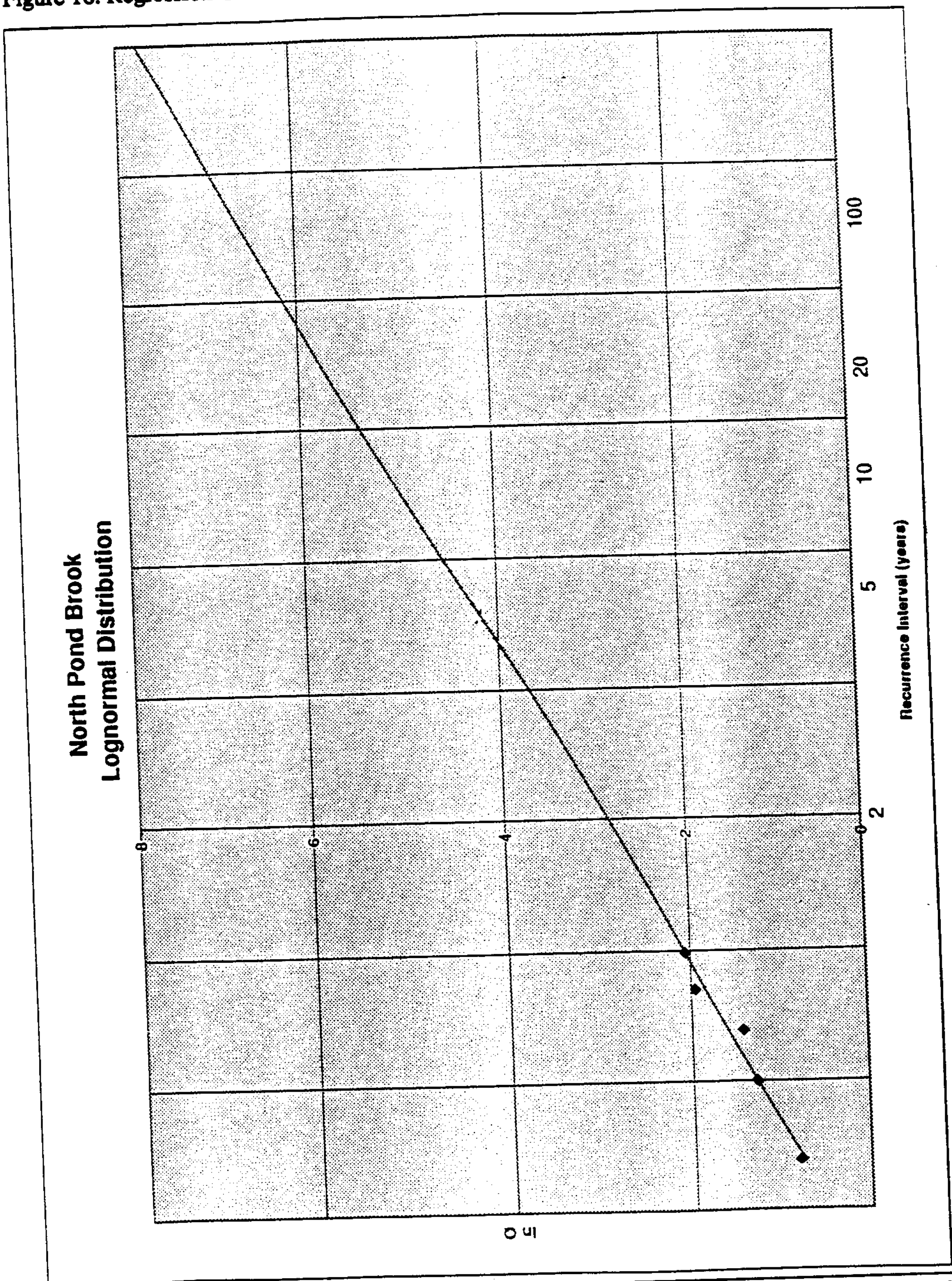


Figure 17: Regression Analysis Output for North Pond Brook

North Pond Brook
Regression Analysis
Lognormal Distribution

SUMMARY OUTPUT

Regression Statistics	
Multiple R	0.97818
R Square	0.95684
Adjusted R Square	0.94246
Standard Error	0.12576
Observations	5

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	1	1.05206	1.05206	66.51612	0.00386
Residual	3	0.04745	0.01582		
Total	4	1.09951			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*
Intercept	2.88513	0.17983	16.04363	0.00053	2.31283	3.45743
X Variable 1	1.63470	0.20044	8.15574	0.00386	0.99682	2.27258

4.5.3 Regional Flood Frequency Analysis Regression Equations

The regional regression equations provided in the User's Guide, Regional Flood Frequency Analysis for the Island of Newfoundland are also applied to determine the peak flows using the parameters given in Table 8, section 4.2.5. The developed coefficients and equations for the 20 year and 100 year peak flows are given in the User's Guide. The spreadsheet provided by the Department of Environment was used to perform the calculations, which are summarized in Table 16. These results are not considered to be reliable when the parameters are out of the recommended range. However, the equations for the Avalon and Burin region are used for this study, which do not require the slope (SLP) variable. The drainage area (DA) is the most critical parameter. The estimates for Logy Bay and Outer Cove are therefore not very reliable. This is particularly true for Outer Cove, which has unrealistically high flows according to the regional regression equation (about 1.2 m^3/s / km^2 for the 20 year flow) and also has a drainage area farthest outside the recommended range. The detailed printouts of the results are given in Appendix J.

Table 16: Regional Flood Frequency Analysis Results

Basin	Q20 (m^3/s)	Q100 (m^3/s)	Parameters Out of Range
Portugal Cove	18.6	23.6	SLP
St. Philips I	26.2	33.1	SLP
St. Philips II	23.7	29.7	None
Logy Bay	3.7	4.6	DA, DRD, SLP
Outer Cove	9.7	16.5	DA, ACL, SLP
Northeast Pond River	7.6	9.4	SLP

4.5.4 Single Station Frequency Analysis with Regional Adjustment

The 20 and 100 year peak flows estimated from a single station frequency analysis of measured flows on a nearby gauged river can also be used with an adjustment factor obtained from the regional flood frequency analysis to estimate the peak flows in ungauged basins.. The regional regression equation is applied to both the basin of interest and the gauged basin. The adjustment factor is the ratio of the flows obtained from the regression equation for the gauged and ungauged basins. This method is outlined in section 4.1.2 of the User's Guide: Regional Flood Frequency Analysis for the Island of Newfoundland, and is applied to the basins of Portugal Cove, Logy Bay, and Outer Cove using the gauged basin of Northeast Pond River. This adjustment reduces the values to 70% of those obtained from the regional regression equation alone. The adjusted values are given in Table 17.

Table 17: Single Station Frequency Analysis with Regional Adjustment

Basin	Q20 (m^3/s)	Q100 (m^3/s)	Gauged Basin Used
Portugal Cove	13.2	17.3	Northeast Pond River
Logy Bay	2.6	3.4	Northeast Pond River
Outer Cove	6.9	12.1	Northeast Pond River

4.6 The Probabilistic Rational Method

A newer version of the Rational method, the probabilistic approach (Pilgrim,1993), combines elements of both the deterministic and streamflow analysis approaches. The traditional Rational formula is still used. However, the runoff coefficient C is determined on a probabilistic basis according to the relationship between rainfall intensity-duration-frequency data and streamflow data. The procedure for deriving design data on a probabilistic basis for use of the rational method in a particular region is outlined in the following steps:

[1] For each gauged basin, carry out a frequency analysis of observed floods to determine values of q(Y) (flow in m^3/s) for a range of recurrence intervals.

[2] Select a design formula for the time of concentration which must be used consistently in all applications of the derived procedure.

[3] For each basin, values of the design rainfall intensity $i(t_c,Y)$ (mm/hr) are determined for each recurrence interval for which flood values were derived in [1], from the design rainfall intensity-duration-frequency data.

[4] Using the values from [1] and [3], values of C(Y) are calculated for each gauged basin using a rearranged form of the Rational Equation, as in Equation 11:

$$\textit{Equation 11: } C(Y) = \frac{q(Y)}{i(t_c,Y) \, x \, F \, x \, A}$$

where F is the conversion factor (0.00278) and A is the area (ha)

[5] A base value of C(Y) is selected for relating to basin characteristics. The two or ten year values C(2) or C(10) are generally convenient as they are subject to relatively low sampling errors. The selected base values of C(Y) are then related to basin characteristics by regression or are mapped over the region.

[6] Regional average values of the ratio of C(Y) to the coefficient value for the selected base average recurrence interval are then determined for each desired value of Y.

[7] Application of the rational method for flood design for any basin in the region then involves use of the adopted formula for t_c, the rainfall intensity-duration-frequency data of the region, the base runoff coefficient from [5], and the relevant frequency ratio from [6].

The design information is only valid if the t_c formula and rainfall data used in the derivation are also used in application of the method. This method has been used to derive consistent flood estimates for drainage basins in Australia (Pilgrim, 1993). The use of the probabilistic method in this study was limited to determining the C(Y) values for Northeast Pond River, since this was the only gauged basin in the region with a sufficiently long record. The C(Y) values could therefore not be related to basin characteristics by regression. Instead of steps [5], [6], and [7], the C values determined for Northeast Pond River for recurrence intervals of 20 and 100 years were directly applied in the Rational formula for the Portugal Cove, Logy Bay, and Outer Cove basin for comparison with those predicted by other methods. This method was not applied to the St. Philips basins, as the Broad Cove Brook records were not of sufficient length. The SCS Curve Number Method of determining the time of concentration was used both in deriving the C values and in the application to the ungauged basins. The frequency curve for the probabilistic rational method is shown in Figure 18. The runoff coefficients developed from Northeast Pond River data are given in Table 18. They appear to be relatively independent of the average recurrence interval. A rounded value of 0.25 was therefore used to calculate both the 20 and 100 year peak flows for the Portugal Cove, Logy Bay, and Outer Cove basins, which are given in Table 19. The peak discharges do not include the 20% increase in precipitation to allow for snowmelt, since the C values are already based on the direct relationship between streamflow and precipitation, which would include effects such as snowmelt.

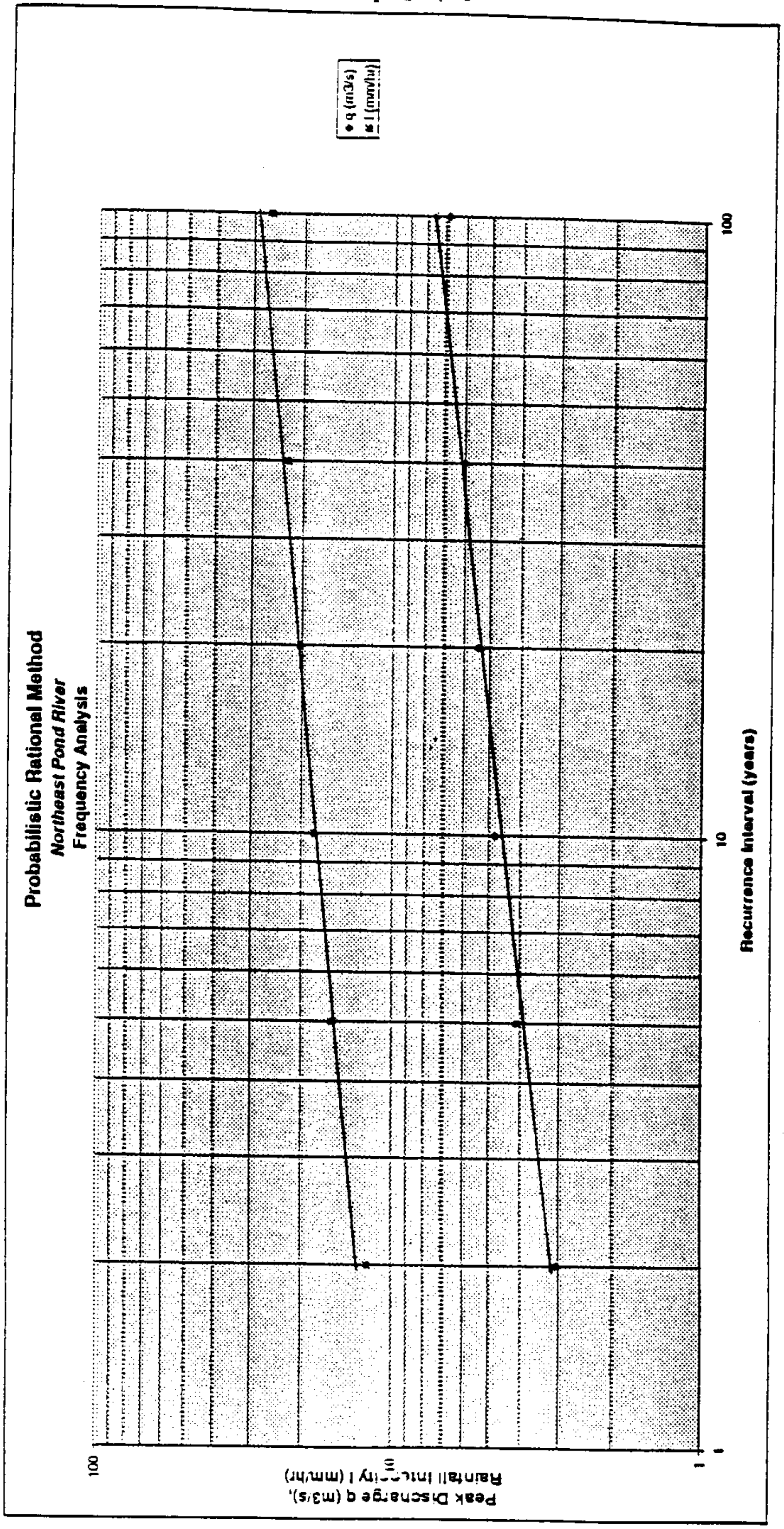
Table 18: Runoff Coefficients Developed from Northeast Pond River Data

Recurrence Interval (yrs)	Peak Discharge q (m^3/s)	Rainfall Intensity I (mm/hr)	Runoff Coefficient (C)
2	3.07	11.88	0.239
5	4.11	15.64	0.243
10	4.79	18.19	0.243
20	5.43	20.32	0.247
40	6.05	22.80	0.245
100	6.87	26.12	0.243

Table 19: Peak Discharges Obtained using Probabilistic Rational Method

Basin	Q20 (m^3/s)	Q100 (m^3/s)
Portugal Cove	13.7	17.7
Logy Bay	2.6	3.2
Outer Cove	1.3	1.7

Figure 18: Probabilistic Rational Method Frequency Curve



4.7 *Analysis of Results*

4.7.1 Verification of Methodology

Since the single station frequency analysis on the five year record of North Pond Brook produced unusable results, the verification of the deterministic approach is based solely on the peak discharges estimated for the gauged basin of Northeast Pond River. Table 20 gives the peak flows estimated by the deterministic approaches for comparison with those obtained in the frequency analysis. The Probabilistic Rational flows have not been included in the table, as they are not independent of the streamflow analysis.

As shown in Table 20, the Rational Method overestimates the flow in the basin by a substantial amount - the average using the different times of concentration is approximately double the value produced through the single station frequency analysis (SSFA). Of the three methods of determining time of concentration, the SCS Curve Number Method results in the lowest flows which is closest to the frequency analysis flows. This is as expected, since the SCS Curve Number Method is recommended over the others in the Drainage Manual (RTAC, 1982).

The ratio between the 20 and 100 year flows is 0.8, which is consistent with that for the SSFA flows. This overestimation of flow can be attributed to a high runoff coefficient. A value of 0.37 was computed based on the basin attributes of soil type and land use (which was lowest of all the basins studied). However, using the Probabilistic Rational Method and working backwards from the 20 and 100 streamflows estimated from measured data, and from the 20 and 100 year precipitation intensities estimated from measured data, a value of approximately 0.25 was obtained. Because the flows obtained using the Rational Method were too high for Northeast Pond River, it is expected that they will also be too high for the other basins.

The SCS TR-55 Chart Method produces flows which are very close to the SSFA flows. The ratio between the 20 and 100 year flows is lower using this method - approximately 0.6 compared to 0.8 for the SSFA. This may be due in part to the fact that, in addition to proportionately higher precipitation depths, the adjustment factors for ponding and swampy areas are higher for the 100 year recurrence interval than for the 20 year recurrence interval. Overall, the SCS TR-55 Chart Method provides good consistency with the SSFA flows, and is therefore the deterministic method of choice.

Table 20: Summary of Peak Discharges for Northeast Pond River

Method	Q20 (m^3/s)	Q100 (m^3/s)
Rational		
t_c SCS Curve Number	9.8	12.6
t_c Bransby-Williams	12.5	15.9
t_c Airport Drainage	11.3	14.3
SCS TR-55 Chart Method	5.1	8.9
SSFA	5.4	6.9
RFFA Equation	7.6	9.4

4.7.2 Portugal Cove

The 20 and 100 year peak flows obtained using all of the methods applied to the Portugal Cove basin are summarized in Table 21. The Rational Method flows are high, as expected from the discussion of the Northeast Pond River basin. The values obtained from all five of the remaining methods are reasonably close, with averages of 15.5 m^3/s and 21.1 m^3/s for the 20 and 100 year flows, respectively. We consider these five methods are equally valid. Therefore the average values of 15.5 m^3/s and 21.1 m^3/s have been selected as the 20 and 100 year flows to be used in the hydraulic analysis.

Table 21: Summary of Peak Discharges for Portugal Cove

Method	Q20 (m^3/s)	Q100 (m^3/s)
Rational		
t_c SCS Curve Number	28.3	36.4
t_c Bransby-Williams	30.9	39.8
t_c Airport Drainage	31.0	40.0
SCS TR-55 Chart Method	16.8	27.4
SSFA (Area Proration)	15.2	19.4
RFFA Equation	18.6	23.6
SSFA (RFFA Adjustment)	13.2	17.3
Probabilistic Rational	13.7	17.7

4.7.3 St. Philips

Tables 22 and 23 summarize the 20 and 100 year peak flows obtained using all of the methods applied to the St. Philips basins. Since the St. Philips I basin is slightly larger than St. Philips II, slightly larger flows would be expected. However, this was not always exactly the case due to the slightly lower curve numbers(75 compared with 76) and runoff coefficients (0.43 compared with 0.44) computed for the larger basin.

Because the flows used in the single station frequency analysis of Broad Cove Brook were regulated (as the flow still is), there is a considerable amount of uncertainty in results obtained from other methods. In this case, the Rational Method flows, although still high, are not as high relative to the other methods as those found in Northeast Pond River and Portugal Cove. Both the SCS TR-55 Chart Method flows and the RFFA flows appear to underestimate the peak discharges, as both are about 75% of those obtained using the single station frequency analysis from Broad Cove Brook (the St. Philips II basin). Taking into consideration the uncertainty involved in estimating the parameters, and hence the flows, in a regulated basin; it is recommended that the results obtained using the single station frequency analysis of Broad Cove Brook be used. The conservative values estimated using the lognormal distribution are recommended. These are applied directly in St. Philips II to give 20 and 100 year flows of 33.0 m^3/s and 49.4 m^3/s, respectively, and are prorated by area to give 20 and 100 year flows of 34.7 m^3/s and 52.0 m^3/s, respectively, in the St. Philips I basin.

Table 22: Summary of Peak Discharges for St. Philips I

Method	Q20 (m^3/s)	Q100 (m^3/s)
Rational		
t_c SCS Curve Number	38.3	48.8
t_c Bransby-Williams	43.2	55.3
t_c Airport Drainage	46.4	59.6
SCS TR-55 Chart Method	23.8	38.3
SSFA (Area Proration)	34.7	52.0
RFFA Equation	26.2	33.1
SSFA (RFFA Adjustment)	36.5	55.1

Table 23: Summary of Peak Discharges for St. Philips II

Method	Q20 (m^3/s)	Q100 (m^3/s)
Rational		
t_c SCS Curve Number	39.5	50.5
t_c Bransby-Williams	43.7	56.1
t_c Airport Drainage	46.1	59.2
SCS TR-55 Chart Method	23.9	38.5
SSFA	33.0	49.4
RFFA Equation	23.7	29.7

4.7.4 Logy Bay

The 1993 hydrologic analysis performed on the Logy Bay basin by the Water Resources Division (Taylor, 1993) was reviewed for comparison with the current study. The flows used in that report were 2.8 m^3/s and 3.6 m^3/s for the 25 year and 100 year, respectively, using the average between the flows obtained from the Rational Method (3.4 m^3/s and 4.1 m^3/s) and the SCS Graphical Method (2.3 m^3/s and 3.1m^3/s). Although these values are within the range, there were significant differences in some of the parameters, most notably the channel length of 1.2 km (this study used a hydraulic length of 2.0 km to the basin divide), and the basin slope of 0.32% (a value of 4.53% was determined in this study). A slightly smaller drainage area (133 ha compared with 146.25 ha), and much smaller runoff coefficients (0.30 compared with 0.42) and curve numbers (70 compared with 76) were used in the 1993 report. Furthermore, the rainfall intensity was not increased to account for snowmelt.

The 20 and 100 year peak flows obtained using all of the methods applied to the Logy Bay basin are summarized in Table 24. In this case, the deterministic method of choice - the SCS TR-55 Chart Method - produced values only a little lower than those obtained from the Rational Method, which is assumed to overestimate the flow by a factor of nearly two. It is difficult to ascertain the reason for this surprising result. When the per unit area flow for the Logy Bay basin is computed, it is apparent that the SCS TR-55 Chart Method flows are too high. Using the 20 year SCS TR-55 Chart Method, values of 1.5 $m^3/s/km^2$ for the Portugal Cove basin and 1.3 $m^3/s/km^2$ for the Northeast Pond River basin were calculated, compared with a value of 3.3 $m^3/s/km^2$ for the Logy Bay basin. The two SSFA methods (area proration and RFFA adjustment) and the Probabilistic Rational method provide the most reasonable flow estimates for Logy Bay, with flows per unit area from 1.4 to 1.8 $m^3/s/km^2$. For the hydraulic analysis of the Logy Bay basin, 20 and 100 year flows of 2.6 m^3/s and 3.2 m^3/s are recommended.

Table 24: Summary of Peak Discharges for Logy Bay

Method	Q20 (m^3/s)	Q100 (m^3/s)
Rational		
t_c SCS Curve Number	5.2	6.5
t_c Bransby-Williams	6.3	8.0
t_c Airport Drainage	5.5	7.0
t_c SCS Upland	5.3	6.8
SCS TR-55 Chart Method	4.8	7.6
SSFA (Area Proration)	2.0	2.6
RFFA Equation	3.7	4.6
SSFA (RFFA Adjustment)	2.6	3.4
Probabilistic Rational	2.6	3.2

4.7.5 Outer Cove

The 20 and 100 year peak flows obtained using all of the methods applied to the Outer Cove basin are summarized in Table 25. These results are similar to those obtained for Logy Bay. When the per unit area flow for the Outer Cove basin is computed, it is apparent that the SCS TR-55 Chart Method flows are too high. Using the 20 year SCS TR-55 Chart Method, a value of 3.8 $m^3/s/km^2$ was computed for the Outer Cove basin, compared with values of 1.5 and 1.3 $m^3/s/km^2$ for the Portugal Cove and Northeast Pond River basins. The RFFA flows cannot be considered at all, as the Outer Cove drainage area is much smaller than those used in the RFFA. For the same reason, the SSFA result with the RFFA adjustment is also unreasonable. Only the SSFA (area proration) method and the Probabilistic Rational method provide reasonable flow estimates for Outer Cove, with flows per unit area from 1.3 and 1.6 $m^3/s/km^2$, respectively. For the hydraulic analysis of the Outer Cove, 20 and 100 year flows of 1.3 m^3/s and 1.7 m^3/s are recommended.

Table 25: Summary of Peak Discharges for Outer Cove

Method	Q20 (m^3/s)	Q100 (m^3/s)
Rational		
t_c SCS Curve Number	2.5	3.2
t_c Bransby-Williams	3.2	4.0
t_c Airport Drainage	2.9	3.6
t_c SCS Upland	3.3	4.2
SCS TR-55 Chart Method	3.1	4.8
SSFA (Area Proration)	1.1	1.5
RFFA Equation	9.7	16.5
SSFA (RFFA Adjustment)	6.9	12.1
Probabilistic Rational	1.3	1.7

4.8 Summary of Hydrologic Investigation

The recommended 20 and 100 year peak discharges for the hydraulic analysis, based on the analysis presented in the previous section, are summarized in Table 26.

Table 26: Recommended Peak Discharges for Hydraulic Analysis

Basin	Q20	Q100
Portugal Cove	15.5	21.1
St. Philips I	34.7	52.0
St. Philips II	33.0	49.4
Logy Bay	2.6	3.2
Outer Cove	1.3	1.7

5.0 HYDRAULIC ANALYSIS

The purpose of the hydraulic analysis is to compute the flood profile for the 1:20 year and 1:100 year events for the study areas. The hydraulic analysis was carried out using two programs. The first program, HY-8, is used for analysing the culverts at St Philips, Outer Cove and Logy Bay areas. The second Program, HEC-2, is used for determining the water levels at various cross sections and bridges at St. Philips and Portugal Cove areas. The results are detailed in Appendices L and M.

A field program was carried out to measure the flow velocity and water depth at various cross sections along the streams. The measured flow velocities were then used to estimate Manning's coefficient (n) which is then used in the analysis in HEC-2. Also some information about the maximum water levels were collected from interviews with the residents, some of whom lived in the area for 40 years. The obtained information showed that the calculated water levels at St. Philips are conservative (higher than the local observations). At Collision Clinic, the obtained observations indicate that the estimated flows are reasonable.

5.1 Field Report

The flow meter provided by the Department of Environment was used to determine the average velocities by counting the number of beeps (revolutions) of the flow meter for a given time interval, and then converting to linear velocity using the following equation, Equation 1, provided with and calibrated for the particular flow meter used.

Equation 1:

$$velocity\,(m/s) \;=\; (0.6754\, x\, rev/\mathrm{sec}) \;+\; 0.0043$$

For flow depths from 0.1 m and 0.75 m, the velocity taken at 0.6 of the depth below the surface in the vertical is the recommended mean velocity in the vertical (Tamburi & Lye, 1985). All of the flow depths encountered during the site visits were within this range, and the flow velocities were therefore measured at 0.6 of the flow depth below the surface.

On October 22, 1995, flow velocity and water depth measurements were taken in Broad Cove Brook, St. Philips, at Dogberry Hill Road Bridge. The measurements were taken at four different locations below the upstream face of the bridge, as given in Table 27.

Table 27: Flow Velocities at Dogberry Hill Road Bridge, October 22, 1996

Distance from left of bridge (m)	Water Depth (cm)	Revolutions/second (rev/s)	Corrected Velocity (m/s)
2.44	27	22/60	0.25
4.04	34	45/60	0.51
5.11	30	37/51	0.49
6.02	28	17/55	0.21

The time required for a scrap of paper to travel under Dogberry Hill Road Bridge, a distance of 25 ft, was also recorded. Three trials gave times of 17, 15, and 13 seconds for an average of 15 seconds, which corresponds to a water surface velocity of 0.51 m/s.

On October 26, a second visit was made to the St. Philips site. Velocity and flow depth measurements were taken at five locations along the upstream face of Dogberry Hill Road Bridge. These are given in Table 28.

Table 28: Flow Velocities at Dogberry Hill Road Bridge, October 26, 1996

Distance from left of bridge (m)	Water Depth (cm)	Revolutions per 60 s	Corrected Velocity (m/s)
2.44	25	11	0.13
2.90	24	26	0.30
4.04	34	41	0.47
5.11	33	42	0.48
6.02	30	21	0.24

At cross section SP-11, upstream of the former Hamelmann residence, flow depths of 25 cm, 22 cm, 20 cm, 32 cm, 40 cm, 33 cm, 30 cm, and 25 cm were recorded from left to right looking downstream. A velocity of 89 revolutions per 60 seconds (1.01 m/s corrected) was recorded at a flow depth of 25 cm.

At the private bridge at the former Hamelmann residence (Station 0+939), the water level was 103 cm below the bottom of the slab at the smooth part of the channel on the left (looking downstream), and 1420 mm below the bottom of the slab at the lower turbulent part on the right.

Residents at SP-13 stated that the maximum water level in 40 years was up to grassy level below the steeper bank section - well below the level of their houses. Water depth measurements of 40 cm, 45 cm, and 39 cm were recorded from left to right looking downstream. At a depth of 43 cm, 51 revolutions were counted in 60 seconds, for a corrected velocity of 0.58 m/s.

At SP-14, three velocities were recorded: 49 rev/60 sec (0.56 m/s) at 28 cm flow depth, 36 rev/60s (0.41 m/s) at 25 cm depth, and 56 rev/60 s (0.63 m/s) at 33 cm depth.

At Rebecca's Road bridge, flow depths of 63 cm, 55 cm, 65 cm, 60 cm, and 50 cm, were measured across the section from left to right looking downstream. The effective flow channel was approximately 3.1 m wide, and a velocity reading was taken at 1.9 m from the left of the effective flow channel. 33 revolutions were recorded in 60 seconds, for a velocity of 0.41 m/s.

Water depth measurements taken at SP-04 were 22 cm, 22 cm, 18 cm, 22 cm, 15 cm, and 23 cm, from left to right looking downstream.

The Portugal Cove site was visited on November 2, beginning at approximately 2:00 p.m. Two velocity measurements were taken at cross section PC-22. At a water depth of 64 cm, the flow meter gave a reading of 34 revolutions in 45 seconds, which corresponds to a corrected linear velocity of 0.51 m/s. At 25 cm water depth, 23 revolutions were recorded in 36 seconds, for a corrected linear velocity of 0.44 m/s.

A resident living adjacent to Churchill's Road bridge stated that "no flooding takes place down this far". He said that the water level can get up to a couple of inches below the bridge.

Further upstream, Mr. Burry described the flooding which occurred on his property and the neighbouring property (a hardware store) of Sterling Kean, due to the undersized culvert at the Storey residence upstream. The culvert was replaced with a bridge. This did not entirely eliminate the problem, as trees and rocks in the stream still cause some flooding. Measurements were taken as Main River passes behind Mr. Burry's residence, between PC-21 and PC-22. At this location, the flow velocity at a depth of 24 cm was recorded as 38 revolutions in 35 seconds, for a corrected velocity of 0.74 m/s. This section was approximately 2.5 m wide.

It was noted that PC-B4 is a bridge built over a collapsed slab. The flow channel is reduced in size due to wood. As the slab supports the bridge, it will not be easy to remove.

A resident living upstream of PC-B3 (arch bridge) stated that he does get water in his basement, but it is caused primarily by seepage due to the water and sewer system.

5.2 Culvert Analysis

The HY-8 computer program (version 4.1) was used for the analysis of all culverts in this study. The input file contains the site data, culvert data, channel data, and flow values. The site data describes the inlet and outlet elevations, culvert length and slope, and number of barrels. The culvert data contains the culvert shape, dimensions (span and rise), material, Manning's coefficient, inlet type, inlet edge and depression. The channel geometry, slope, Manning's coefficient for the channel, invert and outlet elevations, the roadway surface condition, embankment top and width, crest length, and overtopping crest elevation are described in the channel data input. The flow value for which the calculations are to be carried out is also specified in the input data file. According to the input parameters, HY-8 calculates the culvert capacity, headwater and tailwater elevations in the channel and the water elevation when overtopping occurs.

The results of the investigation are provided in the following sections. The capacity of each culvert to pass the design flows is indicated by sufficient (properly sized) or insufficient (under-sized) size. The minimum size required to pass the design flows is also presented for each area of the study.

5.2.1 Logy Bay Area

The present size of the culvert LB-C3 (Fig. 19) is adequate for passing the 20 (2.6 m^3/s) and 100 (3.2 m^3/s) year flows. In fact, the calculations showed that overtopping will occur at much higher flow (5.7 m^3/s). Accordingly we do not anticipate flooding problems resulting from this culvert. The historical flooding that took place at this area is shown in Fig. 20. This was caused by an undersized culvert which has since been replaced by the present one.

5.2.2 Outer Cove Area

Five culverts were analyzed in this area (Fig. 21) as follows:
For culverts OC-C1 and OC-C2, the flow was estimated from their drainage area at 60% of the total area flow. The 20 and 100 year flows for these culverts are .78 and 1. m^3/s, respectively. The results of the analysis are:

OC-C1: The culvert size (.6 m diameter) is not sufficient to pass the 20 year flow. Overtopping will occur at .6 m^3/s discharge.

OC-C2: This culvert is greatly undersized (.3 m diameter) and will cause flooding.

The remaining three culverts OC-C3, OC-C4 and OC-C5 were analyzed using the total flows of the area, 1.3 and 1.7 m^3/s for the 20 and 100 year estimations. The results of the analysis are:

OC-C3: The culvert (.75 m diameter) is undersized. It is not sufficient to pass the 20 year flow. Overtopping will occur at approximately 1 m^3/s flow.

OC-C4: This culvert is adequately sized.

OC-C5: This culvert is by the Collision Clinic and it is undersized. It will only pass 2 year flow without overtopping.

In order to pass the 100 year flow for this area a minimum of 2 culverts of 0.6 m diameter or an equivalent single culvert of 0.85 m diameter is needed. The historical flood levels at the Collision Clinic is shown in Fig. 22.

5.2.3 Portugal Cove Area

A total of 25 culverts were studied in this area from PC-C1 to PC-C25 (Fig. 23). The area of drainage for these culverts is estimated at 4.2% of the total area. Therefore the used 20 and 100 year flows are 0.67 and 0.92 m^3/s. The results of the analysis are as follows:

PC-C1, PC-C2,C3, PC-C4,C5, PC-C6, PC-C7,C8, PC-C11, PC-C12, PC-C14,C15, PC-C16, PC-C17,C18,C19, PC-C20, PC-C21, and PC-C23 are adequately sized. The undersized culverts are PC-C9,C10, PC-C13, PC-C22 and PC-C24,C25. The minimum required single culvert diameter is 0.6 m or two pipe culverts of 0.45 m diameter.

5.3 Water Surface Profile Analysis

The HEC-2 program was used to determine the water level at the St. Philips and Portugal Cove areas. The flows obtained from the Hydrological Analysis section were used in this Hydraulic Analysis section in order to calculate the water levels. The values of Manning's coefficient are obtained from the analysis of the field data.

5.3.1 Starting Water Level

The starting water level downstream was calculated using:

$$Q = (A^{1.67} \times S^{0.5})/(P^{.66} \times n)$$

Where Q represents the discharge in m^3/s, A is the area of the cross section, P is the wetted perimeter, S is the average water slope, and n is the Manning's coefficient. The Manning's coefficient was estimated from the measured data using the field measured discharges. For St. Philips and Portugal Cove, the n coefficient was estimated at .07 and .065, respectively. Although these values appear to be on the high side of n, it was decided to use them to calculate the water levels bearing in mind that the estimated levels will be conservative , i.e. the highest level expected.

The n value was also reduced to 0.05 in order to examine the effect of n variation on the calculations. Marginal difference in the water levels were observed using the lower n value, which indicated that the calculations are not highly sensitive to the variation of n.

5.3.2 Flood Level Delineation and Ice Jam Effect

Many of the water levels obtained from the analysis covered large area of the banks. The cross sections were therefore extended using the 1:2500 scale maps. For the 1:100 year and 1:20 year flows, the calculated water levels in St Philips using main channel Manning's coefficient of .07 are shown below in Table 29. The historical, 1:20 year, and 1:100 year flood levels are plotted in Figs. 24 and 25, respectively.

Table 29 Summary of 1:20 and 1:100 year flood levels
St. Philips Area

Cross Section	Q100 Level (m)	Q20 Level (m)	Historical Level (m)
SP-13, STA 1+127	80.40	80.10	
SP-12, STA 0+939	82.90	82.76	
SP-B3, STA 0+935	84.01	83.34	82.20
SP-B3, STA 0+931.7	84.18	83.71	82.20
SP-11, STA 0+830	85.53	85.31	
SP-10, STA 0+472	100.79	100.56	
SP-09, STA 0+427	101.80	101.59	
SP-08, STA 0+385	103.34	103.09	
SP-07, STA 0+364	104.22	103.86	
SP-06, STA 0+334	104.64	104.45	
SP-05, STA 0+284	105.72	105.48	
SP-B2, STA 0+279	105.79	105.41	
SP-B2, STA 0+274.2	106.10	105.76	
SP-04, STA 0+225	106.55	106.23	
SP-B1, STA 0+177.6	107.73	107.41	
SP-B1, STA 0+170	107.82	107.51	
SP-03, STA 0+167	107.87	107.56	106.20
SP-02, STA 0+085	108.00	107.68	
SP-01, STA 0+000	108.52	108.34	

The results for the 1:100 and 1:20 year peak flows for Portugal Cove area using Manning's coefficient of 0.065 are presented below in Table 30. The historical, 1:20 year, and 1:100 year flood levels are plotted in Figs. 26 and 27, respectively. The flood areas for the culverts PC-C1 to PC-C25 using 1:100 and 1:20 year peak flows are plotted in Figs. 28 and 29.

The cross-sections modelling both streams and the 1:100 year water levels are provided in Appendix M.

It seems that the estimated discharges in the hydrology study were conservative and produced higher water levels than those observed by the residents. Under such high water levels and large width of the flooded areas, the ice blockage of the stream will not affect the calculated levels. It is not reasonable to assume that ice jams will occur over such large areas. Even if jams are present they will be localized on smaller portions of the flooded area and will not restrict the flow. Hence, no analysis were carried out for studying the ice jam effects using reduced channel areas.

Table 30 Summary of 1:20 and 1:100 year flood levels
Portugal Cove Area

Cross Section	Q100 Level (m)	Q20 Level (m)	Historical Level (m)
PC-23, STA 1+681	66.83	66.70	
PC-B7, STA 1+649	68.20	67.95	
PC-B7, STA 1+646	68.44	68.15	
PC-22, STA 1+620	68.90	68.78	
PC-21, STA 1+429	74.61	74.52	78.1
PC-B6, STA 1+416	75.00	74.78	
PC-B6, STA 1+412.5	75.41	75.14	
PC-20, STA 1+361	77.83	77.69	
PC-B5, STA 1+346.8	78.07	78.04	
PC-B5, STA 1+340	78.50	78.27	
PC-19, STA 1+337	78.96	78.75	
PC-B4, STA 1+302	80.10	79.88	
PC-B4, STA 1+299.6	80.23	79.97	
PC-18, STA 1+273	82.16	81.97	
PC-B3, STA 1+254	83.31	83.07	
PC-B3, STA 1+254	83.96	83.59	
PC-17, STA 1+238	84.13	83.80	
PC-16, STA 1+147	86.69	86.60	
PC-14, STA 1+031	94.81	94.68	
PC-C1	154.99	154.88	
PC-C2,C3	148.91	149.51	
PC-C4,C5	149.70	149.40	
PC-C6	148.90	148.67	
PC-C7,C8	148.62	148.32	
PC-C9,C10	147.40	147.35	
PC-C11	146.95	146.75	
PC-C12	143.60	143.40	
PC-C13	134.10	134.05	
PC-C14,C15	128.01	127.90	
PC-C16	115.08	114.88	
PC-C17,C18,C19	114.62	114.42	
PC-C20	113.58	113.28	
PC-C21	113.20	113.00	
PC-C22	112.60	112.55	
PC-C23	109.17	108.97	
PC-C24,C25	108.55	108.50	

5.3.3 Backwater Sensitivity Analysis

A series of simulations was conducted with the hydraulic model to examine the sensitivity of the flood levels to changes in Manning's coefficient, the starting water level, and the flood flow.

Tables 31 and 32 show the effect of changes in n coefficient on the water levels. The channel roughness was reduced from 0.07 to 0.05 in St. Philips (Table 31) and from 0.06 to 0.05 in Portugal Cove area (Table 32). The changes in the computed water levels were small which indicate that the model is insensitive to modest changes in channel roughness.

Tables 33 and 34 show the effect of changing the starting water level for St. Philips and Portugal Cove, respectively. The initial starting water level, computed above, and low and high starting water levels were inputted to the HEC-2 and the water levels at all cross sections were computed for Q100. Again marginal level differences are observed between the three starting levels at all sections. This indicates that the model is insensitive to modest changes in starting water level.

Table 31 Flood Level Sensitivity Analysis for Channel Roughness
St. Philips Area

Cross Section	Level (m) for Q100		Level (m) for Q20	
	n=0.07	n=0.05	n=0.07	n=0.05
SP-13, STA 1+127	80.40	80.40	80.10	80.10
SP-12, STA 0+939	82.90	82.88	82.76	82.74
SP-B3, STA 0+935	84.01	84.06	83.34	83.34
SP-B3, STA 0+931.7	84.18	84.15	83.71	83.60
SP-11, STA 0+830	85.53	85.46	85.31	85.20
SP-10, STA 0+472	100.79	100.86	100.56	100.60
SP-09, STA 0+427	101.80	101.94	101.59	101.67
SP-08, STA 0+385	103.34	103.36	103.09	103.08
SP-07, STA 0+364	104.22	104.26	103.86	103.80
SP-06, STA 0+334	104.64	104.73	104.45	104.52
SP-05, STA 0+284	105.72	105.67	105.48	105.38
SP-B2, STA 0+279	105.79	105.80	105.41	105.39
SP-B2, STA 0+274.2	106.10	106.08	105.76	105.68
SP-04, STA 0+225	106.55	106.38	106.23	106.14
SP-B1, STA 0+177.6	107.73	107.56	107.41	107.21
SP-B1, STA 0+170	107.82	107.69	107.51	107.33
SP-03, STA 0+167	107.87	107.76	107.56	107.41
SP-02, STA 0+085	108.00	107.94	107.68	107.59
SP-01, STA 0+000	108.52	108.45	108.34	108.27

Table 32 Flood Level Sensitivity Analysis for Channel Roughness
Portugal Cove Area

Cross Section	Level (m) for Q100		Level (m) for Q20	
	n=0.065	n=0.05	n=0.065	n=0.05
PC-23, STA 1+681	66.83	66.83	66.70	66.70
PC-B7, STA 1+649	68.20	68.02	67.95	67.79
PC-B7, STA 1+646	68.44	68.32	68.15	68.06
PC-22, STA 1+620	68.90	68.90	68.78	68.81
PC-21, STA 1+429	74.61	74.61	74.52	74.54
PC-B6, STA 1+416	75.00	75.00	74.78	74.78
PC-B6, STA 1+412.5	75.41	75.32	75.14	75.06
PC-20, STA 1+361	77.83	77.84	77.69	77.68
PC-B5, STA 1+346.8	78.07	77.89	78.04	77.87
PC-B5, STA 1+340	78.50	78.37	78.27	78.10
PC-19, STA 1+337	78.96	78.93	78.75	78.63
PC-B4, STA 1+302	80.10	79.79	79.88	79.67
PC-B4, STA 1+299.6	80.23	79.99	79.97	79.78
PC-18, STA 1+273	82.16	82.1	81.97	81.95
PC-B3, STA 1+254	83.31	83.07	83.07	82.85
PC-B3, STA 1+254	83.96	83.66	83.59	83.32
PC-17, STA 1+238	84.13	83.89	83.80	83.45
PC-16, STA 1+147	86.69	86.72	86.60	86.61
PC-14, STA 1+031	94.81	94.81	94.68	94.68

Table 33 Flood Level Sensitivity Analysis For Starting Water Level
St. Philips Area (Q100, n=0.07)

Cross Section	Computed Water Levels (m)		
	Starting Water Level		
	80 m	80.4 m	80.8 m
SP-13, STA 1+127	80.16	80.4	80.80
SP-12, STA 0+939	82.90	82.90	82.83
SP-B3, STA 0+935	84.01	84.01	84.01
SP-B3, STA 0+931.7	84.18	84.18	84.18
SP-11, STA 0+830	85.53	85.53	85.53
SP-10, STA 0+472	100.79	100.79	100.79
SP-09, STA 0+427	101.80	101.80	101.80
SP-08, STA 0+385	103.34	103.34	103.34
SP-07, STA 0+364	104.22	104.22	104.22
SP-06, STA 0+334	104.64	104.64	104.64
SP-05, STA 0+284	105.72	105.72	105.72
SP-B2, STA 0+279	105.79	105.79	105.79
SP-B2, STA 0+274.2	106.10	106.10	106.10
SP-04, STA 0+225	106.55	106.55	106.55
SP-B1, STA 0+177.6	107.73	107.73	107.73
SP-B1, STA 0+170	107.82	107.82	107.82
SP-03, STA 0+167	107.87	107.87	107.87
SP-02, STA 0+085	108.00	108.00	108.00
SP-01, STA 0+000	108.52	108.52	108.52

Table 34 Flood Level Sensitivity Analysis For Starting Water Level Portugal Cove Area (Q100, n=0.065)

Cross Section	Computed Water Levels (m)		
	Starting Water Level		
	66.6 m	66.83 m	67 m
PC-23, STA 1+681	66.82	66.83	67.00
PC-B7, STA 1+649	68.20	68.20	68.17
PC-B7, STA 1+646	68.44	68.44	68.44
PC-22, STA 1+620	68.90	68.90	68.90
PC-21, STA 1+429	74.61	74.61	74.61
PC-B6, STA 1+416	75.00	75.00	75.00
PC-B6, STA 1+412.5	75.41	75.41	75.41
PC-20, STA 1+361	77.83	77.83	77.83
PC-B5, STA 1+346.8	78.07	78.07	78.07
PC-B5, STA 1+340	78.50	78.50	78.50
PC-19, STA 1+337	78.96	78.96	78.96
PC-B4, STA 1+302	80.10	80.10	80.10
PC-B4, STA 1+299.6	80.23	80.23	80.23
PC-18, STA 1+273	82.16	82.16	82.16
PC-B3, STA 1+254	83.31	83.31	83.31
PC-B3, STA 1+254	83.96	83.96	83.96
PC-17, STA 1+238	84.13	84.13	84.13
PC-16, STA 1+147	86.69	86.69	86.69
PC-14, STA 1+031	94.81	94.81	94.81

Tables 35 and 36 show the effect of changing the flow value in the St. Philips and Portugal Cove areas, respectively. The water levels were computed for the range of 85%, 100%, and 115% of the 1:100 year flow. Small variation in the flood levels were observed in both areas which indicates that the HEC-2 is insensitive to modest changes in the flow.

Table 35 Flood Level Sensitivity Analysis For Change In Flow
St. Philips Area (Q100, n=0.07)

Cross Section	Computed Water Levels (m)		
	Q100 - 15%	Q100	Q100 + 15%
SP-13, STA 1+127	80.40	80.40	80.40
SP-12, STA 0+939	82.84	82.90	82.96
SP-B3, STA 0+935	83.52	84.01	84.10
SP-B3, STA 0+931.7	84.13	84.18	84.22
SP-11, STA 0+830	85.43	85.53	85.61
SP-10, STA 0+472	100.69	100.79	100.87
SP-09, STA 0+427	101.72	101.80	101.91
SP-08, STA 0+385	103.23	103.34	103.43
SP-07, STA 0+364	104.00	104.22	104.29
SP-06, STA 0+334	104.57	104.64	104.74
SP-05, STA 0+284	105.62	105.72	105.81
SP-B2, STA 0+279	105.63	105.79	105.88
SP-B2, STA 0+274.2	105.95	106.10	106.23
SP-04, STA 0+225	106.41	106.55	106.66
SP-B1, STA 0+177.6	107.61	107.73	107.84
SP-B1, STA 0+170	107.70	107.82	107.93
SP-03, STA 0+167	107.75	107.87	107.98
SP-02, STA 0+085	107.87	108.00	108.11
SP-01, STA 0+000	108.44	108.52	108.59

Table 36 Flood Level Sensitivity Analysis For Change In Flow
Portugal Cove Area (Q100, n=0.065)

Cross Section	Computed Water Levels (m)		
	Q100 - 15%	Q100	Q100 + 15%
PC-23, STA 1+681	66.83	66.83	66.90
PC-B7, STA 1+649	68.05	68.20	68.32
PC-B7, STA 1+646	68.27	68.44	68.63
PC-22, STA 1+620	68.86	68.90	68.93
PC-21, STA 1+429	74.59	74.61	74.64
PC-B6, STA 1+416	74.88	75.00	75.12
PC-B6, STA 1+412.5	75.27	75.41	75.56
PC-20, STA 1+361	77.76	77.83	77.88
PC-B5, STA 1+346.8	78.06	78.07	78.02
PC-B5, STA 1+340	78.36	78.50	78.67
PC-19, STA 1+337	78.86	78.96	79.02
PC-B4, STA 1+302	79.93	80.10	80.25
PC-B4, STA 1+299.6	80.05	80.23	80.39
PC-18, STA 1+273	82.09	82.16	82.33
PC-B3, STA 1+254	83.16	83.31	83.55
PC-B3, STA 1+254	83.75	83.96	84.20
PC-17, STA 1+238	83.97	84.13	84.37
PC-16, STA 1+147	86.64	86.69	86.75
PC-14, STA 1+031	94.74	94.81	94.87

6.0 Remedial Measures and Recommendations

6.1 General

In this section, remedial measures are suggested in order to minimize or eliminate the flood damage to structures located in the flood-prone areas and to restrict and discourage additional development in these areas. It is important to understand that no floodproofing method will totally protect properties from the effect of severe flooding. The best protection is to avoid development in the floodplain or the floodplain fringe. The floodway is that part of the flood risk area in which most of the flood waters are conveyed. Damages due to flooding to properties in this area are usually substantial. In the fringes of a floodplain, the flow depth is shallow and the water velocity is low.

A flood damage reduction plan consists of:

a) Non-Structural Measures which minimize potential loss to development in the floodplain;

b) Structural Measures which directly affect the flood characteristics.

6.1.1 Non-Structural Measures

The most common non-structural measure is the implementation of Land Use Regulation or Development Control. Land use regulations are used to regulate the development of land within the floodplain. Based on the floodplain mapping, the regulations can be incorporated into municipal plans and zoning bylaws to indicate flood risk areas and what type of structures can be developed. For example, no new buildings should be erected in the floodplain where damage potential is high.

Watershed management practises encompass the management of activities that may increase the magnitude of flooding. Agriculture, forestry and urban expansion are some of these activities that can be regulated to limit increase of flood water.

Other potential measures are residential redevelopment, tax policies to discourage building on the floodplain, warning signs showing past flood area, and acquisition of undeveloped lands and properties.

6.1.2 Structural Measures

Structural measures such as dykes, channel improvements and diversions are often used to control flood water. These measures are usually costly. Modification to culverts are cost effective measures that can be (and have been) implemented in several areas of this study.

Dykes are embankments built to protect low-lying areas from inundation. They prevent flood water from entering to the low-lying areas during high flows. Appropriate design of dykes is required to avoid dyke failure under water pressure or seepage.

Channel improvements may include realignment to eliminate sharp bends, dredging, removal of debris, installation of weirs or drop structures, and provision of bank protection. These measures lead to stabilization of the river course, regulation of gradient and flow velocity, enlargement of channel capacity, and reduction of bank and channel erosion.

Flow diversion involves the redirection of part or all of the flow around a particular location. This may require the construction of dams, installation of inlet and outlet control structures, excavation of a diversion channel, etc. Despite the high cost involved in such measure, it offers a reliable and positive degree of flood control.

Flood proofing encompasses a wide variety of adjustments, additions, and alterations to structures in attempt to minimize or eliminate potential flood damage. These measures include:

- installation of permanent or temporary closures at low level opening in structures;
- raising structures on fill, columns or piers; and
- construction of floodwalls or low berm around structures.

Permanent closure involves permanently closing and sealing all possible openings in a structure through which flood waters could enter. Generally, this technique is used with large structures on the outer fringe of flood-prone areas where flood depths are less than about 0.3 m. Elevating buildings, or building additions above flood levels are suited for areas where permanent closure is difficult.

Contingency flood proofing measures are best suited to areas where the depth or risk of flooding is small. Basic techniques include the installation of watertight barriers around doors and windows.

Emergency flood proofing measures are effective in areas expected to have a shallow water depth and a slow rate of water rise during a flood. However, these measures are labour intensive and are usually undertaken on short notice. Basic techniques include sand-filled bags stacked in such a way to form a barrier against rising flood waters.

Under the Canada-Newfoundland FDR Program, flood proofing is only recommended for the 1:100 flood zone.

6.2 Recommended Remedial Measures

6.2.1 Logy Bay Area

The size of the culvert LB-C3 is adequate for passing the 1:20 and 1:100 year flows. Historical flooding took place before the old culvert was replaced by the present one. Therefore, we do not anticipate flooding problems resulting from the present culvert and no remedial measures are required.

6.2.2 Outer Cove Area

In Outer Cove area there are several culverts that will not pass the 1:20 year flow. Thus flooding will take place in this area where the culverts are undersized. It is recommended that in order to prevent flooding, the undersized culverts should be replaced or additional culvert installed to provide suitable drainage capacity. Potential remedial measures and cost estimates are as follows:

OC-C1: It is recommended to install an additional culvert with minimum diameter of 0.6 m in order to pass the 1:100 year flow. The estimated cost of installation is $1500.

OC-C2: It is recommended to install an additional culvert with minimum diameter of 0.75 m in order to pass the 1:100 year flow. The estimated cost of installation is $1500.

OC-C3: It is recommended to install an additional culvert with minimum diameter of 0.6 m in order to pass the 1:100 year flow. The estimated cost of installation is $1500.

OC-C5: This culvert by the Collision Clinic is undersized. It is recommended to install an additional culvert with minimum diameter of 0.6 m in order to pass the 1:100 year flow. The estimated cost of installation is $7000.

6.2.3 Portugal Cove Area

A total of 25 culverts were studied in this area from PC-C1 to PC-C25. Several culverts were found to be adequately sized. These are PC-C1, PC-C2,C3,C4,C5, PC-C6, PC-C7,C8, PC-C11, PC-C12, PC-C14,C15, PC-C16, PC-C17,C18,C19, PC-C20 and PC-C21. No remedial measures are needed at these locations. The undersized culverts are PC-C9,C10, PC-C13, PC-C22 and PC-C24,C25. It is recommended to replace the culverts at each site with a single culvert of 0.6 m diameter, or install and additional 0.60 m diameter culvert at each of theses three sites. The estimated cost of installation is $1500 for each location.

Some of the historical flooding took place because of an undersized culvert at the Storey property. Since this culvert has been replaced with a bridge, as mentioned in the historical overview section, no remedial measures are needed for that location. However, in several places the flood water level will overflow the banks and bridges. Several locations where there will be flooding are already developed and it is difficult and costly to protect these developments from flooding as the water levels are much higher than the basements of the existing structures. Other areas are not developed and are not recommended for development as they fall under the flood level. Some of the previously mentioned remedial measures can be implemented to minimize the flood damage in these areas.

It is also recommended to keep the channel free of debris and fallen trees as some of the historical flooding occurred because of channel blockage.

6.2.4 St. Philips Area

St. Philips area also has flooding problems according to the conservative analysis performed in this study. Many developed areas and bridges will be flooded. Once more, as the basements of the structures in the area are below the calculated water levels, it is difficult and costly to protect them from flooding. The land that lies below the flood lines are not recommended for development.

It is recommended that some of the previously mentioned remedial measures be implemented to minimize the flood damage in this area, such as restricting developments in flood-prone areas, and flood proofing the existing buildings. Dykes may also be used were the flood level are slightly higher than the channel banks, especially near Hamelmann residence.

7.0 Conclusions

The flood risk area associated with 1:20 year and 1:100 year recurrence interval flood levels were assessed in the St. Philips, Portugal Cove, Outer Cove and Logy Bay areas. Historical data for the flow and water levels in the study areas are very limited. Therefore, the estimated peak flows in this study are chosen on the conservative side.

In Logy Bay area the culvert size is sufficiently large to pass the anticipated flows and no flooding problem is expected in this area.

In Outer Cove area there are several culverts that will not pass the 1:20 year flow. Thus a flooding will take place in this area where the culverts are undersized. Minimum culvert sizes are recommended in the culvert analysis section where the size was found to be inadequate. Cost estimates for the recommended remedial measures are provided.

In Portugal Cove area, several culverts were also analyzed. Similar to Outer Cove area there are places where the culverts are undersized. Therefore some flooding will take place in this area. Minimum culvert sizes are recommended in the culvert analysis section where the size was found to be inadequate.

In Portugal Cove area, the channel was also examined. In several places the flood water level will overflow the banks and bridges. Several locations where there will be flooding are already developed and it is difficult and costly to protect these developments from flooding as the water levels are much higher than the basements of the existing structures. Other areas are not developed and are not recommended for development as they fall under the flood level.

St. Philips area also has flooding problems according to the conservative analysis performed in this study. Many developed areas and bridges may be flooded. Once more, as the basements of the structures in the area are below the calculated water levels, it is difficult and costly to protect them from flooding. Other areas that lie below the flood lines are not recommended for development.

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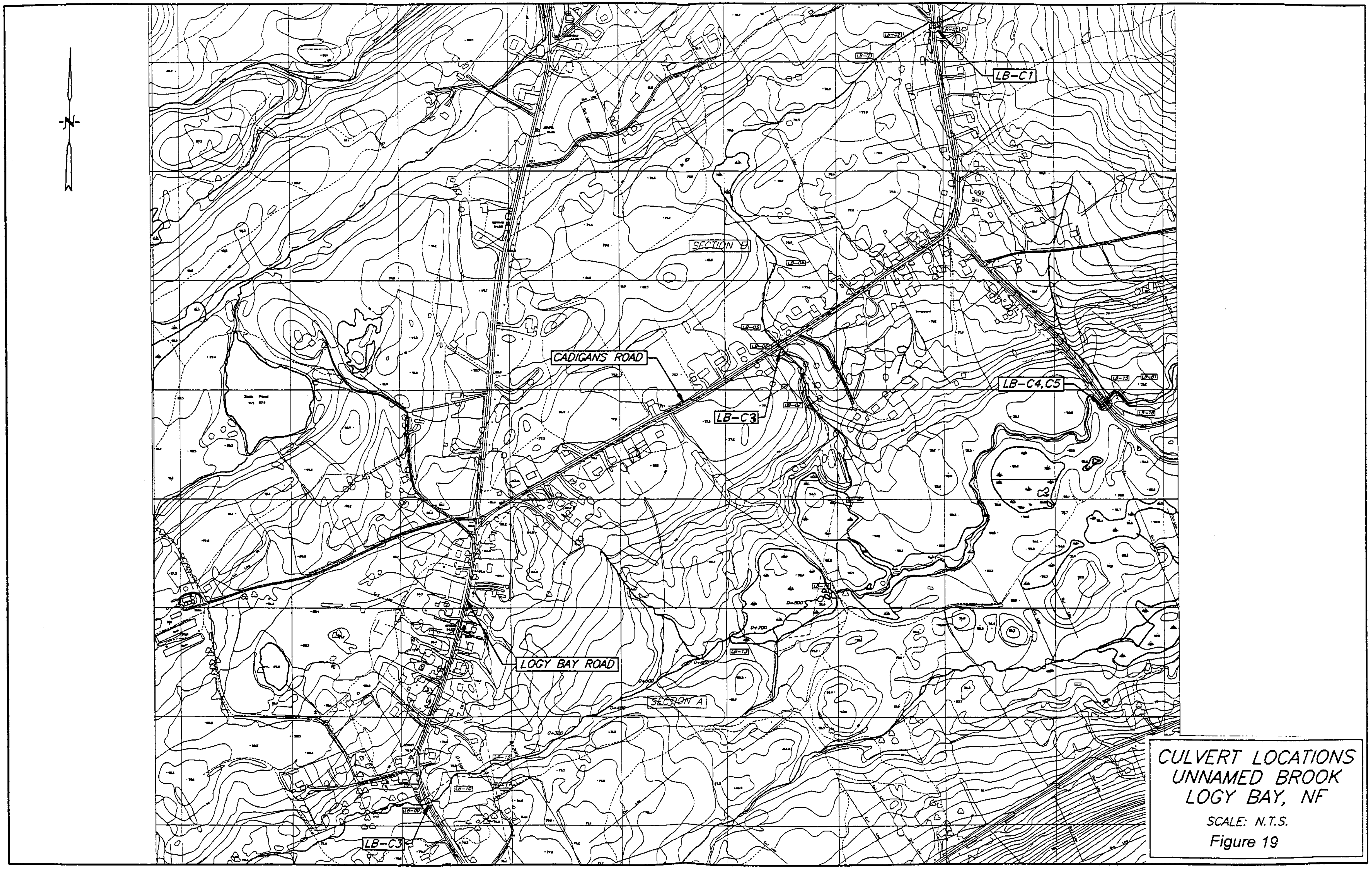
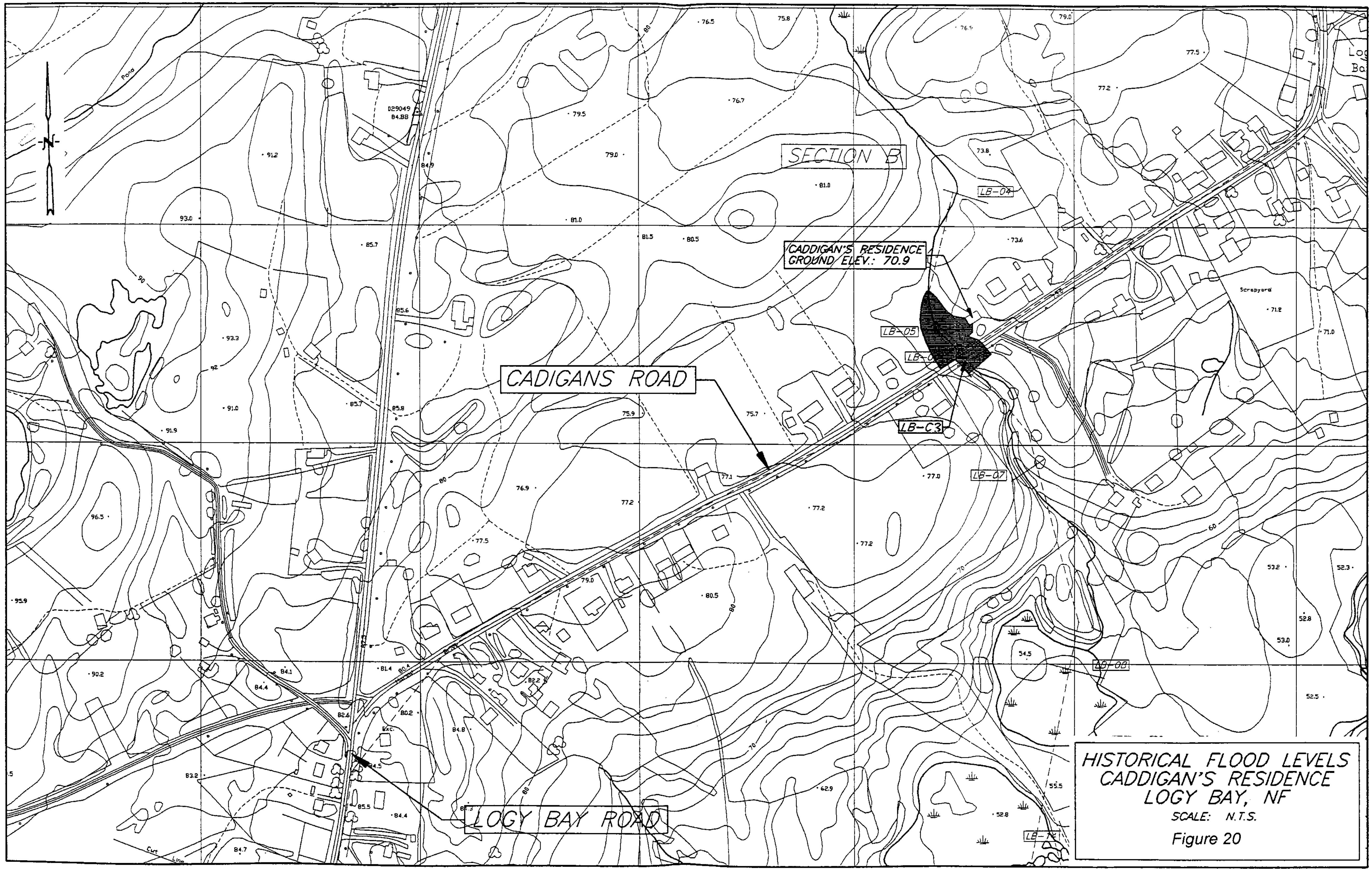


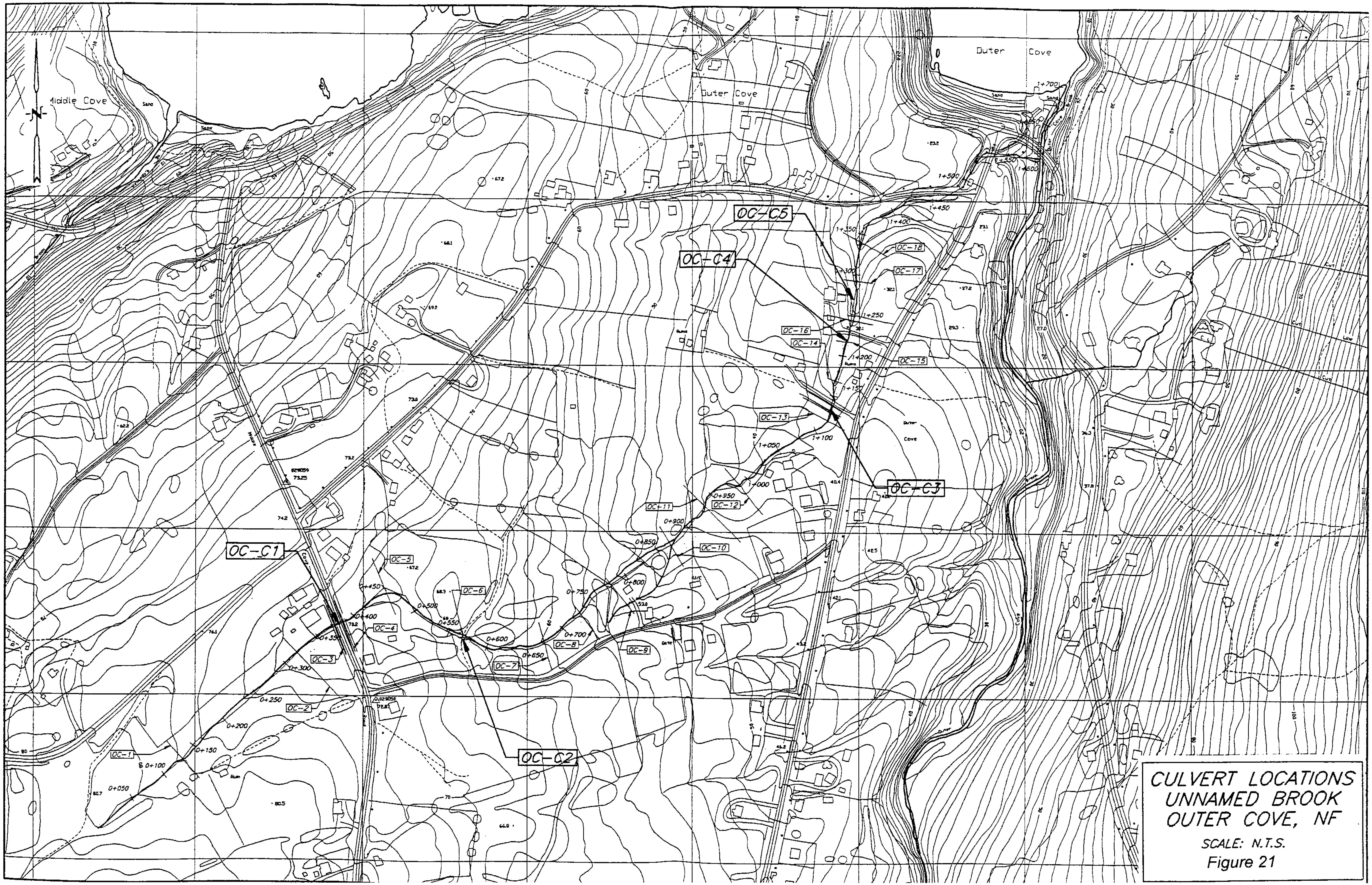
CULVERT LOCATIONS
UNNAMED BROOK
LOGY BAY, NF

SCALE: N.T.S.

Figure 19



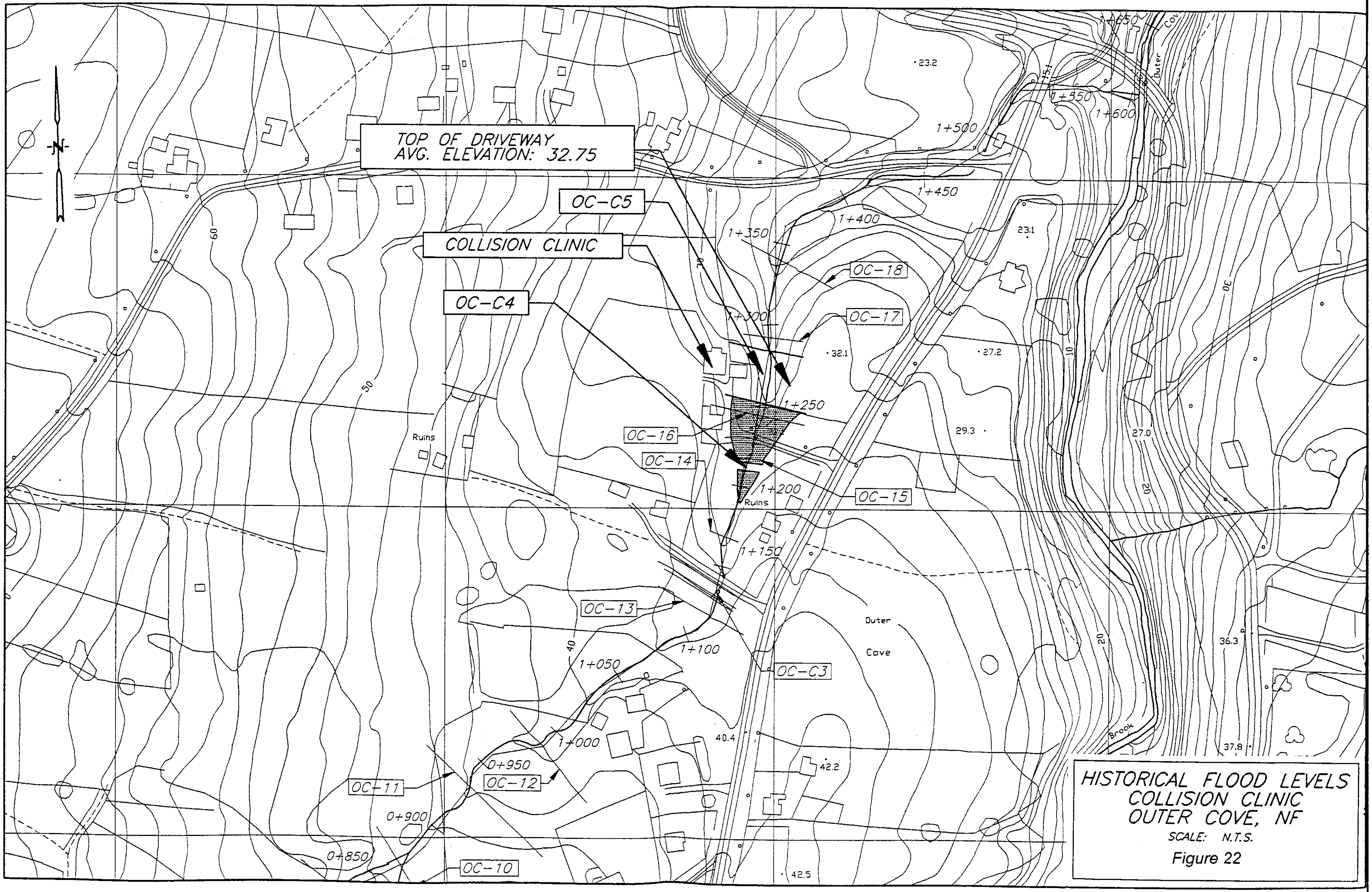
HISTORICAL FLOOD LEVELS
CADDIGAN'S RESIDENCE
LOGY BAY, NF
SCALE: N.T.S.
Figure 20



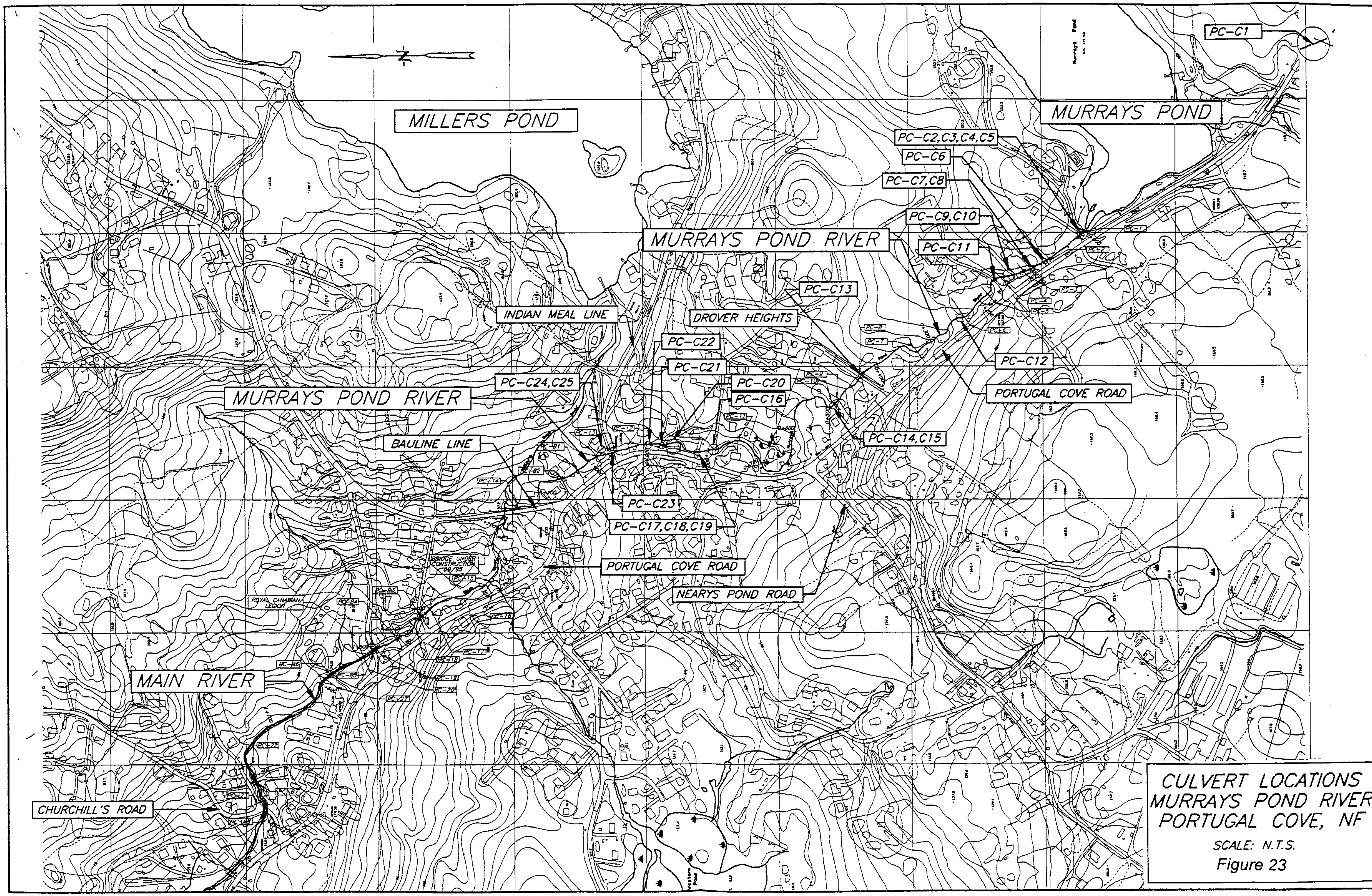
CULVERT LOCATIONS
UNNAMED BROOK
OUTER COVE, NF

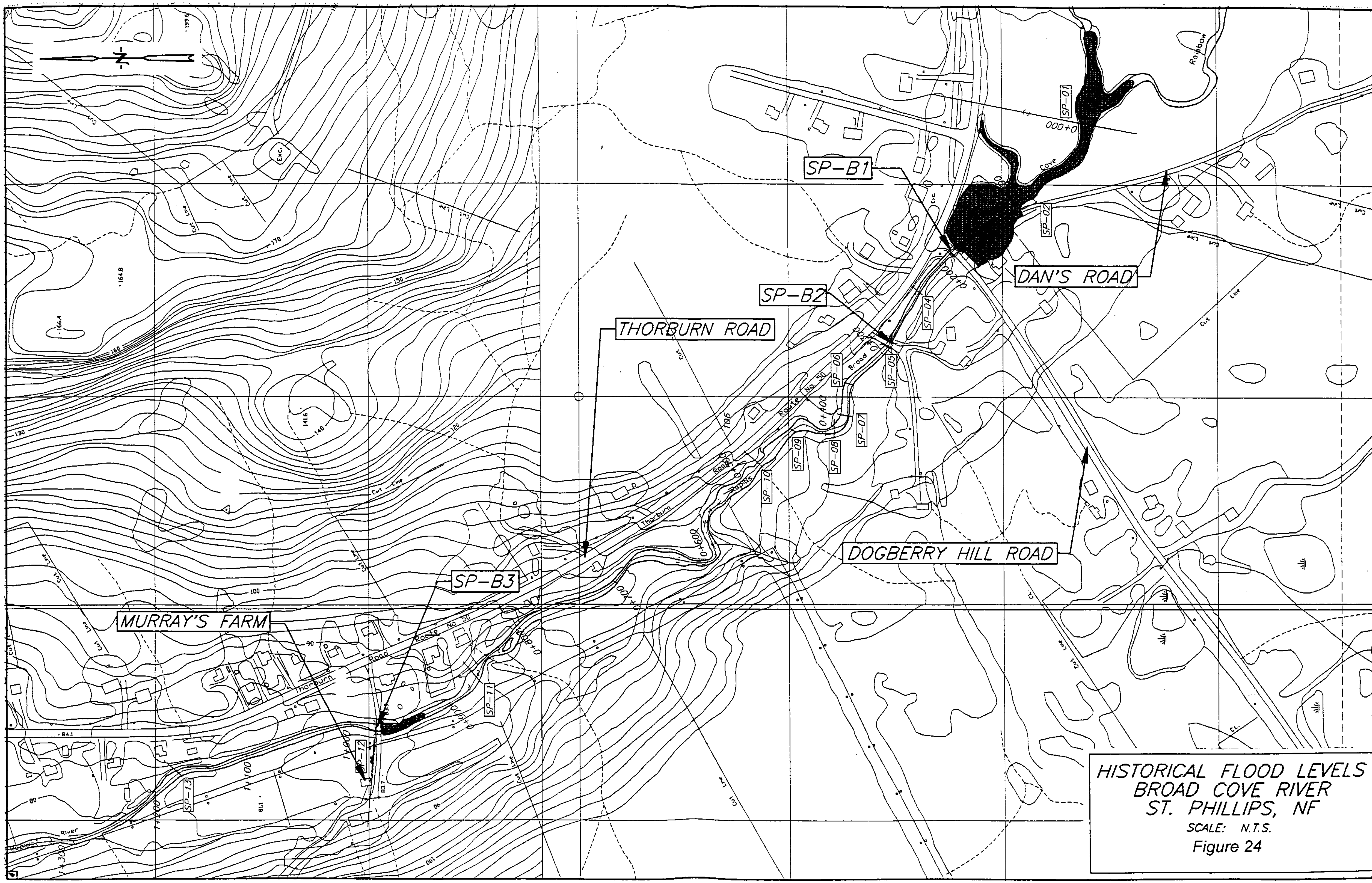
SCALE: N.T.S.

Figure 21



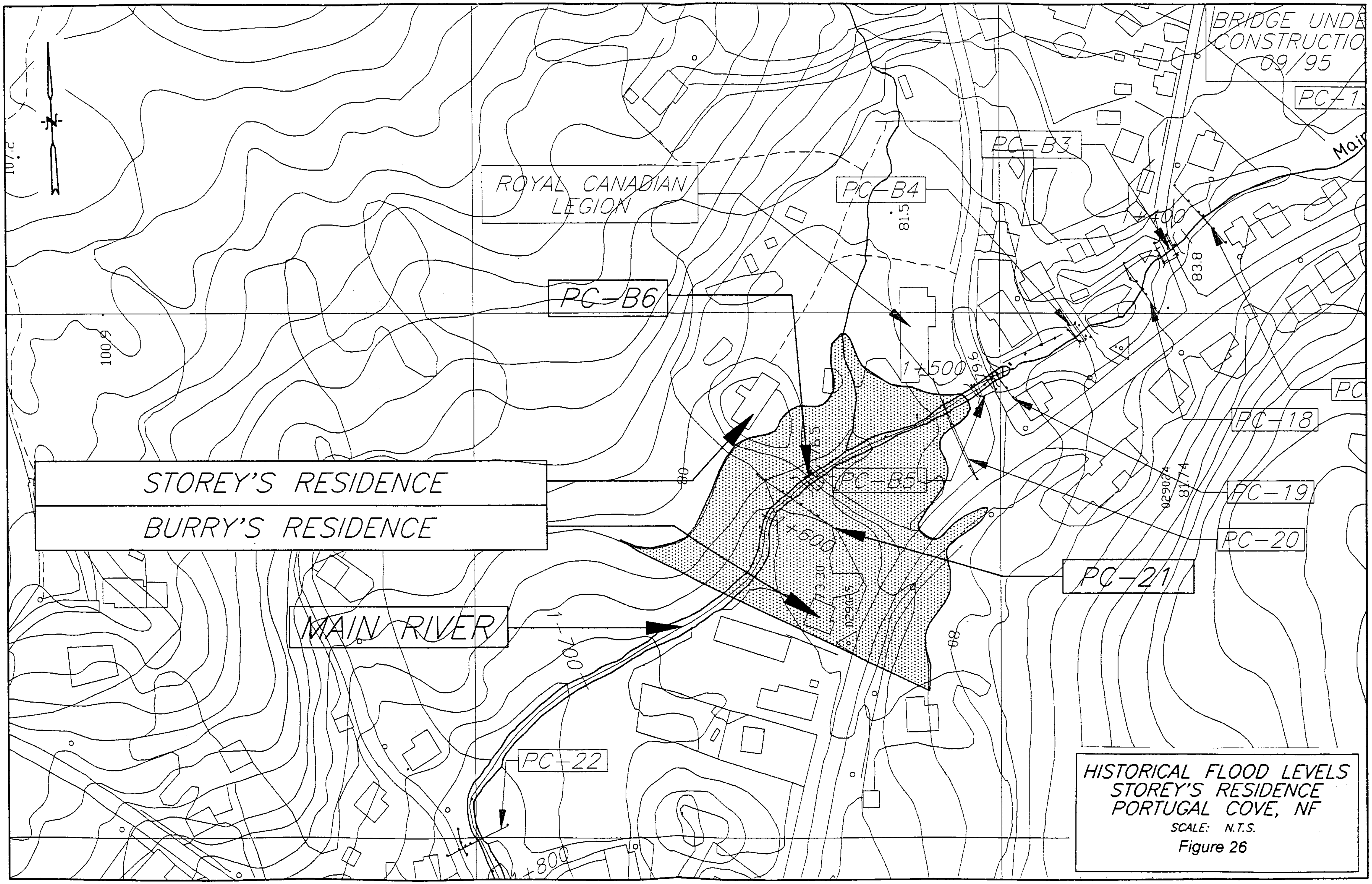
HISTORICAL FLOOD LEVELS
COLLISION CLINIC
OUTER COVE, NF
SCALE: N.T.S.
Figure 22


MILLERS POND
MURRAYS POND
PC-C1
PC-C2,C3,C4,C5
PC-C6
PC-C7,C8
PC-C9,C10
PC-C11
MURRAYS POND RIVER
PC-C13
INDIAN MEAL LINE
DROVER HEIGHTS
PC-C22
PC-C21
PC-C12
PC-C20
PC-C16
PC-C24,C25
MURRAYS POND RIVER
PORTUGAL COVE ROAD
BAULINE LINE
PC-C14,C15
PC-C23
PC-C17,C18,C19
PORTUGAL COVE ROAD
NEARYS POND ROAD
BRIDGE UNDER CONSTRUCTION 04/95
ROYAL CANADIAN LEGION
MAIN RIVER
CHURCHILL'S ROAD
CULVERT LOCATIONS
MURRAYS POND RIVER
PORTUGAL COVE, NF
SCALE: N.T.S.
Figure 23

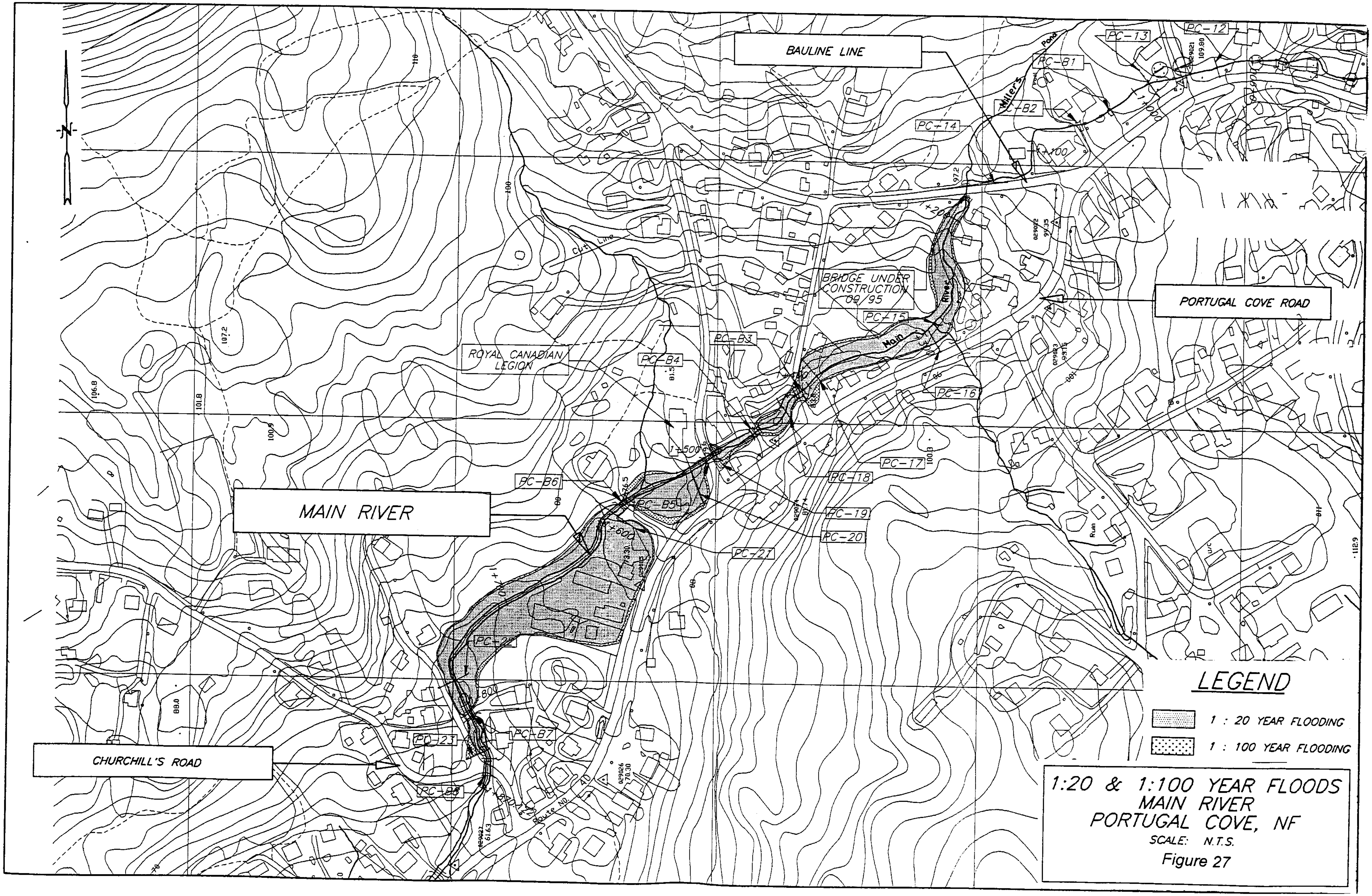


HISTORICAL FLOOD LEVELS
BROAD COVE RIVER
ST. PHILLIPS, NF
SCALE: N.T.S.
Figure 24

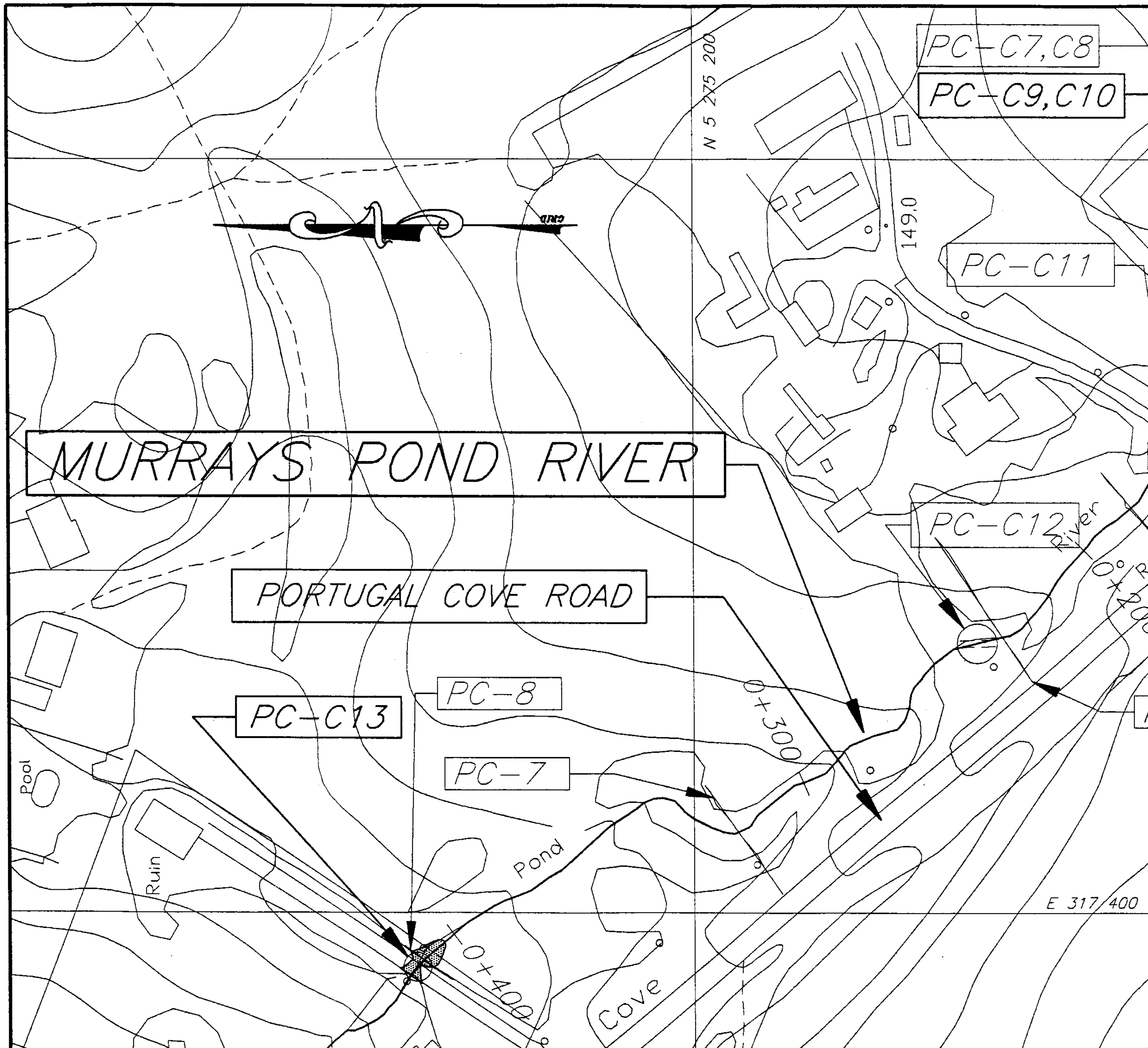
1:20 & 1:100 YEAR FLOODS
BROAD COVE RIVER
ST. PHILLIPS, NF
SCALE: N.T.S.
Figure 25



HISTORICAL FLOOD LEVELS
STOREY'S RESIDENCE
PORTUGAL COVE, NF
SCALE: N.T.S.
Figure 26

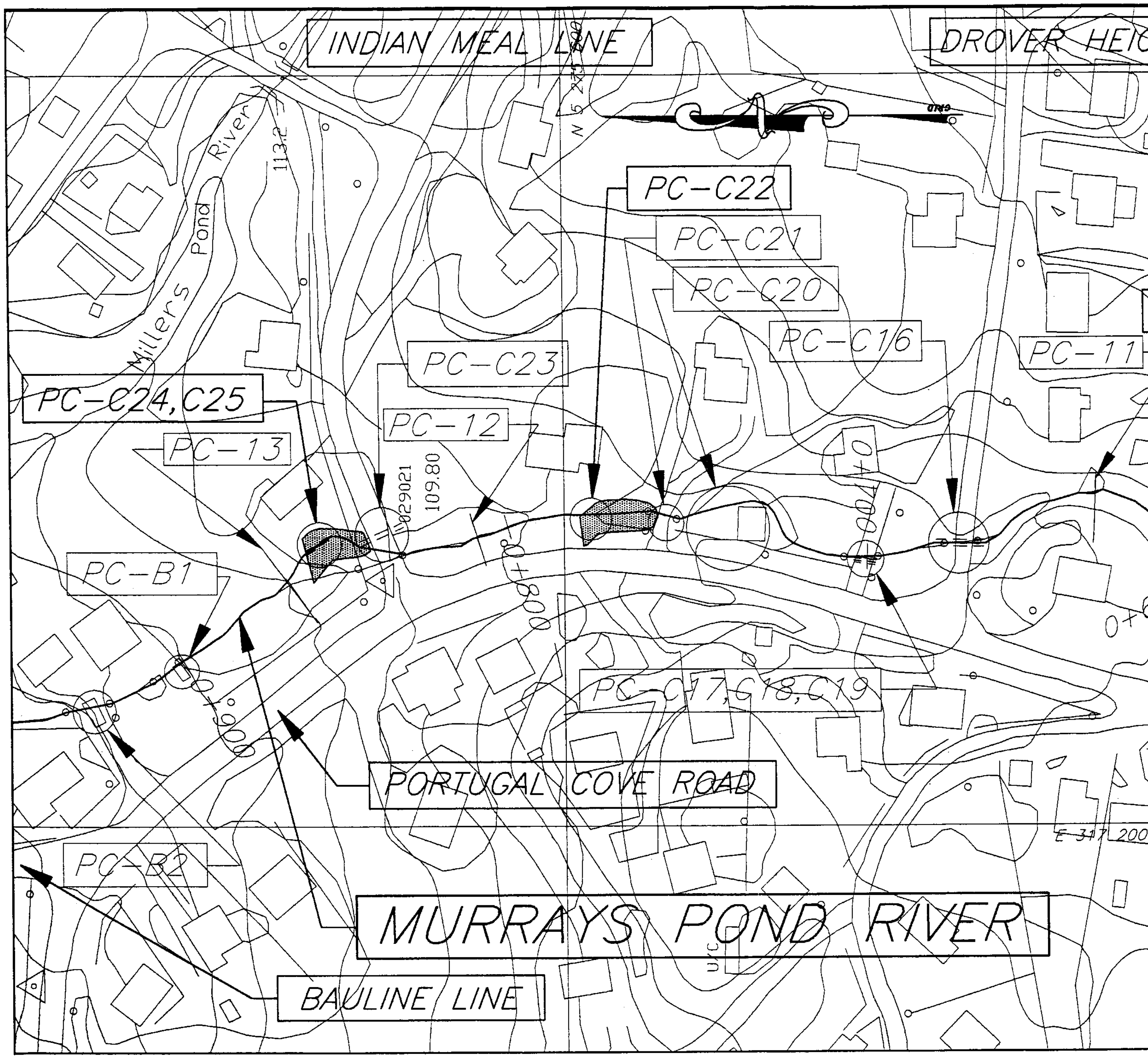


1:20 & 1:100 YEAR FLOODS
MAIN RIVER
PORTUGAL COVE, NF
SCALE: N.T.S.
Figure 27



UNDERSIZE CULVERT
FLOOD AREAS
MURRAY'S POND RIVER
PORTUGAL COVE, NF
SCALE: N.T.S.

Figure 28



UNDERSIZE CULVERT
FLOOD AREAS
MURRAY'S POND RIVER
PORTUGAL COVE, NF
SCALE: N.T.S.

Figure 29