CANADA-NEWFOUNDLAND
AGREEMENT RESPECTING
WATER RESOURCE MANAGEMENT

FLOOD RISK MAPPING STUDY

OF

CARBONEAR, VICTORIA, SALMON COVE, WHITBOURNE HEART'S DELIGHT, WINTERTON, HANT'S HARBOUR

Prepared For: -

Government of Newfoundland and Labrador
Department of Environment
Water Resources Division

Prepared By: -

Sheppard Green Engineering and Associates Limited

In Association With: -

CH2M Hill Engineering Limited

March, 1996

GOVERNMENT OF
NEWFOUNDLAND
AND LABRADOR

Department of Environment

Environment Canada
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This Permit Allows
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To practice Professional Engineering in Newfoundland and Labrador.
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NEWFOUNDLAND & LABRADOR

EXECUTIVE SUMMARY

The main objective of this study was to develop the 1:20 and 1:100 year return period flood peaks and associated backwater profiles for study reaches in Carbonear, Victoria, Salmon Cove, Whitbourne, Heart's Delight, Winterton and Hant's Harbour.

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

The field program extended from October, 1994 to March, 1995 and generally consisted of the survey of channel cross sections and permanent hydraulic structures, the installation of crest gauges and subsequent collection of water level measurements during various runoff events to establish stage versus discharge relationships, and interviews with residents in each community familiar with historical flooding problems. During the course of the field program, flooding was experienced in several communities as a result of high runoff experienced on January 7 and 8, 1995. Flood levels and damage associated with this event were documented through a site visit, personal interviews and photographs.

Computer simulation and statistical techniques were utilized to estimate peak flows and associated flood levels throughout the study area. In particular, peak flows were estimated using the Regional Flood Frequency Analysis regression equations and the instantaneous unit hydrograph model OTTHYMO. The BOSS HEC-2 model was used to derive the 1:20 and 1:100 year flood levels and to perform ice jam analysis.

The sensitivity of each hydraulic model was evaluated with respect to Manning's n, peak discharge, starting water elevation, ice cover and ice jams, and with respect to bridge and culvert structures. On the basis of historical flooding and the 1:20 and 1:100 year floodlines, alternative remedial measures were recommended for each study reach. The recommendations included both structural and non-structural measures. Preliminary cost estimates and location plans were prepared for each study reach.

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1.0 INTRODUCTION

Floods that occur throughout Canada are frequently the result of extreme discharges generated by extreme rainfall and snow melt. Occasionally, glacial outbursts, landslides into reservoirs, or sudden releases from storage such as dam failure also cause flooding. In Newfoundland, high water levels are often due to a reduction in channel capacity by ice and debris blockages or in some cases high tides. The relative importance of each of these mechanisms depends on the size and land use of the drainage basin.

On May 22, 1981, the Province of Newfoundland and the Government of Canada entered into a General Agreement Respecting Flood Damage Reduction (FDR). The objective of this agreement was to reduce flood damage by controlling the development of areas susceptible to flooding and by implementing sound flood plain management strategies.

To identify and delineate the areas susceptible to flooding, An Agreement Respecting Flood Risk Mapping was also signed on May 22, 1981. This agreement was subsequently amended and combined with An Agreement Respecting Flood Damage Reduction in 1988. As a result of combining these Agreements, flood prone areas of the Province were mapped, flood risk areas were delineated, and areas were designated as suitable for only certain types of developments. However, flooding problems were later experienced in over forty (40) communities not included in the original FDR program.

To assist in the protection and conservation of Newfoundland's water resources, an Agreement Respecting Water Resource Management was signed in July 1993. Under this Agreement, flood risk mapping and studies can be carried out in areas not included in the original FDR program; Carbonear, Heart's Delight, Winterton, Victoria, Whitbourne, Salmon Cove and Hant's Harbour are communities that qualified for flood risk studies and mapping to assist in the formulation of Water Resource Management strategies.

1.1 Authorization and Scope of Study

The agreements cited in the aforementioned paragraphs provide for the establishment of two committees; the Steering Committee responsible for general administration of the agreements and the Technical Committee to provide technical support to the Steering Committee.

On August 18, 1994, Sheppard Green Engineering and Associates Limited (SGE), in association with CH2M HILL Engineering Limited (CH2M HILL), were commissioned by the Newfoundland Department of Environment (DOE) to conduct a ***Flood Risk Mapping Study of Carbonear, Victoria, Salmon Cove, Whitbourne, Heart's Delight, Winterton, and Hant's Harbour.***

The objectives of the study are:

1. Maximize the use of existing information, including historical data, to obtain an understanding of the flooding problems and the factors responsible for past flood events in the study area;

2. Evaluate the significance of the various factors affecting flooding problems in the study area;

3. Design a strategy to produce 1:20 and 1:100 year recurrence interval flood profiles for the study area;

4. Select appropriate numerical model(s) capable of simulating the hydraulic behaviour of the study area;

5. Design, co-ordinate and manage a field program to collect any new information required to establish and plot historical flood levels, calibrate and/or verify the numerical models;

6. Review pertinent data and undertake the necessary office studies to fill data voids. Hydrometeorlogical data provided by other agencies must be checked for accuracy;

7. Calibrate, and verify, to the extent possible, the selected numerical models, using split sample techniques;

8. Carry out a sensitivity analysis of the flood profiles to identify overall error;

9. Produce the 1:20 and 1:100 flood profiles for the communities in the study area and plot the flood lines on 1:2500 scale provincial topographic series mapping;

10. Identify and evaluate appropriate remedial measures to alleviate any potential flood damage problems;

11. Establish and maintain an effective reporting mechanism that will facilitate accurate monitoring of the technical resources applied and costs allocated to each Activity/Task.

1.2 Description of Study Area

The study area is located in the western part of the Avalon Peninsula of the Island of Newfoundland. Much of the study area is characterized by barren, irregular and rough topography with numerous rock outcrops. The soil cover is generally thin, and the proximity of bedrock has led to the formation of many bogs and ponds.

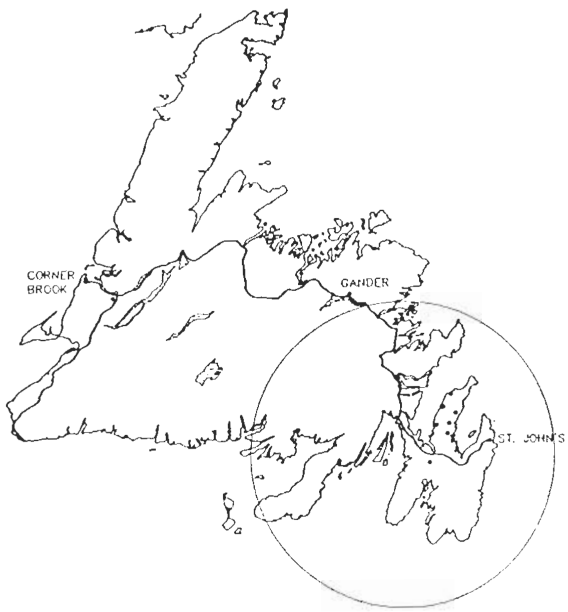
The climate is cool, temperate, and wet as expected from the proximity of the area to the North Atlantic Ocean. The cold Labrador current keeps summer temperature low; the warmest month is August, with a mean temperature at both Argentia and Colinet of about 15° C. The ocean moderates the climate in the winter, and mean temperatures in February, the coldest month, at Argentia and Colinet are -2° C and -4° C respectively.

1.3 Overview of Study Methodology

The accurate determination of the 1:20 and 1:100 year profiles along the study reaches depend on several hydrological and hydraulic factors, including the following:

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

FIGURE 1.1 - LOCATION PLAN



The initial activity in the investigation was the collection and review of available background information and existing data on climatologic and flood characteristics. This is discussed in Chapter 2.0 - Background Information.

The next step was the determination of appropriate 20 and 100 year peak discharge rates. Alternative estimates were derived and compared in Chapter 4.0 - Hydrologic Analyses. This included model calibration and sensitivity analysis in order to achieve an appropriate level of accuracy for the discharge estimates.

Upon completion of the hydrologic analysis, the peak flow estimates were converted to flood water levels along the study reaches by using the Hec2 Model. This analysis also included sensitivity testing of the most important hydraulic parameters and relevant model calibration and verification using documented events.

Finally, the extent of the flood hazard areas were plotted in a digital format on 1:2500 topographic mapping for the study area. Upon completion of the flood risk mapping, potential remedial measures were considered to alleviate the potential for future flood damage These measures are identified in Chapter 6.0.

The "Hydrologic and Hydraulic Procedures for Floodplain Delineation" and "Survey and Mapping Procedures for Floodplain Delineation", developed by Environment Canada were followed where appropriate throughout the course of these investigations.

2.0 BACKGROUND INFORMATION

2.1 Historical Floods and Previous Studies

This section summarizes the historical flooding problems throughout the study area that were previously documented. During the course of this study a significant runoff event occurred on January 7 and 8, 1995 and a summary of the findings of a field investigation was submitted to the DOE on January 16, 1995. Appendix A of the Technical Appendices contains photographs of each study reach while Appendix B contains photographs that document the flood damage of the January 7 and 8, 1995 flood event.

<u>Carbonear</u>

The Town of Carbonear experiences flooding at several locations on Island Pond Brook and Powell's Brook. With respect to Island Pond Brook, gravel accumulates and causes flooding of the land surrounding the concrete bridge and train trestles located at the outlet of Carbonear Pond. Flooding occurs at two locations along Powell's Brook: 1) Below the Highroad South concrete bridge where a section of the railway right of way is subject to erosion and, 2) the section of Powell's Brook upstream of Powell's Drive near the movie theatre experiences sediment build up which again causes flooding; refer to Figure 2.1.

<u>February 10-11, 1990</u>

Heavy rains caused extensive flooding in Powell's and Valley Brooks. Roads were washed away when ice broke loose in these brooks and caused considerable damage with roads washed away in Irishtown, Valley Road, Lower Southside and Powell's Drive.

<u>Victoria</u>

The Town of Victoria frequently experiences flooding on Salmon Cove River; known locally as Powell's Brook; refer to Figure 2.2. The flooding typically occurs 400 to 500 m downstream

FIGURE 2. 1 - CARBONEAR SITE PLAN

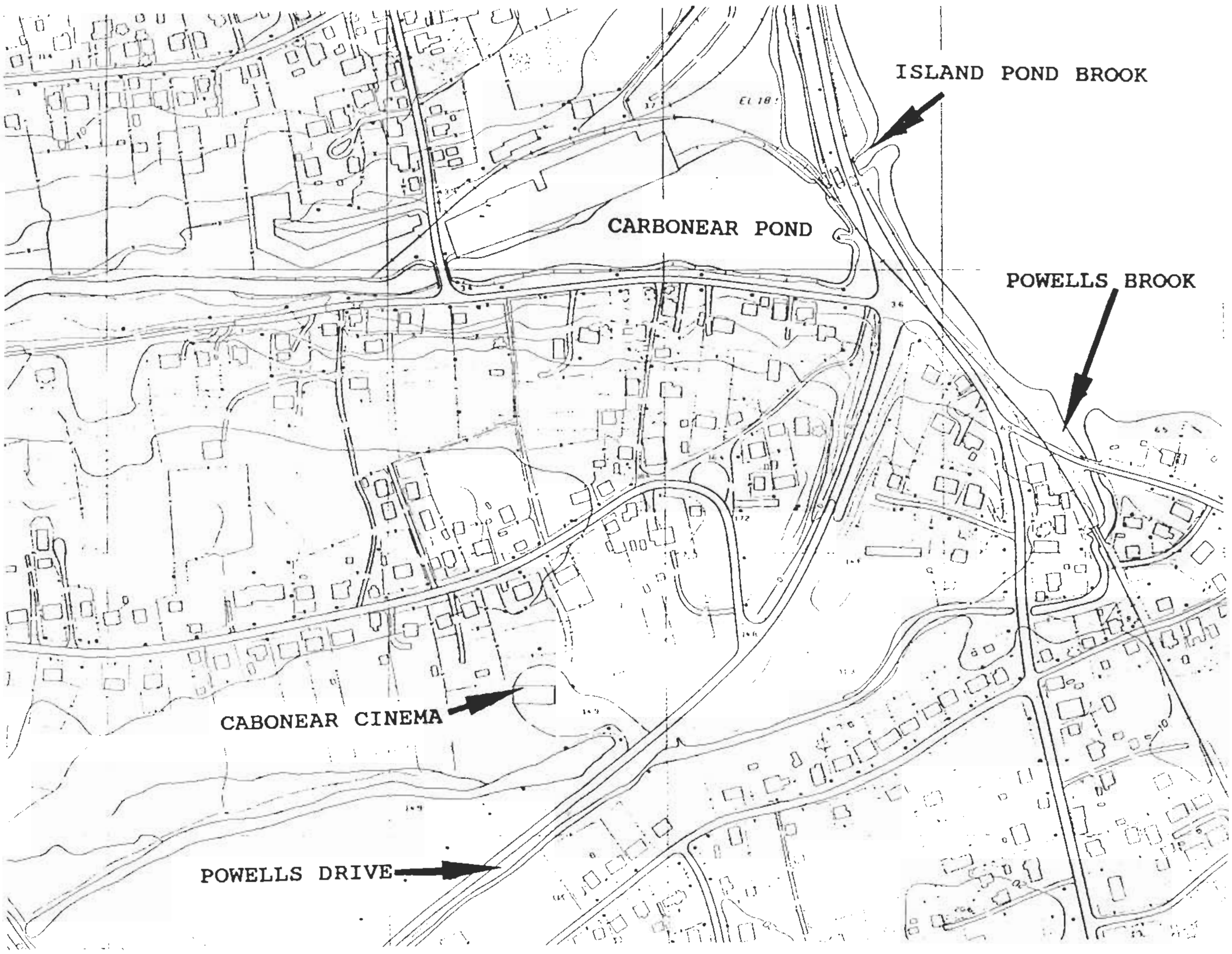
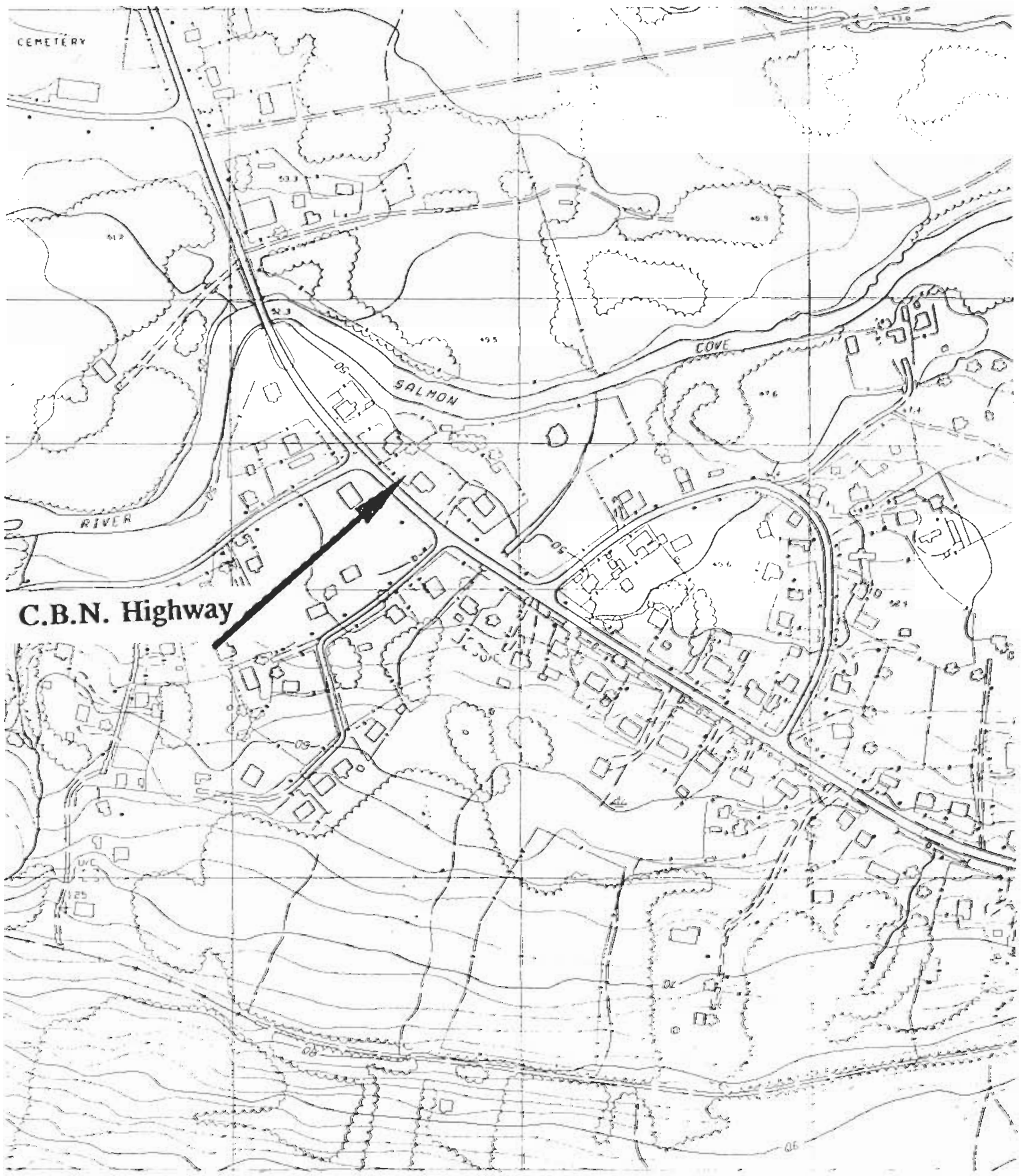


FIGURE 2.2 - VICTORIA SITE PLAN



of the bridge on the Conception Bay North Highway mainly due to ice jamming. Berms have been proposed for the riverbanks to prevent further flooding of several residences adjacent to the river which have main floor levels on, or near, ground level.

February 1 - 2, 1994

An excavator was used to remove an ice jam to prevent flooding on this occasion.

February 16, 1991

Heavy rains combined with snow melt to cause flood damage in several residences.

January 27, 1990

The cover of the river broke and jammed approximately 400 m downstream of the bridge. Some of the ice and water bypassed the ice jam and flooded the surrounding area.

February 10 - 11, 1990

The ice jammed in the same location as January 27, 1990. However, the jam occurred during the night of February 10, 1990. Due to darkness and the fragmented ice cover which were flowing through the area, measures to rectify the situation were delayed until daylight; three houses were evacuated.

May 4, 1990

An ice jam was cleared before serious damage could occur.

In 1990 Robert Picco of the Water Resources Division of DOE prepared a report on Flooding in Salmon Cove River. This report looked at the history of flooding in the area, previous work that had been performed, and made recommendations to minimum future flooding.

FIGURE 2.3 - SALMON COVE SITE PLAN

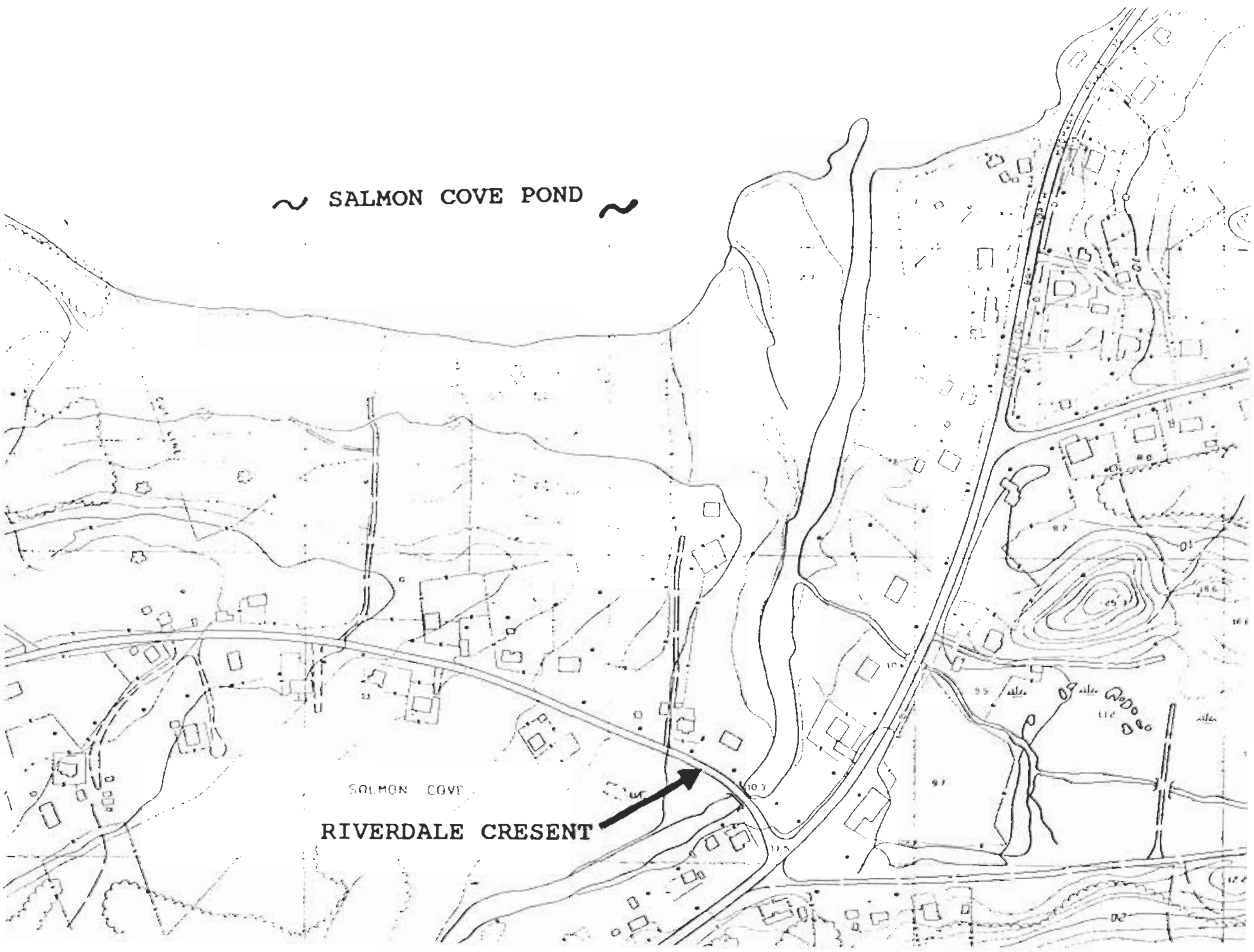
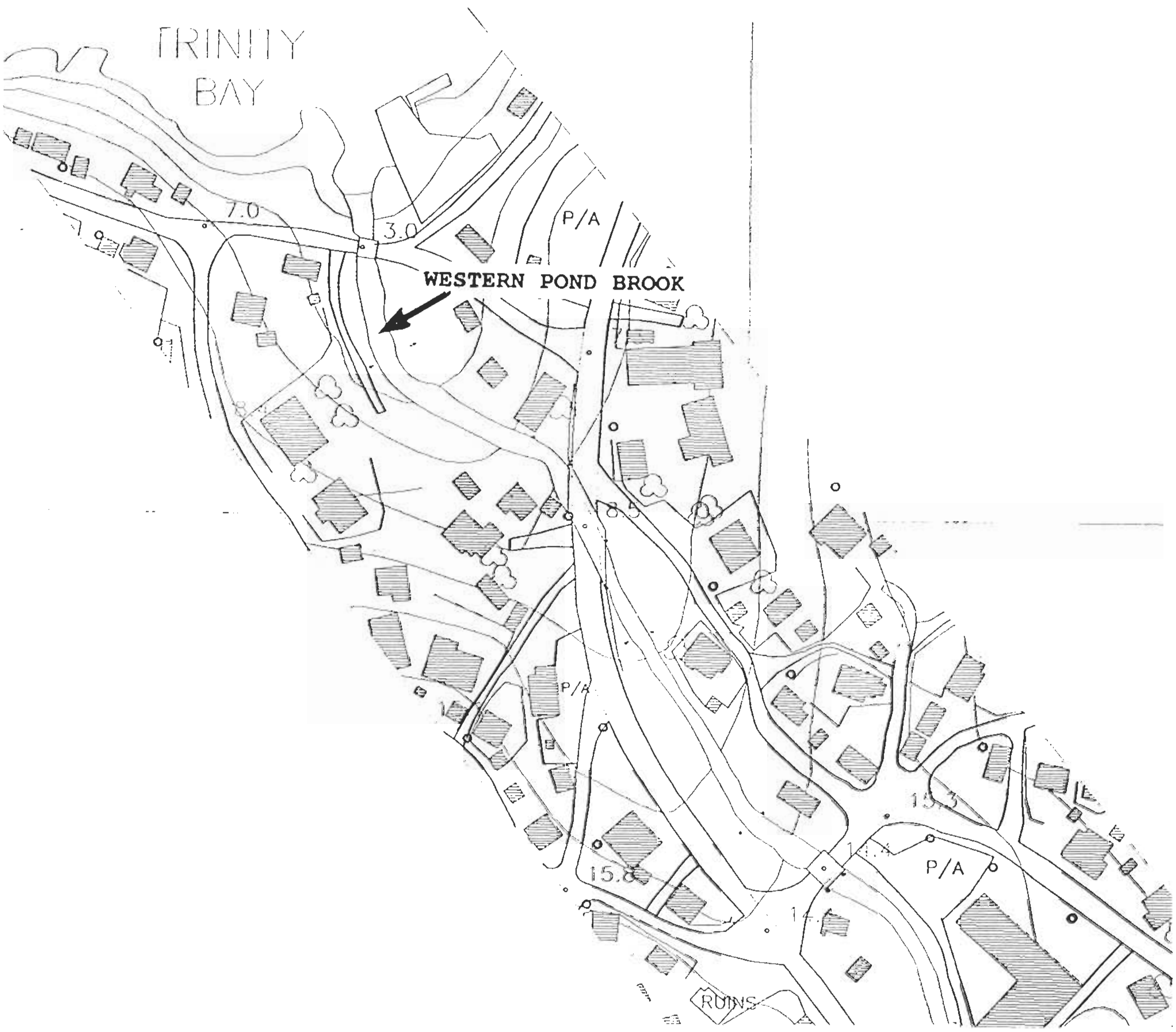


FIGURE 2.4 - WINTERTON SITE PLAN



Salmon Cove

Prior to 1992, Salmon Cove frequently experienced flooding of Riverdale Crescent. In 1992 the culverts under this road were replaced with a bridge. The areas previously experiencing flooding have not had any problems since this construction, but a previously unflooded area downstream of the bridge flooded in February 1994. Flooding problems have also been experienced on Slades Avenue and on Birch Cliff Drive at the intersection of Ridge View Road. In both of these situations, a culvert under the road either becomes blocked or is inadequately sized and consequently caused flooding of the road and nearby residences; refer to Figure 2.3.

Winterton

The Town of Winterton has experienced flooding along Western Pond Brook. The flooding usually occurs as a combination between rainfall and snowmelt, however a likely contributing factor was the 'skewed' alignment of the old bridge on Route 80 at Western Pond Brook; refer to Figure 2.4

February 15, 1991

Approximately 63 mm of rain, combined with rapid snow melt as a result of 11°C temperatures, caused flooding conditions. The Water Resources Division of DOE estimated this flow to equal the 25 year reoccurrence interval flow. The two upstream bridges were cleared of fragmented ice but water flowed over the highway and also overtopped the river banks.

In 1991, Peter D. Lester, Chief Bridge Engineer, Department of Works Services and Transportation, (DWST) prepared a report on the Winterton Bridge. This report details the flood which occurred in February 15, 1991, and describes the history of flooding in the area and makes recommendations. In 1986, Martin Goebel, DOE, prepared an investigation of flooding

FIGURE 2.5 - WHITBOURNE SITE PLAN

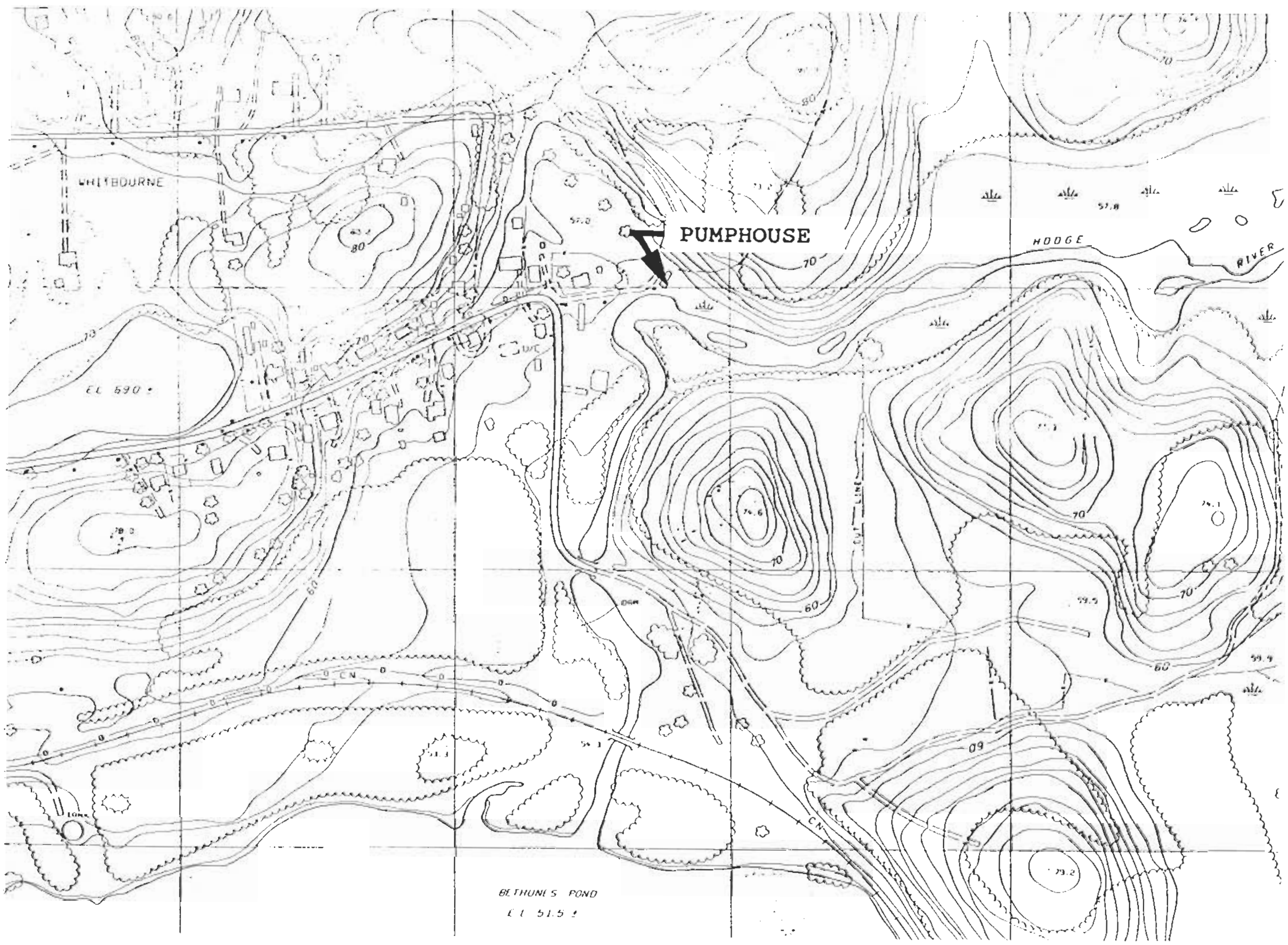
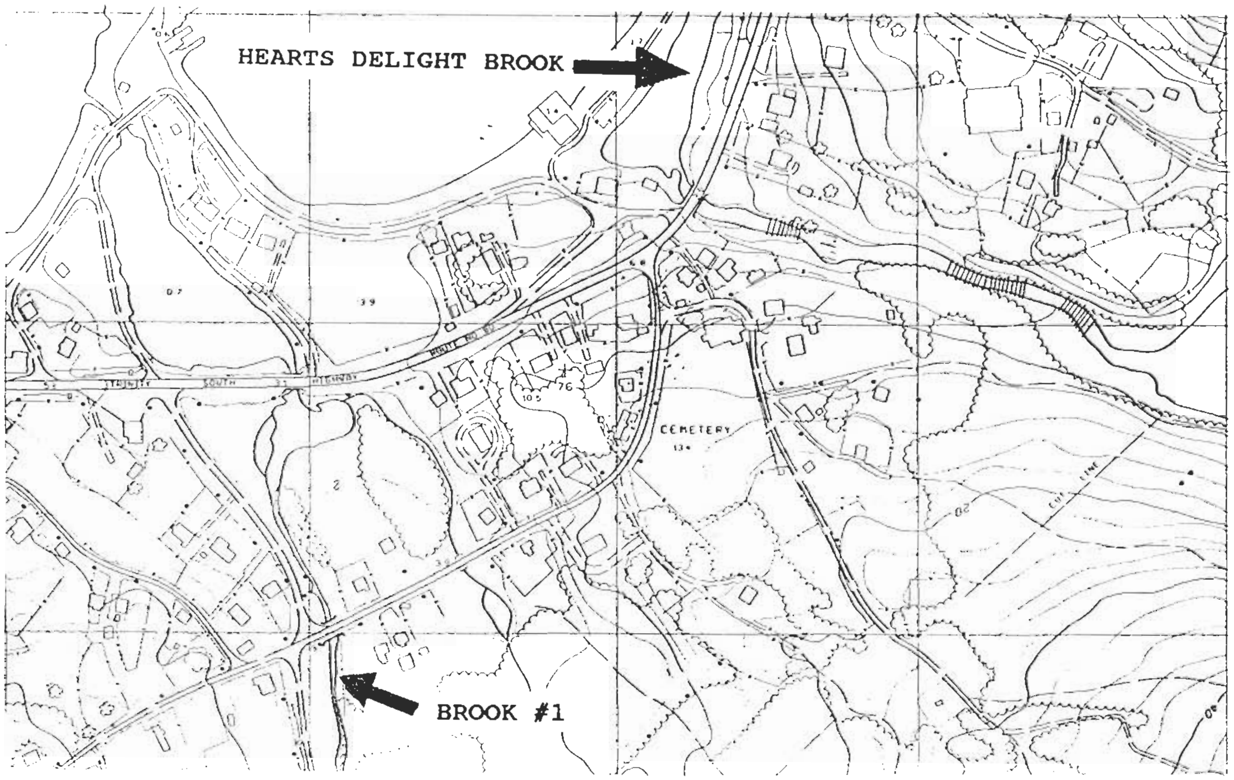


FIGURE 2.6 - HEART'S DELIGHT SITE PLAN



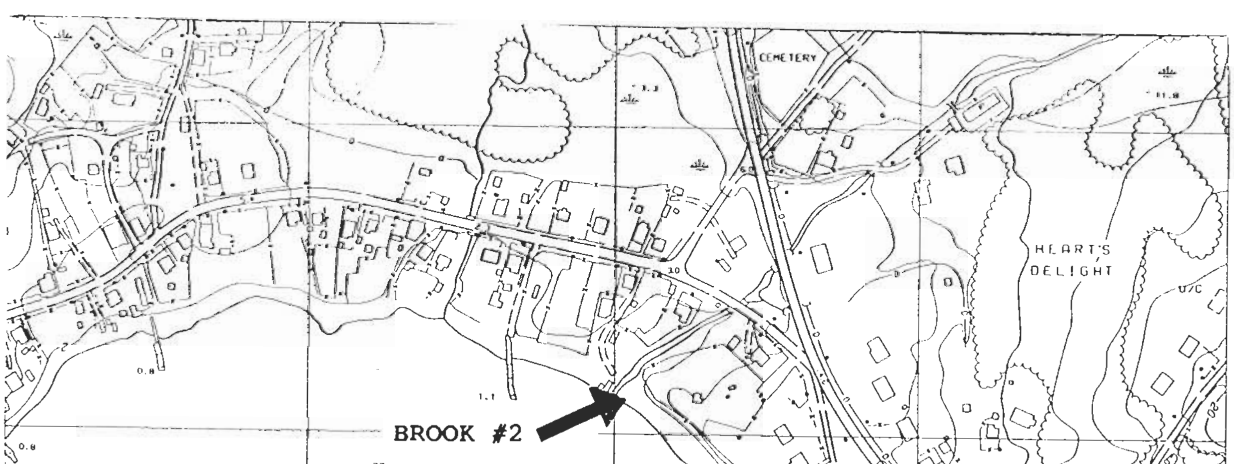
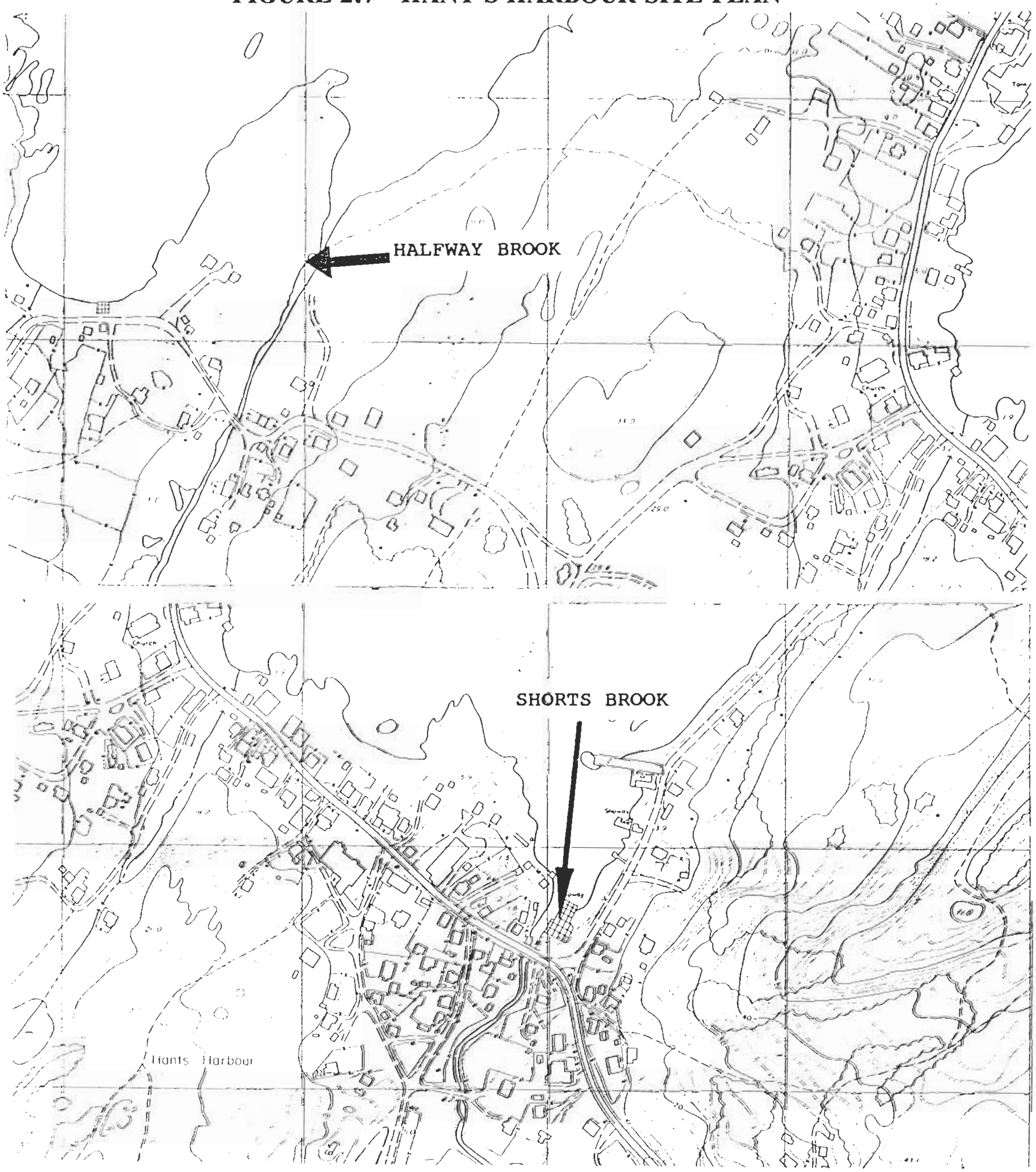


FIGURE 2.7 - HANT'S HARBOUR SITE PLAN



on Western Pond Brook; this report investigated ice related flooding on the brook and made several recommendations.

Whitbourne

Stephen Young prepared a report in 1988 on flood damage reduction for the Hodges River in Whitbourne. This report investigated historic flooding events, set up a hydraulic model on the river, and used the model to simulate for high flow periods and structural constrictions. Some flood damage reduction options were also presented.

Heart's Delight

The Town of Heart's Delight has experienced flooding on Heart's Delight Brook. A combination of factors such as high flows in Main Brook, high tides, storm or wind induced surge, and blockage of the Barachois outlet by ice or gravel are believed to cause the flooding. The residence of Mr. Vernon Mercer has been flooded on several occasions; refer to Figure 2.6.

Flooding has also occurred on the river (Brook #1) that drains the water supply. In this case, a culvert crossing Church Road appears undersized and results in the flooding of two nearby residences.

Brook #2 in the East part of the town has also experienced flooding. This brook flows under Route 80 and secondary road. An insufficiently sized culvert causes flooding of a nearby residence.

In December, 1989, Martin Goebel investigated the flooding on Heart's Delight Brook and determined the elevations of the physical features in the area. He also prepared a report on the history of flooding, the survey results, recommended remedial measures, and made some recommendations.

Hant's Harbour

Hant's Harbour experienced flooding in Halfway Brook on February 2, 1987 when the brook was blocked by ice accumulation. With the brook more or less filled with ice, rain and snowmelt produced runoff which caused the brook to overflow. The flow of water was about 30 to 40 cm over the road. A summer home in the area was completely surrounded by water and access to five other homes and two summer cottages were inaccessible; refer to Figure 2.7.

A site investigation was performed by Martin Goebel on February 2, 1987. At this time he prepared a report which included a background investigation, a report of damages, and recommendations for remedial works.

2.2 January 7 and 8, 1995 Flood Event

The following comments/observations are based on a review of the flooding problems in Carbonear, Victoria, Salmon Cove, Whitbourne, Heart's Delight, Winterton and Hant's Harbour on January 9, 1995 following the rainstorm event of January 7 and 8, 1995. Photographs that completely document the damage associated with this event are in Appendix "B" of the Technical Appendices while photograph 2.1 and 2.2 have been included in this section for illustration purposes.

Rainfall recorded at St. John's Airport was 38.2 mm on Saturday and Sunday.

The following problems were caused by heavy rainfall, warm temperatures and a significant amount of snow melt. Ice blockage was not a factor.

PHOTOGRAPH 2.1 - POWELL'S BROOK (CARBONEAR)

PHOTOGRAPH 2.2 - SHORT'S BROOK (HANT'S HARBOUR)

Carbonear

- Jim Burden, working foreman with the Town of Carbonear, was used as a tour guide to summarize flooding damage.

- The problems on Powell's Brook peaked in the early morning hours of Sunday, January 8, 1995 (Burden, 1995).

- Major road erosion occurred at both culverts installations on Industrial Loop. The roads were over-topped at both locations.

  Adjacent to Newfoundland Power, the road erosion was more severe on the downstream side of the road.

- At Carbonear Cinema (looking downstream) the two culverts on the left conveyed most of the flow while the two culverts on the right were approximately half full.

  The road was not over-topped but Powell's Brook, spilled onto the parking lot of the Carbonear Cinema and Dicks & Co. Office Supplies. Powell's Drive experienced erosion.

  The peak water elevation in the parking lot reached the concrete steps in front of the Carbonear Cinema.

- At the train trestle on Powell's Brook, the river over-topped at the bend upstream of the train trestle. There was noticeable erosion at this location.

- The rail bed adjacent to train trestle at the outlet of Carbonear Pond was overtopped.

- The damage at intersection of Pond Side Road and Beach Road was due to the inadequacy of the culvert installation at the Carbonear Cinema. The damage was not caused by a blockage at the outlet of Carbonear Pond (Burden, 1995).

- Pond Side Road did not flood.

- Valley Road was partially submerged near the residence of Mr. Clarke. The double culvert installation was inadequate and the access road was almost completely destroyed.

  The downstream flooding was caused in part by two old concrete bridge abutments in the river.

- At the intersection of Pikes Lane and Beach Road, there was some flooding and the portions of Pikes Lane was submerged (Burden, 1995).

- The triple culvert installation on CBN Highway conveyed the runoff without incident (Burden, 1995).

- Downstream of the triple culvert installation, but upstream of the bridge at the intersection of Pond Side Road and Cross Road, the river banks were overtopped.

- Water levels in Carbonear Pond reached the underside of the train trestle.

- The outlet of the concrete bridge on Carbonear Pond was considerably altered from the last observations. Flow is now directly through the bridge.

- The flow on Island Pond Brook at 10:30 A.M. on January 9, 1995 was at its highest point. Water level immediately upstream of bridge was to the underside of "K" in "CHUCK" which is painted on the old concrete abutment. Water was approximately 1.0 metre below low chord.

Victoria

- Mr. Clyde Antle stated that river flow reached the top of a triangular rock behind his shed. There was some minor spillage of water due to turbulent conditions but no major flooding.

- The flooding on his property was due to surface runoff and inadequate driveway culverts on his property.

- The water level at the concrete bridge on the CBN Highway reached the top of the PVC deck drain on left abutment (looking downstream).

Salmon Cove

- Met with Mr. Gary Butt, Mayor, Mrs. Jacqueline Deering, Town Manager, and Mr. Sterling Slade, Councillor.

- Salmon Cove Brook at concrete bridge on Main Road overtopped the river banks and spilled onto Forest Road. This caused considerable damage to surrounding property and main road.

- The new concrete bridge on Riverdale Crescent was overtopped with approximately 200 mm of water on the deck during peak conditions (Slade, 1995).

- At 12:00 noon, January 9, 1995, the river elevation reached the underside of the bridge on Riverdale Crescent.

- Nelson Penney (white and green house adjacent to the bridge) had approximately 450 mm of water in basement and confirmed observations of Mr. Sterling Slade regarding water overtopping the bridge.

- White bungalow on right hand side of river (looking downstream), had back lawn submerged by 150 mm of water (Slade, 1995).

- There was no flooding on Birch Clift Drive.

- Salmon Cove Ridge Road transects the ridge to the south of Salmon Cove. The culverts which cross the road were blocked and/or inadequate. Consequently, surface runoff from the ridge was diverted along the road way and caused considerable erosion.

- The concrete bridge adjacent to the United Church was overtopped. Flooding occurred on both sides and at a grey two-storey house the water rose to 8" below main front window (Slade, 1995).

- Mr. Slade made the following comments with respect to peak flood conditions in Salmon Cove:

- There was approximately 200 mm of water at the base of the concrete War Memorial;
- There was 300 mm of water over top of concrete bridge near United Church;
- The fire hydrant near the Church was 50% submerged;
- At the L.O.L. Lodge, water reached the top of concrete foundation wall;
- The main road to the Municipal Building was submerged with approximately 50 mm of water;
- The bridge on CBN Highway conveyed flow without incident;
- At Mr. Edison King's white 2-storey house, water reached to the top of the patio.

Winterton

- Mrs. Joan Hiscock, Town Manager and the Superintendent of Works stated that there was no flooding at the new bridge on Route 80. However, the river channel was full and there was minor spillage onto Route 80.

Heart's Delight Brook #1

- Mr. Pottle stated that water levels reached the underside of the clapboard on his house.
- There was approximately 300 mm of water on Church Street when the flood was at its peak. (Pottle, 1995)

- The summer home belonging to Mr. Kirby had water up to concrete step.

Heart's Delight Brook #2

- Mr. Albert Reid stated that flood levels were at the finished floor elevation of his house and to the bottom of the door threshold on the blue bungalow downstream of the second culvert installation.

- Flood waters extended along roadway to the intersection behind Mr. Albert Reid's.

- The upper culvert installation had sufficient capacity to pass the flow and Route 80 was not overtopped. The second culvert installation was overtopped. This was caused by poor alignment with respect to flow direction.

Heart's Delight (Heart's Delight Brook)

- Mr. Vern Mercer stated that there was no flooding on Heart's Delight Brook but the water level in the Barachois was significantly increased. The river was discharging normally and the outlet was approximately 15 metres wide.

- The water level was almost at the top of the embankment adjacent to his property.

- The old concrete abutment downstream of existing concrete was partially submerged.

- Significant erosion occurred at the upstream face of the concrete bridge on Route 80.

- The flow under the bridge reached the lower chord (Mercer, 1995).

<u>Hant's Harbour (Shorts Brook)</u>

- Mrs. Doris Short, Town Manager, stated that peak flow occurred at approximately 7:00 A.M. on Sunday, January 8, 1995. The river level at this time was approximately 100 mm from the lower chord of the bridge on the main road.

- The triple culvert installation upstream of concrete bridge was overtopped and there was significant erosion.

- There was minor flooding on Marsh Road which runs parallel to Shorts Brook.

<u>Whitbourne</u>

- Mr. Lloyd Gosse and Mr. Gary Phillips stated that river flow increased significantly but no flooding occurred.

2.3 Relative Importance of Factors Affecting Flooding

The factors affecting or contributing to flood events vary according to the site. It is necessary to identify and evaluate site specific factors in order to identify areas of further investigation and/or plan cost effective floodplain management strategies. A qualitative assessment of the relative importance of factors contributing to flood events will provide input for the quantitative evaluation pertaining to the 1:20 and 1:100 year flood events.

This section of the report was prepared by reviewing the existing information on historical flooding and through a field reconnaissance program conducted on October 3 and 4, 1994. Historical information was reviewed in light of the information that was recorded during the field reconnaissance program.

2.3.1 Field Observations

The study team conducted a field reconnaissance program on October 3 and 4, 1994. Both map studies and a detailed 'walkover' of each study reach was conducted. Morphological features such as rapids, boulders, river bends, and constrictions along with permanent structures in the study reaches were identified, photographed and documented. Field notes were compiled and discussions were held with Town Council representatives. The field reconnaissance program also served as the basis for planning the detailed field program.

Carbonear

The Town Council office in Carbonear was visited on October 3, 1994 and the flooding problems were discussed with Mr. Jim Walsh (Town Manager). The problem areas in Carbonear were identified and discussed prior to proceeding to the field. It was revealed through these discussions that the full scope of the recent flooding problems in Carbonear was not fully addressed in the Terms of Reference (TOR).

Powell's Brook

The culvert installation on Industrial Crescent adjacent to Penny's Transport building consists of four 1780 x 1150 mm arch culverts. Existing documentation indicates that sediment build-up at this location has been the primary cause of the flooding problems. However, since the flooding problem generally occurs during the spring break-up, it appears that sediment build-up, combined with high flows and ice and/or debris, is the condition most likely to result in flooding.

The second culvert installation on Industrial Crescent (Downstream of Penny's Transport), also consists of four 1780 x 1150 mm arch culverts. There was significant accumulation of sediment immediately upstream of the culvert inlets. The accumulation was so severe that the inlet of one culvert was completely blocked with debris. The main channel flow was conveyed by two culverts rather than all four.

The culvert installation on Powell's Drive, adjacent to the movie theatre, consists of four (4) 1780 x 1150 mm arch culverts. The main channel flow is conveyed through the two (2) culverts on the left side looking downstream while sediment has accumulated at the inlets of the other two culverts. Even though sediment accumulation is a factor, the alignment of the culverts, with respect to stream flow appears to be the main contributing factor.

The flooding problem in the vicinity of the Lower Southside Road bridge on Powell's Brook is caused by several factors. River bends, constrictions, rapids and large boulders are predominant features in this study reach. The river channel is so irregular that it has two, 60 degree, changes in flow direction over a distance of approximately thirty (30) metres (m). These changes in flow direction are immediately upstream of the train trestle and, in all likelihood, cause significant backwater effects resulting in the flooding adjacent residential units.

Island Pond Brook

At the concrete road bridge on Island Pond Brook, downstream of Carbonear Pond, flooding is caused by the combination of high tides, inadequate effective flow area, and low surrounding terrain.

A new triple culvert installation on the Conception Bay North Highway appears to have alleviated the flooding problem on Island Pond Brook where the 'old' highway bridge was located. Steel piles installed immediately upstream of the new culverts prevent ice blockage at the inlets of the culverts but may divert water to the recreational field adjacent to the river.

Victoria

Mr. Walter Hiscock, Town Manager, was consulted prior to proceeding with the preliminary field investigation. A review of existing information indicated that flooding historically occurred approximately 500 m downstream the bridge on the Conception Bay North Highway. Morphological features, including a river bend, large boulders and a minor constriction, combined with ice sheets, high flows and low lying terrain have caused past flooding events.

Salmon Cove

The bridge on Salmon Cove River, which provides access to Riverdale Crescent, was replaced in 1992 and since that time flooding has occurred on only one occasion; see Section 2.2. It appears that past flood events have been caused by the combination of high flows, ice sheet accumulation and an inadequate hydraulic structure.

With respect to the flooding problems on Slades Avenue and Birch Clift Drive, the problems at these locations are caused by a lack of ditching and blocked culverts.

Whitbourne

Mr. John Vokey, Superintendent of Works, stated that past flood events on the Hodges River in Whitbourne were caused by the combination of high flows, ice blockages and an inadequate bridge structure.

Heart's Delight

Mr. Stanley Legge (Superintendent of Works) was consulted with regard to the nature and location of the flooding problems in the community.

There are three locations in Heart's Delight which have experienced flooding in the past. With respect to the flooding problem on Heart's Delight Brook, it is well documented that the flooding problems are caused by high flows, wind induced surge and blockage of the barachois outlet. It is worthy of note that Heart's Delight Brook is significantly constricted and changes direction immediately downstream of the new bridge where the 'old' bridge abutment exists. However, the preliminary field reconnaissance program revealed that the flooding problems are due, in large part, to the location of the residental property. In particular, the elevation of Mr. Mercers house is extremely low in relation to the water level in the Barachois. With respect to the other two (2) locations, the problem is caused by high flows, sediment accumulation, ice blockage and inadequately sized culverts.

Winterton

The bridge on Western Pond Brook was replaced in 1992. The new bridge was re-aligned to reduce the 'skew' that was apparently the main factor contributing to the flooding problem.

Hant's Harbour

During the field reconnaissance program, it became apparent through discussions with the Mrs. Short, Town Clerk, that the main flooding area in Hant's Harbour is on 'Shorts Brook' rather than Halfway Brook.

With respect to Halfway Brook, the past flood events have been caused by a combination of high flows and ice blockages at the bridge. Tidal effects may also have contributed since the elevation of the river is very close to sea level at the bridge location.

With respect to the flooding problems on Short's Brook, the main contributing factors appear to be high flows, ice accumulation, and inadequate hydraulic structures.

2.3.2 Summary

The field reconnaissance program, in conjunction with the literary search, revealed important information and a greater understanding of the flooding problems. In particular, the scope of the flooding problem in Carbonear was greater than anticipated and consequently, the field program was expanded.

The flooding problem in Hant's Harbour was discovered to be greater on Short's Brook rather than Halfway Brook. This information was evaluated and again, the detailed field program was modified such that all relevant information was collected for two (2) study reaches rather than one (1).

In virtually all study reaches, the flooding problems can be attributed to a combination of morphological features, high flows, and ice conditions. Tidal considerations are important at Island Pond Brook in Carbonear, Heart' Delight Brook and both brooks in Hant's Harbour. Inadequate hydraulic structures also contribute significantly to flooding problems throughout the study area.

2.4 Interviews

During the field reconnaissance program of October 3 and 4, 1994 and throughout the detailed field program, interviews were conducted local residents familiar with the historical flooding problems in each community. The purpose of the interviews was to:

- Obtain a better understanding of historical flooding;
- Compile and review existing photographs, videotape, documentation, etc.; and,
- Verify the adequacy of the preliminary field program.

Carbonear

- Mr. Niall Butt, Town Clerk, was interviewed on October 3 and 26, 1994. Mr. Butt stated that the flooding problem on Island Pond Brook was most serious in the vicinity of Carbonear Pond. However flooding has occurred at the bridge adjacent to the intersection of Pond Side Road and Cross Road. This information was collaborated on October 26, 1994, by Mr. Jim Walsh, Town Manager.

- Mr. Croke was interviewed on October 27, 1994, and stated that flood levels in the vicinity of the Carbonear Cinema typically reach the level of the concrete steps at the entrance of the cinema.

Victoria

- Mr. Walter Hiscock, Town Manager, was interviewed on October 3, 1994, and verified what is already well documented; that the flooding problem generally occurs 400 to 500 metres downstream of the bridge on Route 80 and adjacent to the property of Mr. Clyde Antle.

- Mrs. Sharon Snook, Town Clerk, was also interviewed and agreed with virtually everything Mr Walsh stated.

- Mr. Clyde Antle was interviewed on November 28, 1994. He outlined the limits of historical flooding and agreed to maintain the crest gauge which was installed near his residence.

Salmon Cove

- Mrs. Jacqueline Deering, Town Manager, was interviewed on October 3, 1994. She outlined the flooding history of Salmon Cove river at the bridge entering Riverdale Crescent.

- Mr. Nelson Penny was interviewed on October 28,1994. He stated that past floods have reached the finished ground elevation adjacent to his property. He agreed to maintain the crest gauge installed upstream of the bridge.

Whitbourne

- Mr. John Vokey, Superintendent of Works, and Mrs. Wanda Lynch, Town Clerk, were interviewed on October 4, 1994. The flooding problems on Hodges River were discussed. However, very little documentation was available from the Town.

- Mr. Gary Philips was interviewed on December 13, 1994. Mr. Philips lives adjacent to Hodges River and has personally experienced the flooding on several occasions. In 1993, there was an ice blockage at the new bridge and at the river bend immediately at the rear of his property. During this event Mr. Philips had 42 inches of water in his basement and a portion of his property was submerged.

Hearts's Delight

- Mr. Stanley Legge, Superintendent of Works, is locally regarded as the authority on past floods in this community. Mr. Legge was interviewed on October 4, 1994 and confirmed what is already well documented.

- Mr. Vernon Mercer and Mr. David Harnum were interviewed on November 23, 1994. Again, the problem on Heart's Delight Brook was stated to be caused by high tides, wind induced surge and blockage of the river outlet. Both stated that there has never been flooding associated with ice blockage at the road bridge on Route 80. However, ice does accumulate at the 'old' concrete abutments downstream of the existing bridge.

- Mr. Mercer has placed approximately 70 loads of fill between his property and the barachois. This seems to have minimized the water infiltration in his basement during recent high water levels.

Winterton

- Mr. Lionel Piercy was interviewed on November, 22, 1994 and stated that past flood events have resulted in water levels reaching the finished ground elevation adjacent to his property. However, he stated that the new bridge has eliminated the skew, improved channel flow, and eliminated ice blockages. He also stated that there have not been any serious increase in water level at the new bridge.

- Mr. Ernest Swaggart was interviewed on November 22, 1994 and confirmed the statements of Mr. Piercy. Both agreed that maximum water elevations of past flood events were reached when the old bridge was overtopped. Flooding of downstream houses was the result of the bridge blockage.

- A video of the 1991 flood event was reviewed at the Town Office. Mrs. Joan Hiscock furnished the video but was not able to provide any additional information to document past flood events.

Hant's Harbour

- Mrs. Doris Short was interviewed on October 4 and November 21, 1994. Mrs. Short stated that the most serious flooding problem occurs on Short's Brook rather than Halfway Brook. The most serious event occurred in 1992 and was caused by a washed-out culvert being jammed at the bridge on the main road. The flood level was approximately 150 mm from the finish floor elevation of her house which immediately upstream of the road bridge.

- Mrs. short also stated that the flooding on Halfway Brook was caused by an ice blockage at the bridge and probable influence from high tides.

3.0 FIELD PROGRAM

3.1 Background

The ultimate objective of the field program was to gather the information to produce flood risk profiles and maps for Carbonear, Victoria, Salmon Cove, Whitbourne, Heart's Delight, Winterton and Hant's Harbour. Consequently, hydrotechnical investigations were conducted with the following goals:

- to obtain background information on local historical flooding to document and verify existing sources;
- to determine the influence of physiographic and cultural influences on flooding;
- analyze the influence of ice jams; and,
- confirm the accuracy of existing 1:2500 mapping.

The field program generally consisted of:

- surveys of channel cross sections and permanent hydraulic structures;
- installation of crest gauges and the subsequent collection of water level measurements during various runoff events during the course of the study to establish stage versus discharge relationships;
- interviews with residents knowledgeable of the historical flooding problems to obtain a better understanding of the causes and severity.

3.2 Cross Sections

Cross section locations were determined through a field reconnaissance program and review of existing 1:2500 mapping.

The field survey was undertaken utilizing a Mikon C-100 Total Station complete with a field computer and data collector. All study reaches were referenced to horizontal and vertical control monumentation according to the "Hydrologic and Hydraulic Procedures for Flood plain Delineation"

The distances of the cross sections along each stream were measured in meters along the channel centre lines starting with chainage of 0+000 from the outlets of the ocean, except for Whitbourne for which cross sections were measured from Bethunes Pond, Salmon Cove from Salmon Cove Pond and Victoria for which the first cross section was assigned a chainage of 100. The right edge of the brook (looking downstream) at normal low flow (that shown on the maps) for each cross section was given a reference number of 1000 when entered into the model. Horizontal distances to the right of the channel centre (looking downstream) were added to the reference number, while distances to the left were subtracted. The channel lengths between cross sections were surveyed along the channel centre line. The left and right overbank flow lengths were determined using the 1:2500 topographic mapping and these lengths represent the anticipated path of the centre of mass of the overbank flow; refer to Figure 3.1 to 3.9 for cross section location plans. Note that Figure 3.1 - Cross Section Location Plan (Carbonear) is in Appendix I of the Technical Appendices.

FIGURE 3.9 – CROSS SECTION LOCATION MAP (HANT'S HARBOUR)

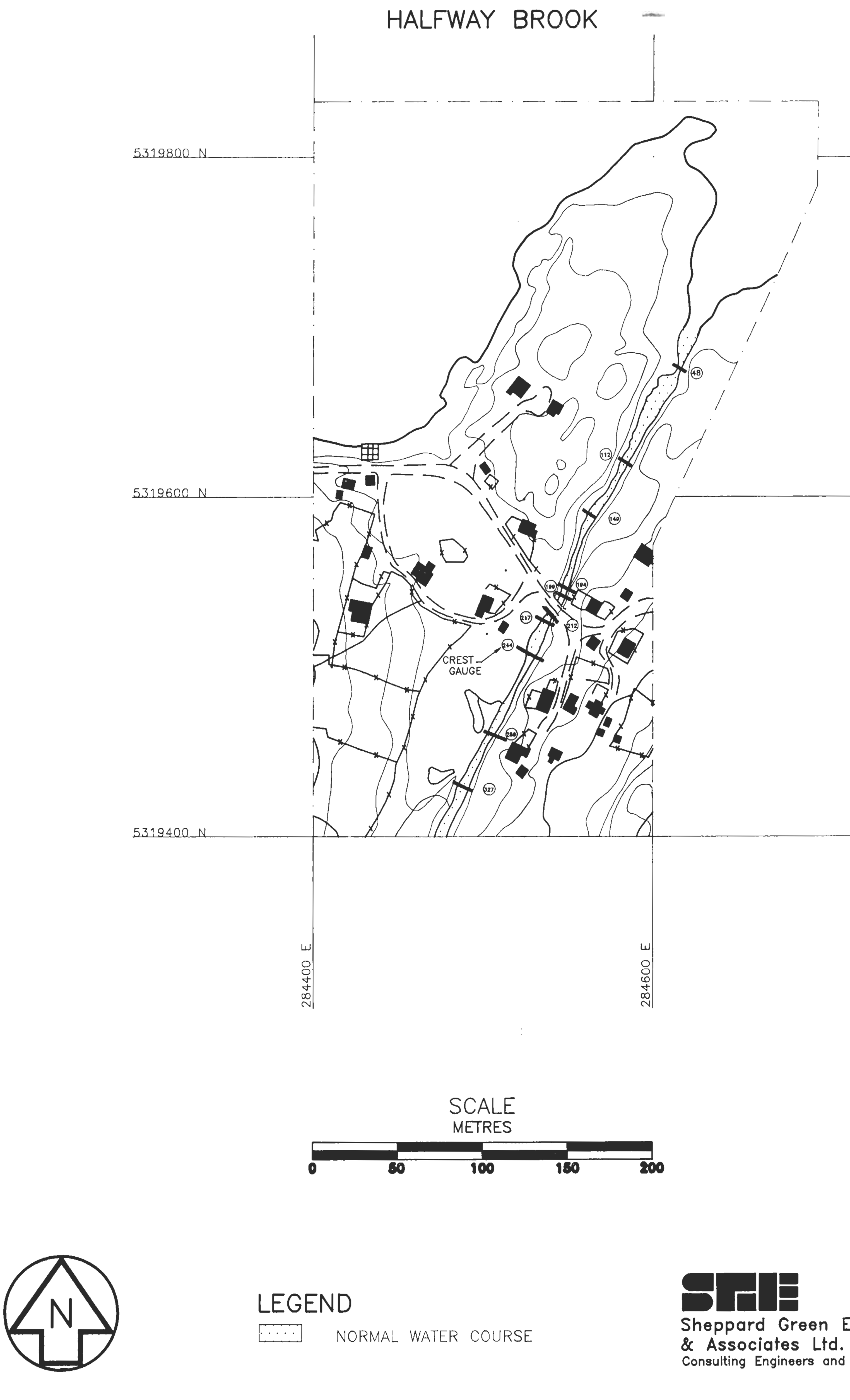


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FIGURE 3.7 – CROSS SECTION LOCATION PLAN (WINTERTON)

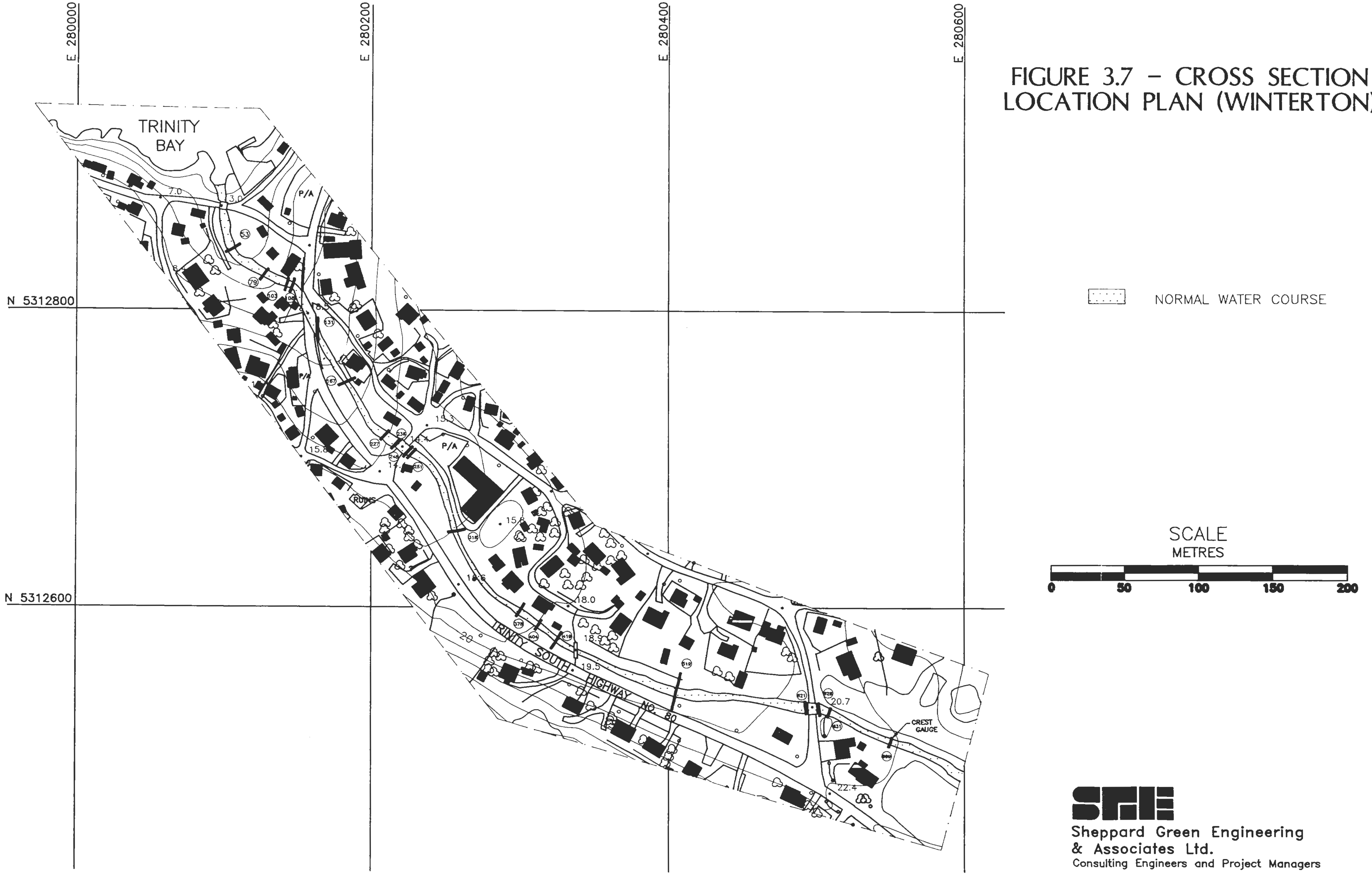

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FIGURE 3.6 – CROSS SECTION LOCATION PLAN (HEART'S DELIGHT)

BROOK #2

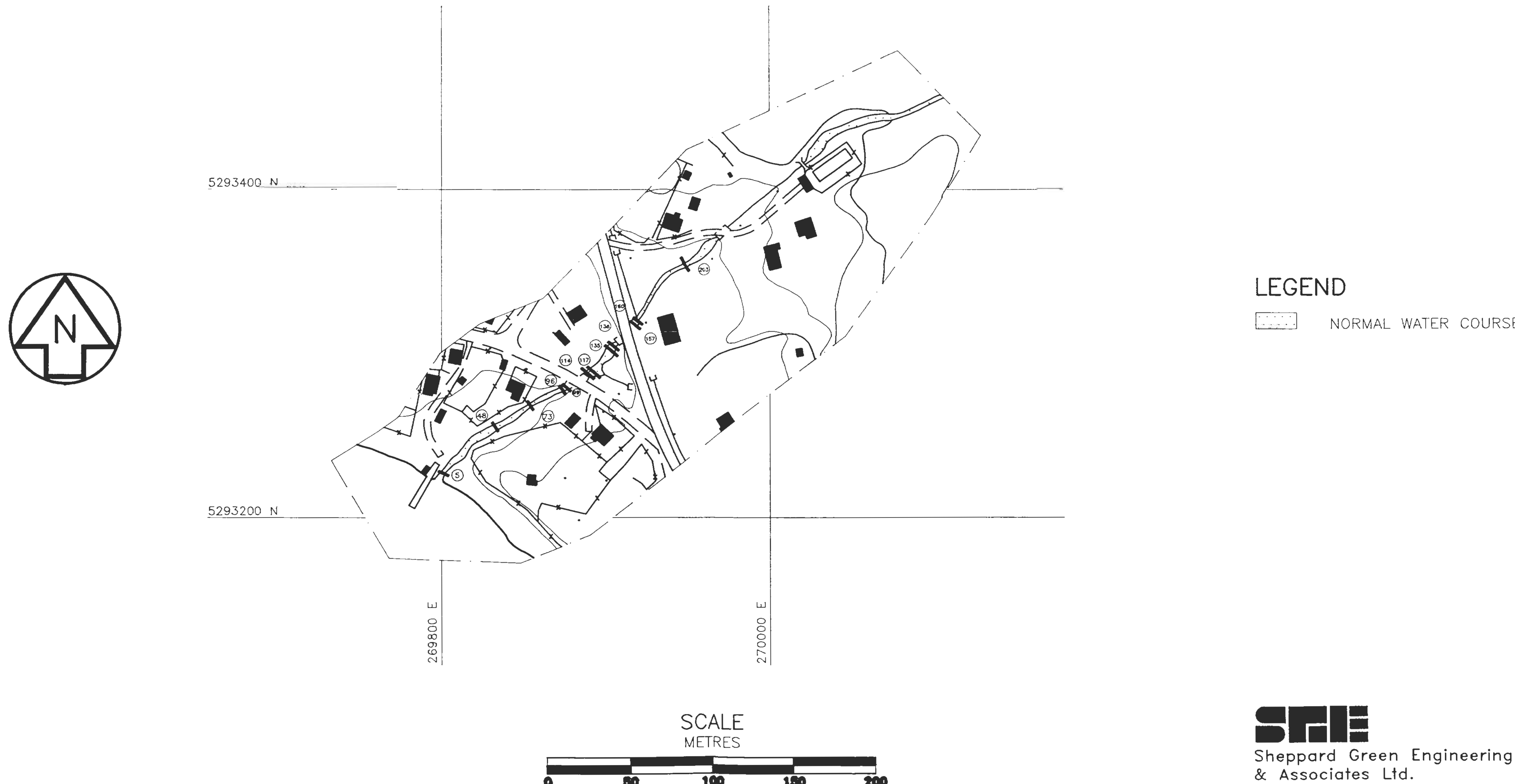

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FIGURE 3.5 – CROSS SECTION LOCATION PLAN (HEART'S DELIGHT)

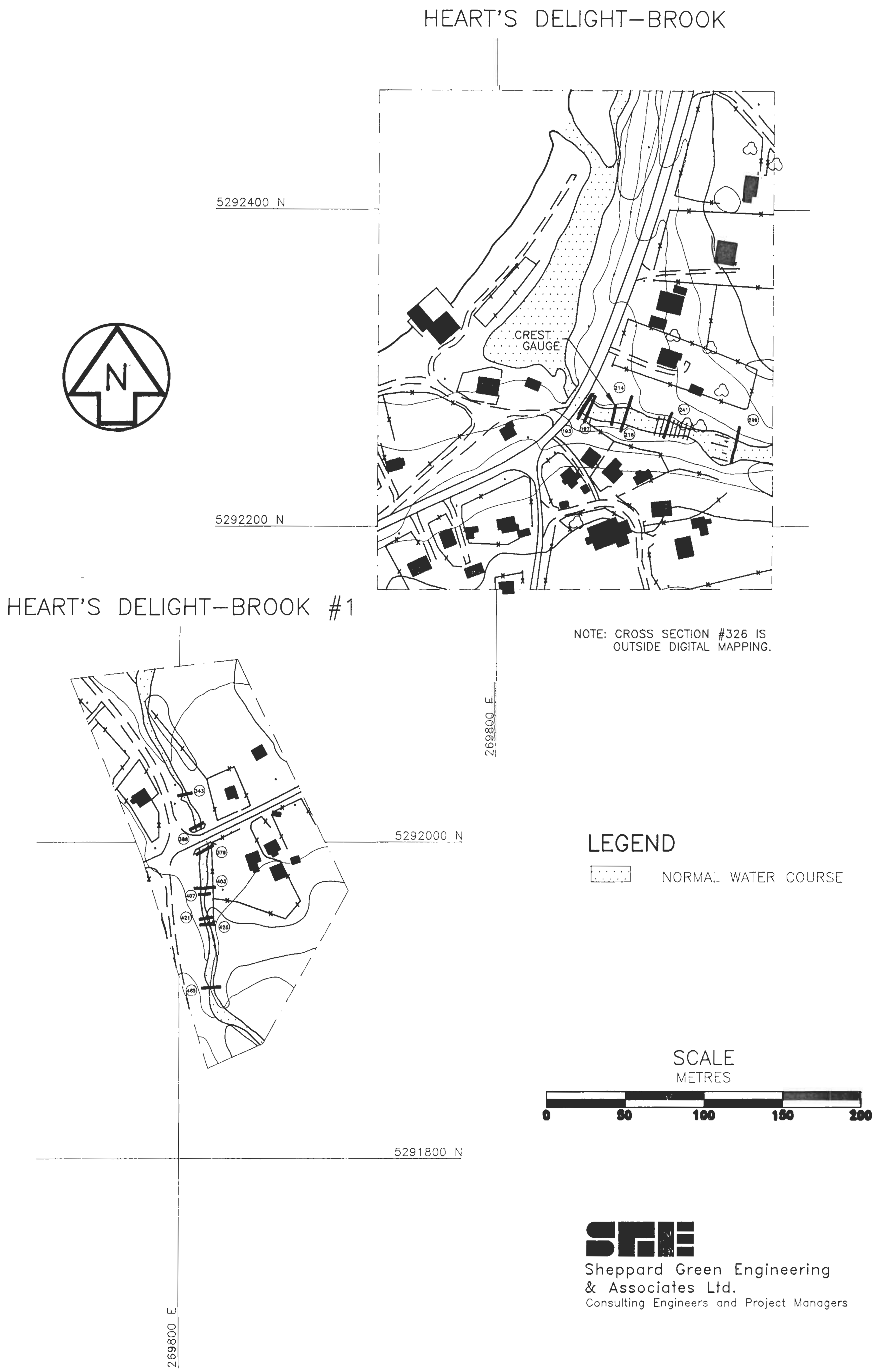

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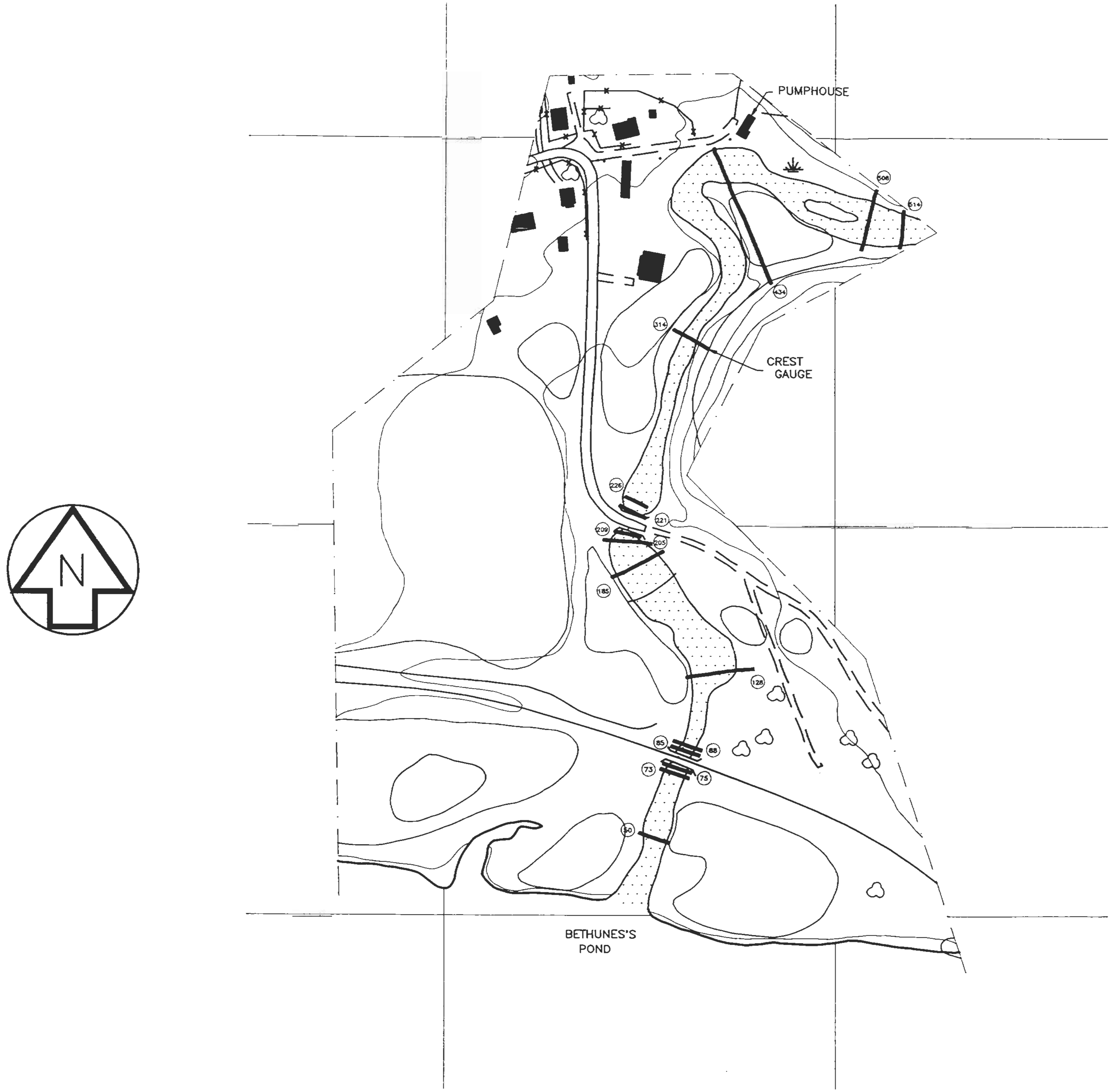
FIGURE 3.4
CROSS SECTION LOCATION MAP (WHITBOURNE)



LEGEND

NORMAL WATER COURSE

SCALE
METRES

0 50 100 150 200

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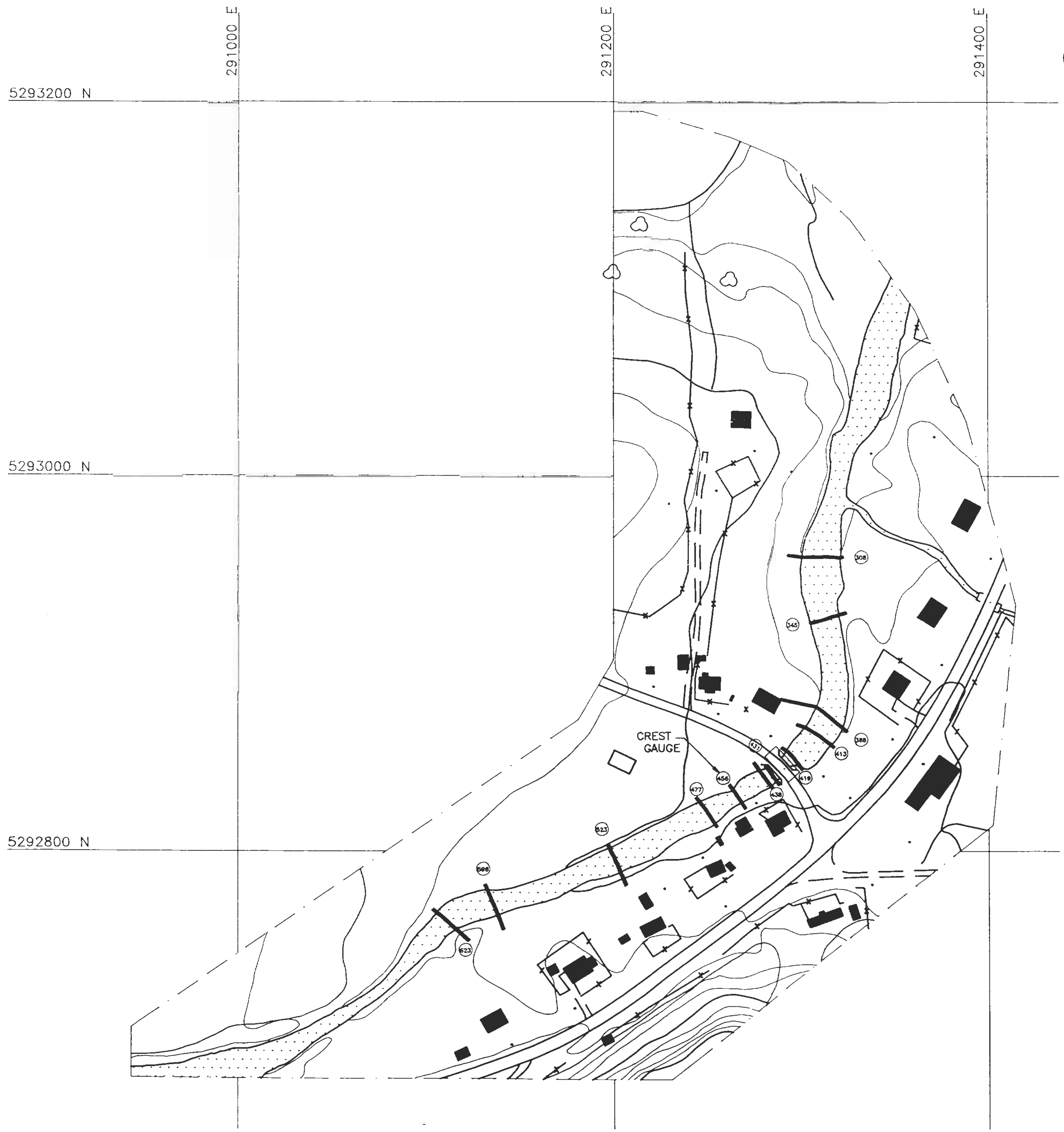
FIGURE 3.3
CROSS SECTION LOCATION MAP
(SALMON COVE)



NORMAL WATER COURSE

SCALE
METRES
0 50 100 150 200

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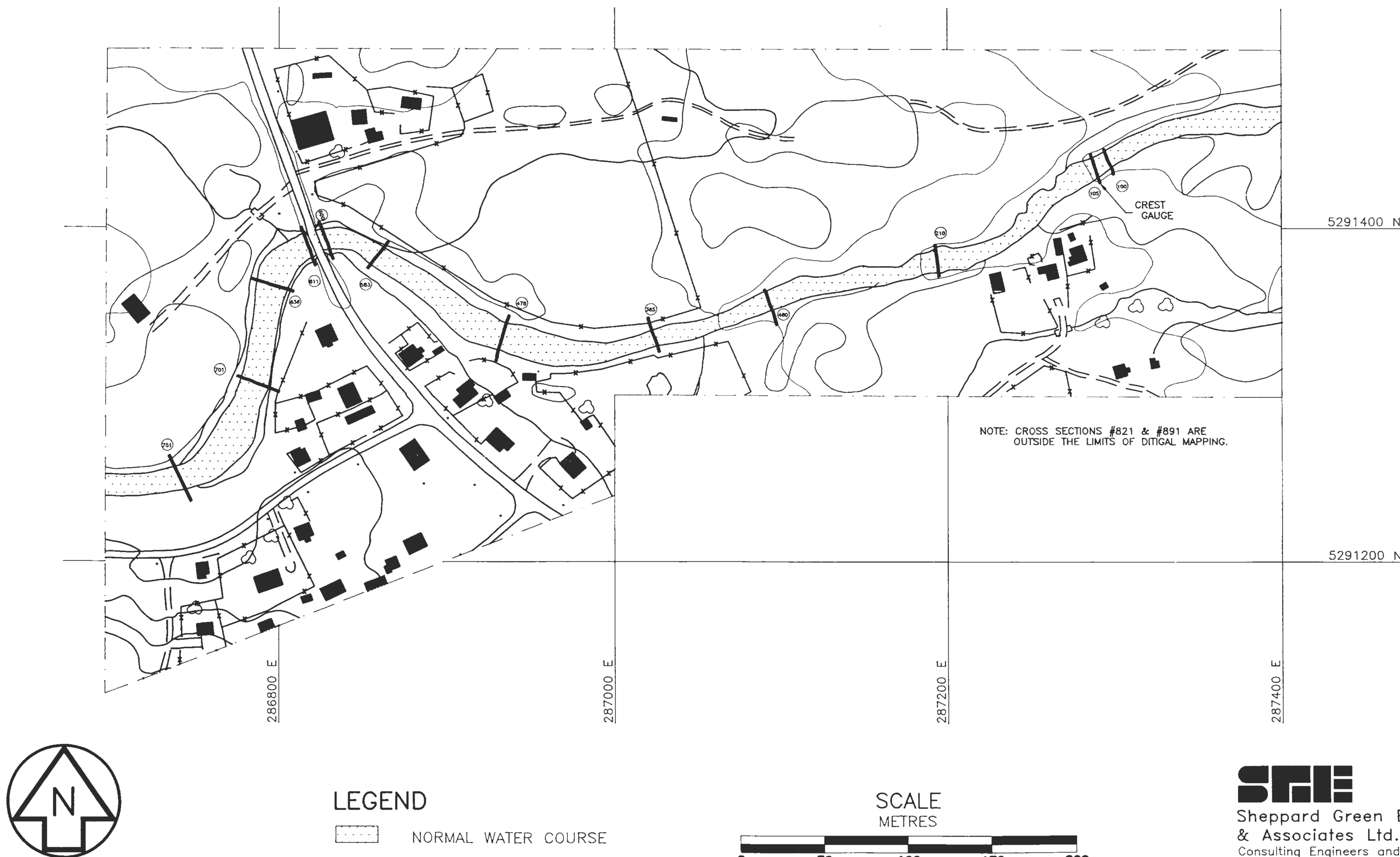
FIGURE 3.2 – CROSS SECTION LOCATION PLAN (VICTORIA)



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Carbonear (Powell's Brook)

A total of twenty nine (29) cross sections were collected along Powell's Brook. The information was collected from the concrete bridge on Lower Southside Road to upstream of the culvert installation adjacent to Penny's Transport. The cross sections consisted of twenty three channel cross sections and six (6) sections immediately upstream of the hydraulic structures in the study area.

The cross sectional information for Powell's Brook was referenced to CLM 81G2033 (N: 5287982.018, E: 287809.873, Elevation: 8.316).

Carbonear (Island Pond Brook)

A total of sixteen (16) cross sections were collected along Island Pond Brook from the bridge at the intersection of Pond Side Road and Cross Road to approximately 350 metres upstream of the triple culvert installation on the Conception Bay North Highway. The information was referenced to CLM 81G2027 (N: 5287813.301, E: 286161.047, Elevation: 47.836) and CLM 81G2031 (N: 5288215.479, E: 287308.699, Elevation: 3.039).

Victoria

A total of eight (8) cross sections were collected along Salmon Cove River in Victoria. The cross sections extended from 110 metres upstream of the concrete bridge on the Conception Bay North Highway to approximately 500 metres downstream of the bridge. The information was referenced to CLM 81G2274 (N: 5291617.336, E: 286895.970, Elevation: 51.466).

Salmon Cove

A total of seven (7) cross sections were collected along the Salmon Cove River in Salmon Cove. The sections consisted of six (6) channel cross sections and one (1) section immediately upstream of the concrete bridge which provides access to Riverdale Crescent. The cross sectional information that was collected downstream of the bridge was necessary to study the flooding which has occurred downstream of Riverdale Crescent since the bridge was replaced in 1992.

With respect to the flooding problems on Slades Avenue and Birch Cliff Drive, it became apparent during the field program that the flooding problems at these locations are caused by a lack of ditching and blocked culverts. Consequently, the study team did not collect any cross sectional information at these locations but rather collected information on the culverts, i.e. size and type to make recommendations with respect to ditching and culverts replacement.

The cross sectional information in Salmon Cove was referenced to CLM2291 (N: 5293181.855, E: 291590.945, Elevation: 24.456).

Whitbourne

A total of eight (8) cross sections were collected on the Hodges River from the train trestle to approximately seventy-five (75) metres upstream of the municipal pumphouse. The sections consisted of six (6) channel cross sections and two (2) sections immediately upstream of the hydraulic structures, i.e. train trestle and road bridge.

The survey information was referenced to CLM 82374 (N: 5253762.925, E: 265091.91, Elevation: 83.604).

Heart's Delight

There were three different study reaches in Heart's Delight that were surveyed for cross sectional information. The three study reaches are Brook #1, (originating from water supply), Heart's Delight Brook, and Brook #2 (east end of Heart's Delight).

With respect to Heart's Delight Brook, a total of five (5) cross sections were collected. They consisted of four (4) channel cross sections and one (1) section immediately upstream of the concrete bridge on route 80.

With respect to Brook #1, two (2) cross sections were collected; one at the culvert installation on Church Road and one (1) approximately thirty (30) metres upstream.

With respect to Brook #2, four sections were collected. The sections consisted of two (2) channel cross sections and one (1) section at each of the two culvert installations.

The cross sectional information was referenced to CLM 85G5538 (N: 5292443.668, E: 270085.043, Elevation: 7.006) and CLM 85G5534 (N: 5293638.855, E: 269964.898, Elev: 8.504).

Winterton

A total of ten (10) cross sections were collected along Western Pond Brook. The sections extended from the outlet of Western Pond Brook to the crest gauge located approximately 650 metres upstream

The sections consisted of seven (7) channel cross sections and one (1) section immediately upstream of each concrete bridge in the study area.

The cross sections were referenced to CLM 85G5509 (N: 5312822.749, E: 280275, Elev: 27.385).

Hant's Harbour

During the field program, it became apparent that the main flooding area in Hant's Harbour was on 'Shorts Brook' rather than Halfway Brook. Consequently, the DOE was informed of this information and the decision was made to proceed with a flood risk mapping program for Short's Brook in addition to Halfway Brook.

With respect to Halfway Brook, a total of four (4) cross sections were collected. They consisted of three (3) channel cross sections and one (1) immediately upstream of the concrete bridge. The sections extend from the outlet of the brook to 150 metres upstream of the concrete bridge.

With respect to Short's Brook, a total of six (6) sections were collected. The sections consisted of four (4) channel cross sections, one (1) immediately upstream of the triple culvert installation and one (1) immediately upstream of the concrete bridge on the main road.

The information was referenced on CLM 630350 (N: 5319387.399, E: 285126.139, Elev: 30.706).

3.3 Hydraulic Structure Surveys

Hydraulic structure surveys included a definition of structure type and sizes. The information was referenced to horizontal and vertical geodetic control monumentation. The following information was compiled on Structural Data Sheets in Appendix C of the Technical Appendices.

With respect to bridges, the following information was collected:

- clear span opening between piers;
- height of bridge deck above channel bottom;
- top road elevation;

- low chord elevation; and,
- depth of flow at centre of span or channel.

With respect to culverts, the following information was collected:

- number of culverts;
- culvert size;
- culvert type;
- invert elevation; and,
- elevation of road above culvert.

3.4 Crest Gauge Installations

Seven (7) crest gauge installations were required to obtain the necessary water level measurements throughout the study area; refer to Figures 3.1 to 3.9 for crest gauge locations.

The gauges were located in the main stream channels where structural or natural features did not cause backwater effects. The gauges consisted of 50 mm diameter ABS pipe complete with longitudinal 'slots' to permit water infiltration and measurement of water depth. A fluorescent cork float was inserted in each piece of ABS pipe to serve as an indicator of water depth and fluorescent cork grindings were placed in the ABS pipe to record the maximum water levels resulting from significant runoff events.

Base elevations of each crest gauge, i.e. river bottom elevation, was referenced to geodetic datum and level circuits for the cross sections. Detailed information related to each crest gauge is included in Appendix "D" of the Technical Appendices.

3.5 Stage-Discharge Curves

Velocity-Discharge measurements were carried out in conjunction with the crest gauge measurements to determine the stage versus discharge relationships throughout the study area. This information was used to form the initial set of data required to calibrate and verify the mathematical model. Refer to Appendix "E" of the Technical Appendices.

3.6 Summary

This information will enable the study team to plot the areal extent of historical flood levels on 1:2500 mapping for the various communities, proceed with mathematical model calibration and verification, compile a database of information that can be used in conjunction with previously documented information, and compile a list of potential remedial measures.

4.0 HYDROLOGIC ANALYSIS

4.1 Introduction

The 1:20 and 1:100 year recurrence interval peak flows were estimated using the Regional Flood Frequency Analysis regression equations and using the instantaneous unit hydrograph model OTTHYMO (University of Ottawa Hydrologic Model). In addition, the regional flood flow estimates from other sites and flows from statistical analyses from other sites were used to determine average unit flow rates and were applied to each watershed. The results of these analyses were compared and used to develop the final peak flow estimates.

4.2 Regional Flood Frequency Analysis

Regression equations developed under the Canada-Newfoundland Flood Damage Reduction Program (Environment Canada and Newfoundland Environment, 1984) and obtained from the Regional Flood Frequency Analysis for the Island of Newfoundland Users' Guide (March, 1986) were utilized in this study to derive estimates of the 1:20 and 1:100 year peak instantaneous flood flows for each of the study areas.

The regression equations developed in the above noted study are based on a single station instantaneous flood frequency analysis at 11 hydrometric stations with at least 10 years of record located on the Island of Newfoundland.

TABLE 4.1 - STEPWISE REGRESSION COEFFICIENTS FOR Qp(20)

EQUATION	K	a	b	c	d	SE		MUTIPLE-R
South Region								
Full	-3.7581	0.8499	2.3604	-1.5248	-1.6244	19.3	-16.2	0.9962
One Step Back	-1.0453	0.7349	1.5514	-1.7142	-	31.4	-23.9	0.9892
Two Steps Back	-6.6899	0.7906	2.262	-	-	54.5	-36.2	0.9684

TABLE 4.2 STEPWISE REGRESSION COEFFICIENTS FOR Qp(100)

EQUATION	K	a	b	c	d	SE		MUTIPLE-R
South Region								
Full	-3.915	0.8141	2.409	-1.6223	-1.4277	24.2	-19.5	0.9941
One Step Back	-1.2072	0.713	1.698	-1.7888	-	32.1	-24.3	0.9885
Two Step Back	-7.0975	0.7712	2.4395	-	-	56.8	-36.2	0.9653

TABLE 4.3 APPLICABLE RANGES FOR REGIONAL REGRESSION EQUATION PARAMETERS

PARAMETER	RANGES
DA (km2) Drainage Area	3.9 to 4400
MAR (mm) Mean Annual Runoff	788 to 2124
ACLS (%) Area Controlled by Lakes and Swamps	55 to 100
SHAPE = (0.28 x basin perimeter) / sqrt(DA)	1.24 to 2.45

NOTES:

1. Full equation is $\log_{10}Qpt = K + a \log_{10} DA + b \log_{10} MAR + c \log_{10} ACLS + d \log_{10} SHAPE$
2. One step back equation is $\log_{10} Qpt = K + a \log_{10} DA + b \log_{10} MAR + c \log_{10} ACLS$
3. Two step back equation is $\log_{10} Qpt = K + a \log_{10} MAR + b \log_{10} MAR$
4. SE = Standard Error. The higher the value, the poorrer the accuracy of prediction
5. Mutiple-R = Mutiple correlation coefficients. Values closer to one indicate greater accuracy.

The regression equations are developed in the following form:

$$\log_{10} QP_T = K + a \log_{10} DA + b \log_{10} MAR + c \log_{10} ACLS + d \log_{10} SHAPE \qquad \text{Equation (4.1)}$$

where:

QP_T	= T year maximum instantaneous peak flow;
K,a,b,c,d	= constants (refer to Tables 4.1 to 4.3);
DA	= Drainage Area (sq km);
SHAPE	= (0.28 x basin perimeter)sqroot(DA) (1/km);
MAR	= Mean Annual Runoff (mm) over the area from the Mean Annual Runoff map developed during the Regional Analysis; and,
ACLS	= area controlled by lake and swamp (% of drainage area) from 1:50,000 NTS maps using criteria that lake or swamp with surface areas at least 1% of the drainage area to the lake or swamp outlet controls the area to the outlet.

The values of the coefficients of the equation depends on the return period of interest and the location of the study area. Equations were developed for the north and south regions in Newfoundland and for the entire Island. The equation for the entire Island should be used if the area under study is close to the border between the north and south regions. The South Equation was used for this study since all watersheds are well within the South Region.

Table 4.4 provides the values for each of the required inputs to the equations for each of the 11 study areas. Table 4.5 summarizes the 1:20 and 1:100 year instantaneous peak flow estimates for each study area using the set of coefficients developed for the South Region. Some of the parameter values fell outside of the applicable range for the full equations. Under these conditions, equations developed for Regional Frequency Analysis excluding the third, fourth and fifth sum in equation 4.1 along with corresponding coefficients, were applied.

In addition to these equations, updated regression equations developed in a similar manner and obtained from the Regional Flood Frequency Analysis for the Island of Newfoundland Users' Guide (August, 1990) were used to derive flood flow estimates. The regression equation for the Avalon and Burin Peninsulas is of the following form:

$$\mathbf{\log_{10} QP_T = C + a1 \log_{10} DA + a2 \log_{10} LSF} \quad \text{Equation (4.2)}$$

where:

QP_T	= T year maximum instantaneous peak flow;
C,a1,a2	= constants (refer to Table 4.6);
DA	= Drainage Area (km^2);
LSF	= Lakes and Swamps Factor =(1+FACLS)FLSAR/(1+FACLS);
FACLS	= Fraction of Drainage Area Controlled by Lakes and Swamps; and,
FLSAS	= Fraction of Drainage Area Covered by Lakes and Swamps.

Table 4.6 provides the values for each of the required inputs to equation 4.2 for each of the eleven drainage areas. Several of the parameter values fell outside of the applicable range for the equation as noted in Table 4.6.

4.2.1 Regional Flood Flow Estimates from Other Sites

The average of the unit 1:20 and 1:100 year regional flood flow estimates for three nearby stations were calculated and applied to the study areas. The following stations were selected due to hydrologic similarity:

- 02ZH002 Come by Chance River near Goobies;
- 02ZM006 Northeast Pond River at Northeast Pond; and,
- 02ZN001 Northwest Brook at Northwest Pond.

TABLE 4.4
PARAMETER VALUES FOR THE REGIONAL REGRESSION EQUATIONS

Location	Description	Drainage Area (DA) (sq. km)	Mean Annual Runoff (MAR) (mm)	Area Controlled by Lakes & Swamps (ACLS) (%)		Drainage Area Perimeter (P) (km)	Shape Factor	
Carbonear	Powell's Bk. to outlet	8.0	1000	55		13.0	1.29	
	Island Pond Bk. to outlet	37.8	1000	74		23.0	1.05	●
Hant's Harbour	Shorts Bk. to outlet	7.9	1000	20	●	11.7	1.17	●
	Halfway Bk. to outlet	19.1	1000	87		23.0	1.47	
Heart's Delight	Heart's Delight Bk. to outlet	44.1	1000	98		34.7	1.46	
	(south) brook to outlet	5.5	1000	80		10.0	1.19	●
	(north) brook to outlet	2.0	1000	5	●	6.5	1.29	
Salmon Cove	Salmon Cove R. to study area	52.4	1000	59		48.0	1.86	
Victoria	Salmon Cove R. to study area	40.8	1000	57		40.3	1.77	
Whitbourne	Hodges R. to Bethunes Pond	48.4	1300	99		43.3	1.74	
Winterton	Western Pond Bk. to outlet	13.0	1000	86		19.6	1.52	

Notes:
1. ●Indicates parameter value is outside of applicable range for full equations
2. Shape Factor = 0.28 P/sqroot DA

TABLE 4.5
ESTIMATED PEAK DISCHARGES AND 95% CONFIDENCE LIMITS USING THE REGIONAL REGRESSION EQUATIONS(1986) FOR THE ISLAND OF NEWFOUNDLAND

Location	Description	Coefficients used Qp(100)	Qp(100)	95% Confidence Limits (cms)		Qp (20)	95% Confidence Limits (cms)	
				South Region Equation				
Carbonear	Powell's Brook to outlet	full equation	24.6	36.3	15.2	18.2	25.0	12.4
Hant's Harbour	Halfway Brook to outlet	full equation	19.6	28.8	12.1	15.2	20.9	10.4
Heart's Delight	Heart's Delight Bk to outlet	full equation	32.2	47.5	19.9	26.1	36.0	17.8
Salmon Cove	Salmon Cove R to study area	full equation	60.1	88.6	37.1	44.4	61.2	30.3
Victoria	Salmon Cove R to study area	full equation	55.6	82.0	34.3	41.0	56.5	28.0
Whitbourne	Hodges River to Bethunes Pond	full equation	50.1	73.9	31.0	38.8	53.5	26.5
Winterton	Western Pond Bk to outlet	full equation	13.9	20.5	8.6	10.6	14.6	7.2
				South Region Equation				
Carbonear	Island Pond Bk to outlet	one step back	46.5	75.8	24.4	36.6	59.2	19.5
Hant's Harbour	Shorts Bk to outlet	two steps back	8.2	17.3	2.4	6.4	13.2	2.0
Heart's Delight	(north) brook to outlet	two steps back	2.8	6.0	0.8	2.2	4.5	0.7
Heart's Delight	(south) brook to outlet	one step back	10.2	16.7	5.4	7.8	12.6	4.1

NOTES:
1. Full equation is log 10 Qpt = K + a log 10 DA + b log 10 MAR + c log ACLS + d log 10 SHAPE
2. One step back equation is log 10 Qpt = K + a log 10 DA + b log 10 MAR + c log 10 ACLS
3. Two step back equation is log 10 Qpt = K + a log 10 DA + b log 10 MAR
4. The one step back equation was used for Island Pond Brook in Carbonear and (south) brook in Heart's Delight because their SHAPE parameters were outside of the applicable range.
5. The two step back equation was used for Shorts Brook in Hant's Harbour and (north) brook in Heart's Delight because their SHAPE and/or ACLS parameter values were outside of the applicable range.

TABLE 4.6
PARAMETER VALUES FOR THE REGIONAL EQUATIONS (1990 EQUATIONS)

Location	Description	Draiange Area (DA) (sq. km)		Area Covered by Lakes & Swamps (FLCLS) (fraction)		Area Covered by Lakes & Swamps (FLSAR) (fraction)		Lakes & Swamps Factor	Qp(100) (m3/s)	Qp(20) (m3/s)
Carbonear	Powells Bk.	8		0.55		0.04	*	1.52	29.9	22.4
	Island Pond Bk.	37.8		0.74		0.12		1.67	65.2	50.7
Hant's Harbour	Shorts Bk.	7.9		0.2	*	0.06	*	1.15	70.1	47.8
	Halfway Bk.	19.1		0.87		0.12		1.81	32.3	25.7
Hearts Delight	Heart's Delight Bk.	44.1		0.98		0.15		1.90	48.6	39.5
Brook #1	(south) Brook	5.5		0.8		0.1		1.74	15.3	12.0
Brook #2	(north) Brook	2	*	0.05	*	0.04	*	1.01	40.5	26.3
Salmon Cove	Salmon Cove R.	52.4		0.59		0.12		1.51	110.1	83.0
Victoria	Salmon Cove R.	40.8		0.57		0.1		1.51	94.4	70.9
Whitbourne	Hodges R.	48.4		0.99		0.3		1.84	57.6	46.3
Winterton	Western Pond Bk.	13		0.86		0.15		1.78	25.9	20.5

Notes:

1. Parameterlimits are as follws:
   DA 2.9−285 km2
   FLSAR 0.09−0.30
   FACLS 0.55−1.0
2. The equation takes the following form: Log10(Qt)=C+a1*Log10(DA)+a2Log10(LSF)
3. Constants for the equation are:
   Qp(100) C=1.41701, a1=0.684, a2=3.057
   Qp(20) C=1.22701, a1=0.688, a2=2.723
4. * Denotes parameter out of range
5. Lakes and Swamps Factor = (1+FACLS)−FLSAR/(1+FACLS)

TABLE 4.7
SUMMARY OF DATA FOR HYDROMETRIC STATION STATISTICAL ANALYSES

Station Name and Number	DA (sq km)	MAR (mm)	ACLS (%)	Shape Factor
Come By Chance River near Goobies (02ZH002)	43.3	1364	92	1.66
Northeast Pond River at Northeast Pond (02ZM006)	3.9	1100	100	1.24
Northwest Brook at Northwest Pond (02ZN001)	53.3	1847	100	2.06
Station Name and Number	South Region Regression Analysis		Single Station Frequency Analysis	
	QP (100) (m^3/s)	QP (20) (m^3/s)	QP (100) (m^3/s)	QP (20) (m^3/s)
Come By Chance River near Goobies (02ZH002)	62.0	47.9	75.3	53.2
Northeast Pond River at Northeast Pond (02ZM006)	6.9	5.3	7.1	5.4
Northwest Brook at Northwest Brook Pond (02ZN001)	97.8	72.5	90.1	64.3
	Qp/DA	Qp/DA	Qp/DA	Qp/DA
Come By Chance River near Goobies (02ZH002)	1.43	1.11	1.74	1.23
Northeast Pond River at Northeast Pond (02ZM006)	1.77	1.35	1.82	1.38
Northwest Brook at Northwest Brook Pond (02ZN001)	1.84	1.36	1.69	1.21
Average Flow (cms/sq km)	1.68	1.27	1.75	1.27

NOTES:
DA = drainage area
MAR = mean annual runoff
ACLS = area controlled by lakes and swamps
Qp = instantaneous peak flow m^3/s
Shape Factor = (0.28 x perimeter of drainage area)/sqrt(DA)

FIGURE 4.2 - CARBONEAR AND VICTORIA SUBWATERSHEDS

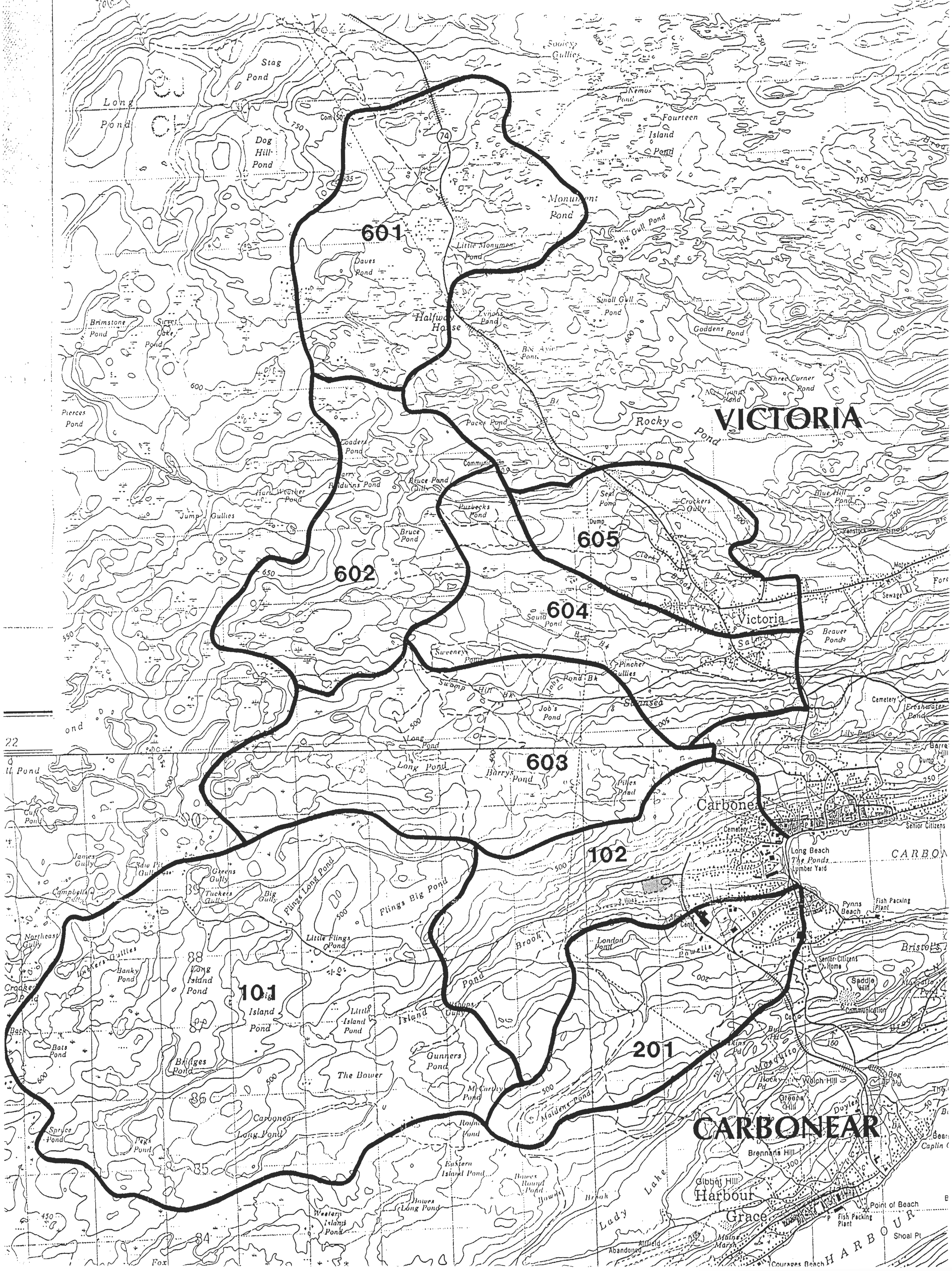


FIGURE 4.3 - SALMON COVE SUBWATERSHEDS

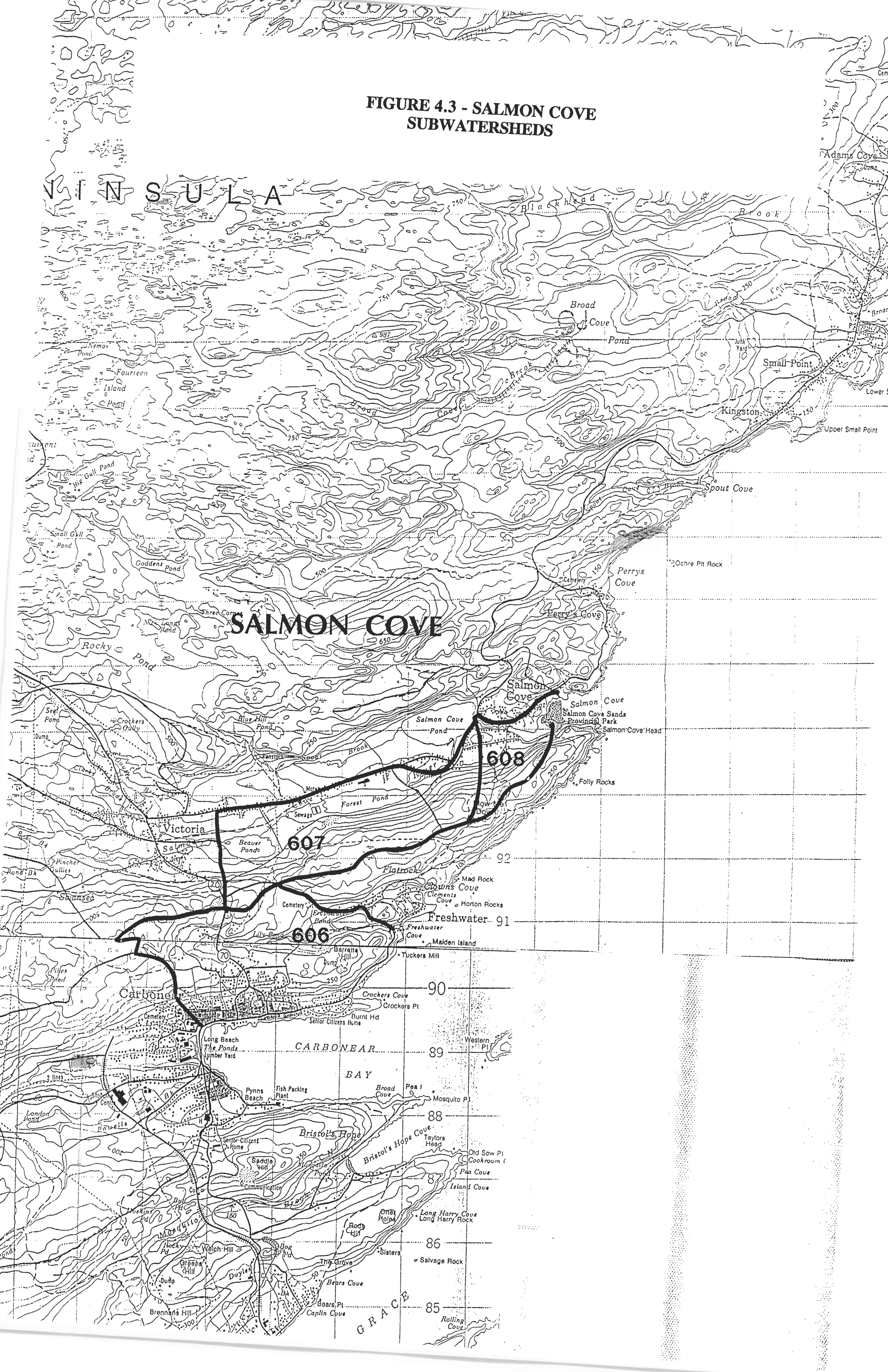


FIGURE 4.4 - WHITBOURNE SUBWATERSHEDS

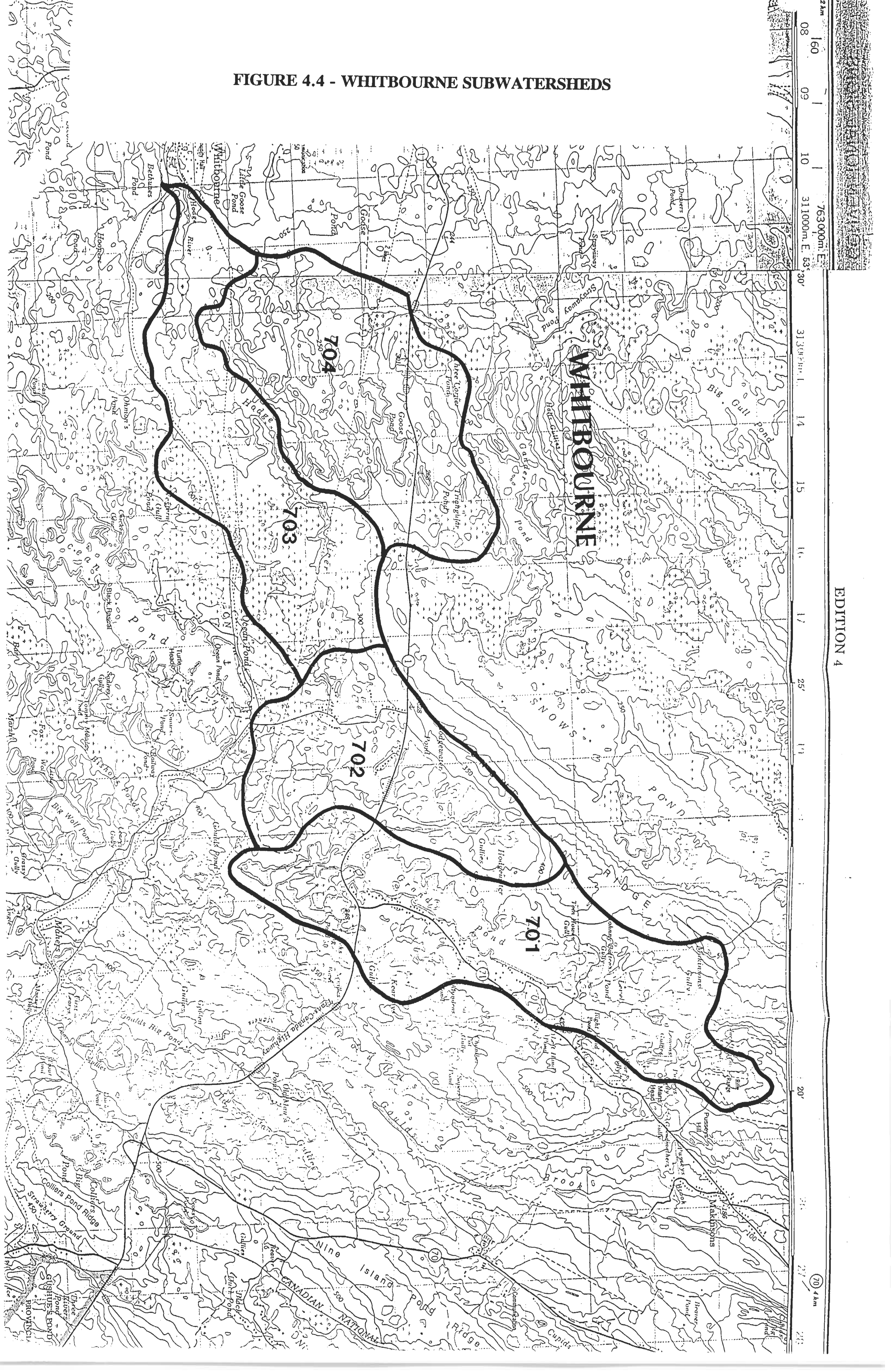


FIGURE 4.5 - HEART'S DELIGHT SUBWATERSHEDS

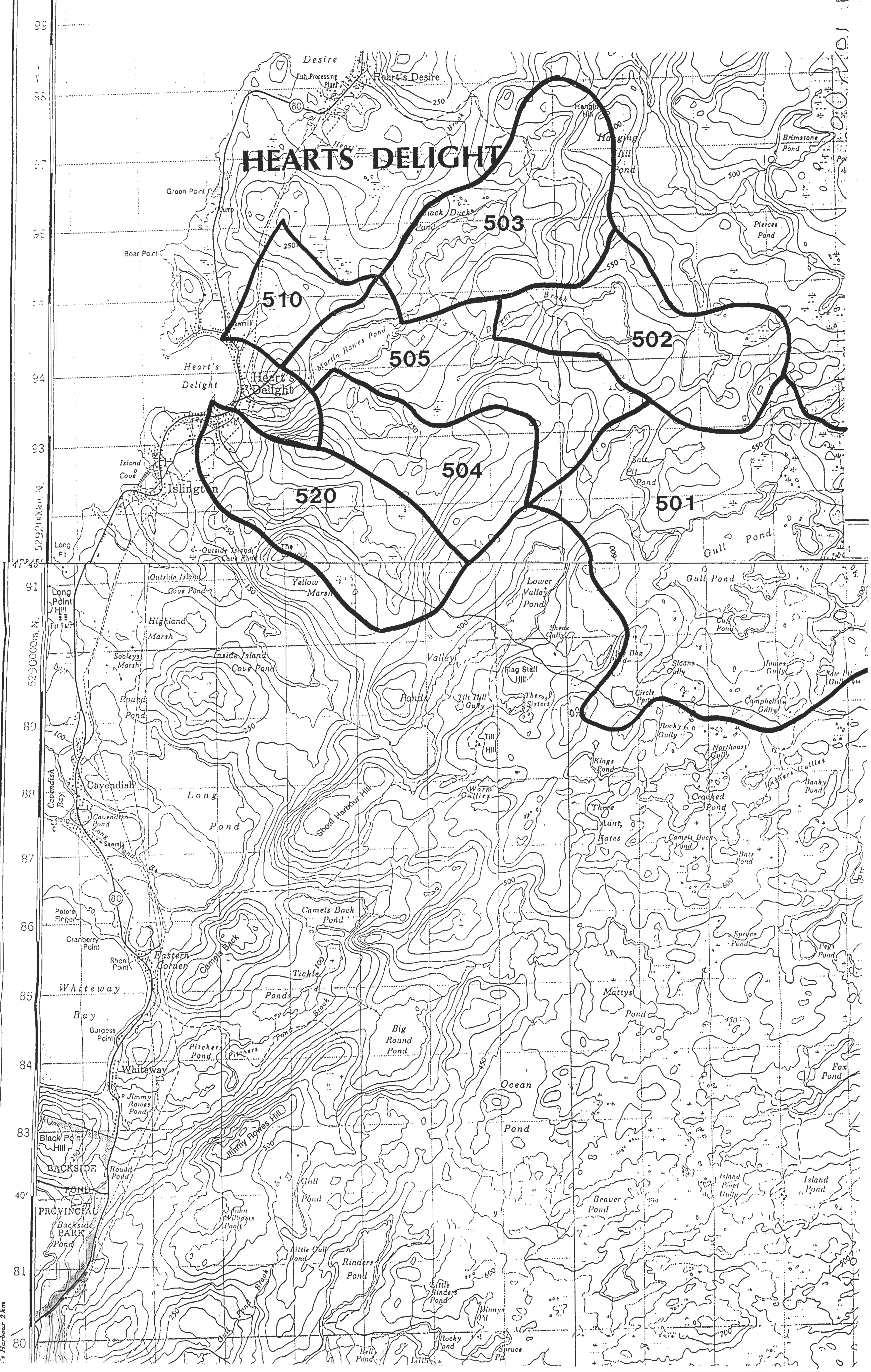
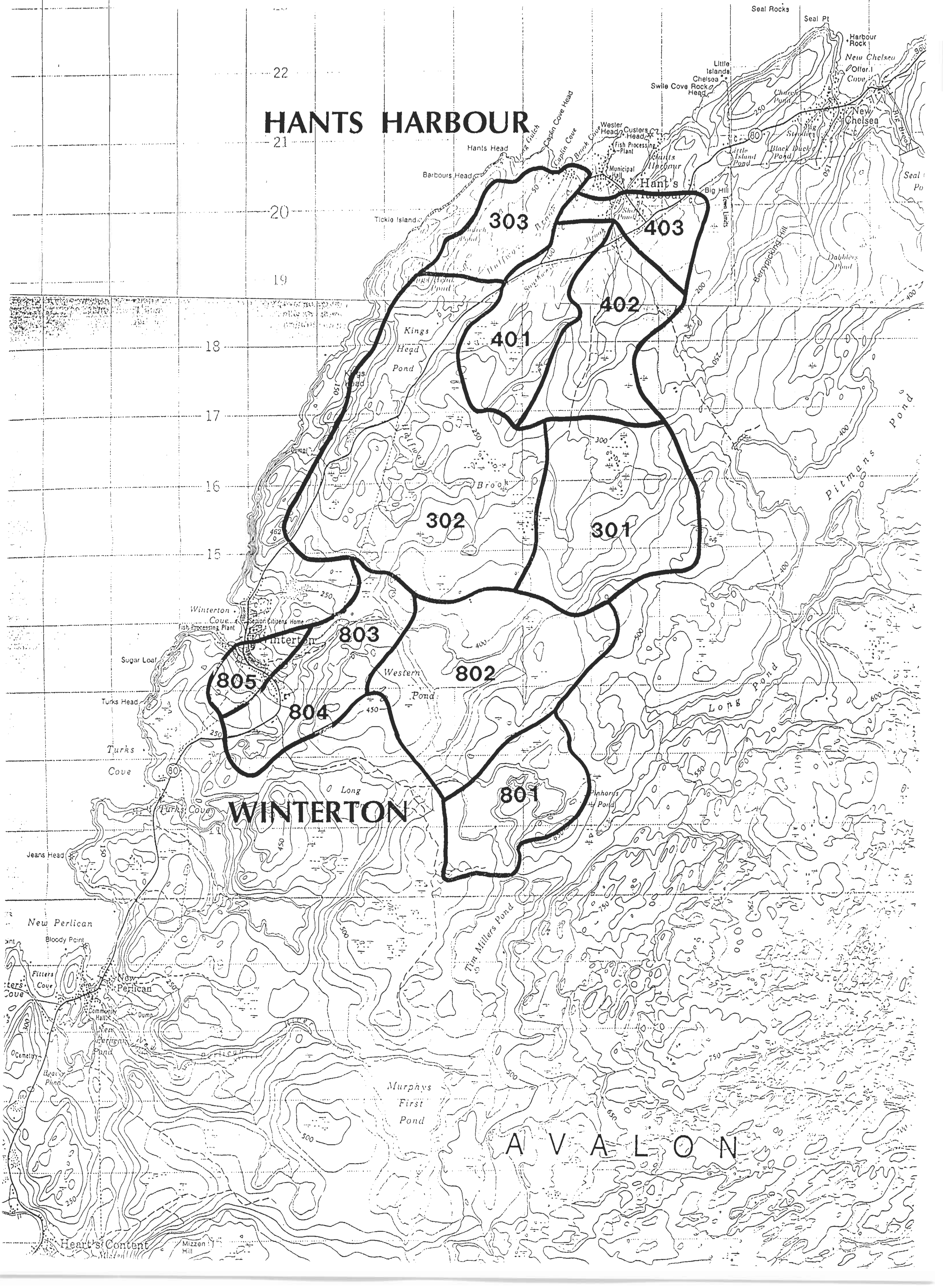


FIGURE 4.6 - WINTERTON & HANT'S HARBOUR SUBWATERSHEDS



Regional instantaneous flood flow rates for each of the three stations were taken from the Regional Flood Frequency Analysis using the equations for the south region of Newfoundland. The watershed parameters and the unit flow rates are summarized in Table 4.7. The average unit flow rates were then applied to the study area watersheds. These peak flow rates are included in Table 4.13.

4.2.2 Single Station Statistical Analysis

Peak flow rates determined by performing single station frequency analyses for the three nearby stations were also used to determine average unit flow rates for the 1:20 and 1:100 year recurrence intervals. These values were obtained from the Regional Flood Frequency Analysis Users' Guide (March, 1986). The average unit flow rates were then applied to the study area watersheds. These peak flow rates are included in Table 4.13.

4.3 OTTHYMO Modelling

The OTTHYMO (University of Ottawa Hydrologic Model) used in this study (version 2.0) is based on one of the most frequently used hydrologic models in Canada which is HYMO. HYMO is a computer package made up of a series of subroutines each of which corresponds to a specific hydrologic command (e.g. COMPUTE HYD - compute hydrograph). It was intended as a hydrologic language to perform watershed modelling. New subroutines with their corresponding hydrologic commands can be added easily to the package. The modifications made to HYMO at the University of Ottawa included a subroutine for rural flows based on the Nash model (COMPUTE NASHYD), and a methodology for use of a modified SCS CN procedure for rainfall losses.

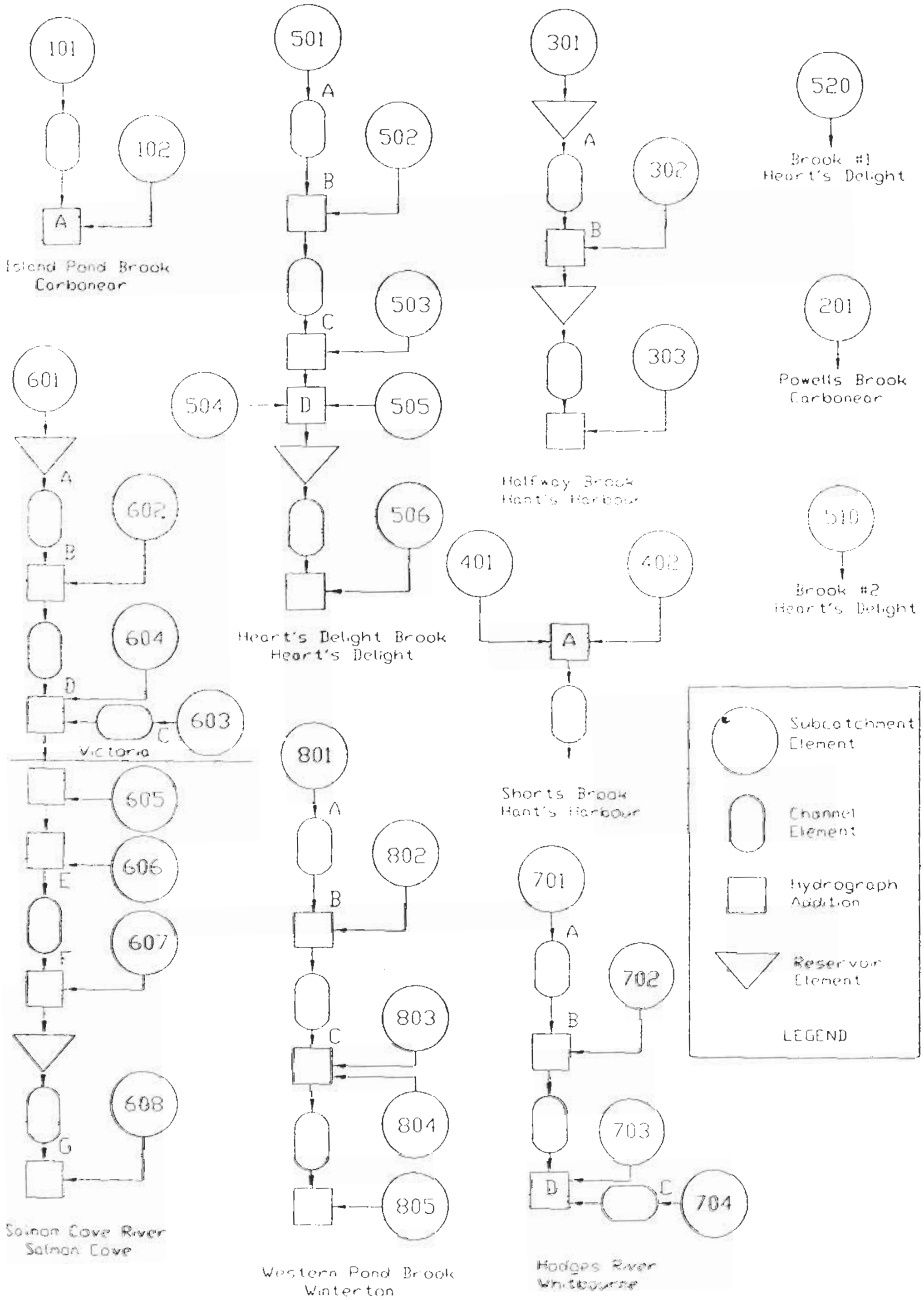
The principle in using OTTHYMO for a watershed application is the same as in the HYMO model. For a typical application, OTTHYMO simulates the hydrograph for each subwatershed, routes the hydrograph through a natural channel or a pipe and then adds this routed hydrograph to a hydrograph from another subwatershed downstream. The process starts upstream and continues down the drainage network to the outlet. There is also the capability of routing through lakes and reservoirs for stormwater management purposes.

Ten OTTHYMO models were developed for this study. Watershed boundaries were defined for the following streams and locations:

- Island Pond Brook - Carbonear;
- Powells Brook - Carbonear;
- Halfway Brook - Hant's Harbour;
- Shorts Brook - Hant's Harbour;
- Heart's Delight Brook - Heart's Delight;
- Brook # 1 - Heart's Delight;
- Brook # 2 - Heart's Delight;
- Salmon Cove River - Victoria /Salmon Cove;
- Hodges River - Whitbourne; and,
- Western Pond Brook - Winterton.

The watersheds were divided into subcatchments based on the natural drainage patterns and watershed characteristics of the terrain such as slope and soil classification; Figures 4.1 to 4.6 illustrate the flowcharts and subcatchments for each watershed. The subcatchment parameters required for the OTTHYMO models were determined with the use of 1:50 000 scale and 1:2500 scale topographic mapping along with a map showing the surficial geology of the Trinity Bay area. The OTTHYMO COMPUTE NASHYD command was chosen to be used to compute the hydrograph from each subcatchment since it was specially designed for estimating flows from rural watersheds; refer to Appendix F in the Technical Appendices.

FIGURE 4.1 - OTTHYMO FLOWCHART



The required parameters for the COMPUTE NASHYD include the subcatchment area, slope, characteristic length, runoff curve number (CN), initial abstraction, number of linear reservoirs, and time to peak (t_p). The 1:50 000 scale mapping was used to determine the area of each subcatchment. The subcatchment characteristic lengths and slopes were also determined from the 1:50 000 maps, except where more detail was required, such as small subcatchment areas containing few contour lines. These parameters were used to determine the travel time for each subcatchment based on the following equation:

$$\mathbf{tp = 4.63A^{0.422}SLP^{-0.46}(L/W)^{0.133}} \quad \text{Equation (4.3)}$$

where:

A = watershed area in sq. miles;

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

L/W = watershed length-width ratio.

This equation, developed by F. R. Williams for the original HYMO model, was chosen since it provided more reasonable results than other equations typically found in reference hydrology texts and because it has been used in other floodline mapping studies where subcatchments were predominantly rural. The calculated values of the travel time were increased to account for ponds not specifically dealt with in the models.

The surficial geology map was used to identify the hydrologic soil groups for each subcatchment which were then translated into CN numbers with the aid of tables relating land uses and hydrologic soil groups to CN values. The hydrologic soil groups consisted of:

- \- Moraine (including ground moraine and end moraine mainly comprised of well graded, stony, glacial till with silt to gravel sizes; isolated zones of stratified drift; mainly thin and discontinuous; locally, bedrock outcrops may be numerous)
- \- Glaciofluvial (including outwash deposits of gravel, sand and silt; outwash valley train and deltaic sediments; kames and kame terraces; mainly well sorted with little silt); and,
- \- Bedrock (including extensive outcropping or bedrock concealed by thin veneer of topsoil).

Initial CN values of 80, 77 and 83 were given to soil groups 1, 2 and 3, respectively (Bedient and Huber, 1992). These CN values assume normal antecedent moisture condition II. Consideration was given to the amount of forest cover and residential area when developing the CN values for each subcatchment. The CN values went as high as 88 for catchments with large residential areas and areas without forest cover. The CN values were also decreased to account for storage and attenuation of storm runoff due to ponds not specifically dealt with in the models. The amount of the reduction depended on the area of ponds in the subcatchment and values reached as low as 60.

The initial abstraction was set to 3 mm for all watersheds. This value is typical or normal based on values used in other studies in the area and it was found to be applicable for our study areas. Table 4.8 summarizes the physiographic parameters used for each subcatchment.

The parameters required for each ROUTE command in OTTHYMO consist of channel reach lengths, Manning's coefficients of friction (n) for the channel reaches and the right and left overbank areas, channel and floodplain slope and a typical cross section for the reach. The channel reach lengths were determined using the 1: 50 000 topographic mapping.

TABLE 4.8
PHYSIOGRAPHIC PARAMETERS FOR SUB-WATERSHEDS

Watershed	Basin No.	Area (sq.km)	Length (m)	Height (m)	Time to Peak (hrs.)	CN
Island Pond Brook	101	27.9	7325	46	2.76	63
	102	9.9	5325	91	1.18	75
Powell's Brook	201	8.0	6325	137	1.04	79
Halfway Brook	301	5.5	2325	30	0.90	88
	302	11.3	5350	76	1.34	88
	303	2.3	2325	37	0.65	92
Shorts Brook	401	3.0	3100	67	0.65	73
	402	3.5	2670	67	0.61	72
	403	1.4	750	9	0.45	73
Heart's Delight Brook	501	20.0	5500	15	3.37	68
	502	5.7	4350	107	0.81	71
	503	6.7	4150	76	0.96	72
	504	4.7	3325	99	0.65	71
	505	6.1	5150	119	0.89	71
	506	0.9	1325	46	0.30	80
Brook # 2 - Heart's Delight	510	2.0	1650	55	0.40	78
Brook # 1 - Heart's Delight	520	5.9	4460	82	0.94	62
Salmon Cove River	601	11.3	5250	53	1.55	83
	602	9.3	3600	61	1.05	82
	603	11.8	6425	76	1.55	83
	604	8.4	5100	61	1.31	84
	605	5.8	4025	82	0.87	88
	606	1.5	1050	38	0.31	88
	607	3.1	2400	61	0.57	88
	608	1.2	1200	69	0.25	88
Hodges River	701	15.1	8200	15	4.15	76
	702	9.7	5300	8	3.66	78
	703	11.4	8650	23	3.30	79
	704	12.2	6000	5	5.42	76
Western Pond Bk	801	2.9	1700	14	0.86	60
	802	5.9	3325	46	0.99	60
	803	2.0	2170	67	0.45	79
	804	1.3	1725	44	0.40	79
	805	0.9	1000	24	0.32	79

NOTES: 1. Time to peak is calculated by tp = 4.63A0.422SLP-0.46(L/W)0.133 for the OTTHYMO model
2. CN = SCS Curve Number based on soil type and vegetation cover for the OTTHYMO model.

The Manning's n values were determined with the use of other studies performed in the area, photographs and descriptions of the area from the field survey along with tables relating channel and overbank characteristics with Manning's n values. The final Manning's coefficients were selected comparing all of the data with the guides and typical photos for selecting Manning's coefficients given in V.T. Chow's **Open Channel Flow Handbook**. A value of 0.045, representing channel bottoms consisting of gravels, stones and boulders, was selected for all the channels since their characteristics, in general, were very similar. A value of 0.06, representing channel banks with bushes, shrubs and small trees, was used for the floodplain in each watershed, again, because the floodplain characteristics did not vary greatly.

It should be noted that the Manning's n coefficients used in the OTTHYMO models differ from those used in the HEC-2 models (Section **5.0 Hydraulic Investigations**). OTTHYMO uses average or overall values for the various reaches in the model for simplicity. The HEC-2 models were developed for specific areas along these reaches, thus requiring Manning's n values specific to these areas. In addition, flood elevations computed with HEC-2 are sensitive to the Manning's n values and are, therefore, adjusted for the purpose of model calibration. The results from OTTHYMO are not greatly sensitive to the selection of the Manning's n values.

The channel and floodplain slope were determined not to differ significantly and were thus set equal. The channel slope was calculated using the 1:50 000 topographic mapping and the 1:2500 mapping when necessary. Typical cross sections were found on and taken from the 1:2500 mapping giving consideration to the average width of the channel reaches and to the average entrenchment. The channel depths were determined assuming that the width to depth ratios of the channels in the study areas are characteristic of the entire channel. Often, the width to depth ratio of a stream is a characteristic feature in areas of the watershed with similar conditions.

The same cross-section data and coordinates were used in both the hydrologic (OTTHYMO) and hydraulic (HEC-2) analyses.

Pond discharge-storage relationships are necessary to simulate the ponds and lakes present in the study catchment areas. Eight ponds were determined to have a significant affect on the flows and, thus, required the flows to be routed through them. To develop the discharge-storage relationships, the areas of the ponds were measured with a planimeter using the 1:50 000 scale mapping. The ponds were assumed to be filled and the initial storage was assumed to be zero at the pond elevations indicated on the mapping. Incremental storage values above these pond water elevations were then calculated based on the surface area of the ponds and using a linear relationship. An increase in elevation of 0.5 m would result in an increase in storage equal to the incremental water depth (0.5m) times the surface area of the pond. The discharge rating tables were developed using the Manning's equation of flow and estimated dimensions of the outflow channels. The channel dimensions and slopes were estimated using 1:2500 mapping and the cross sections in the study areas. The Manning's friction factors of 0.045, representing channel bottoms with gravels and stones, were selected using V.T. Chow's **Open Channel Flow Handbook**.

4.4 Meteorological Data

To estimate the 1:20 year and 1:100 year rainfall volumes and distributions for generating the design flows, rainfall statistical information was obtained from the Atmospheric Environment Service. The volume data set consisted of a Rainfall Frequency Atlas for Canada which contains rainfall frequency maps for durations of 5 minutes to 24 hours. Statistics for longer durations (24 hour to 30 days) were also obtained from meteorological stations located in St. John's and New Chelsea, Nfld.

Using rainfall isoquants given in the Rainfall Frequency Atlas, rainfall volumes were found for all the watersheds for each of the 1:20 year and 1:100 year return periods and various rainfall durations. Since the watersheds lie in the same climate region, only one set of values was required and selected. The 1:20 year and 1:100 year volumes for the 24 hour storm were compared to the 24 hour volumes in the statistics from New Chelsea and St. John's and were found to be reasonable as shown in Table 4.9. It was assumed that the shorter duration volumes, such as the 1 hour, obtained from the Frequency Atlas would also be reasonable estimates since these statistics were not available for New Chelsea or St. John's so that a comparison could be made. Table 4.10 provides the rainfall volumes for the 1:20 and 1:100 year recurrence interval for storm durations of 5 minutes to 24 hours.

TABLE 4.9 - COMPARISON OF RETURN PERIOD RAINFALL DEPTHS (mm)

Source	1:20	1:100
New Chelsea	92.4	118.2
St. John's Airport	104.1	129.1
St. John's	105.5	137.2
Frequency Atlas	95.0	116.8

Rain plus snowmelt statistics were also obtained and analyzed. However, the statistics showed smaller recurrence interval volumes when snowmelt was included and the snowmelt period was analyzed. Table 4.11 shows the return period rainfall volumes obtained by analysing data throughout 51 years and the rainfall plus snowmelt volumes obtained by analysing the combination of rainfall and snowmelt for the period from October to May for 51 years. The data used is from the St. John's Airport station. The results are typical of other stations which indicates that the larger return period storms are not as likely to occur during the period of spring runoff. Therefore, a snowmelt related runoff analysis was not carried out. The 1:20 and 1:100 year rainfall only event statistics were analyzed due to the larger resulting volumes.

TABLE 4.10
TOTAL RAINFALL VOLUMES FOR STUDY AREAS

Rainfall Duration	Mean Annual Extreme (mm)	Standard Deviation (mm)	1:20 (mm)	1:100 (mm)
5 min.	5.5	2	9.2	11.8
10 min.	7.5	2.5	12.2	15.3
15 min.	9	3.5	15.5	20.0
30 min.	13	5	22.3	28.7
1 hour	17	6	28.2	35.8
2 hours	23	8.5	38.9	49.7
6 hours	41	10	59.7	72.4
12 hours	50	13	74.3	90.8
24 hours	65	16.5	95	116.8

NOTES: 1. Rainfall Data are from Rainfall Frequency Atlas for Canada, 1985

TABLE 4.11
COMPARISON OF RAINFALL AND SNOWMELT STATISTICS

Comparison of Rainfall and Rain plus Snowmelt Statistics for St. John's Airport (51 years analyzed)				
	Return Period Values (mm)			
Return Period	Rainfall Only		Rain plus Snowmelt	
	1 day	30 days	1 day	30 days
2	67.39	223.36	41.14	191.00
5	83.89	267.70	52.78	245.93
10	94.84	297.13	60.50	282.39
25	108.65	334.25	70.23	328.36
100	129.07	389.14	84.63	396.35

Various storm durations with their corresponding 1:20 and 1:100 year rainfall volumes were evaluated when transforming design rainfall into hyetographs. One hour, 12 hour and 24 hour duration design storms were analyzed. The 24 hour storm hyetograph was developed using the

SCS Type II storm distribution which is one of the standard 24 hour storm distributions. The 1 hour and 12 hour distributions were based on studies performed by the Atmospheric Environment Service which analyzed gauged storms to come up with distributions for different storm durations which would maintain return period flows (Hogg, 1982). The distributions developed for the St. John's gauge by Atmospheric Environment Services were used. The three distributions are provided in Table 4.12.

The response of the watersheds to the various hyetographs were analyzed to determine which caused discharges closer in value to those determined with the regional regression equations and single station analysis. The 24 hour storm produced the greatest discharges in most of the watersheds, however, the values were much greater than those determined with the Regional regression equations. The 24 hour SCS Type II distribution has a tendency to over estimate peak discharges (Bedient & Huber, 1992). The 12 hour storm produced discharges closer in value to the Regional values and were chosen for comparison.

4.5 Flood Flow Estimate Discussion

The brooks and rivers under study have not been gauged over the years. As a result, flow measurements were not available for calibrating the OTTHYMO models. The flow estimates generated from the models were, instead, compared and verified with statistical methods as discussed before. The three statistical methods used included Regional Regression Analysis (1986 and 1990 equations), averaged Regional Flow from other stations, and averaged single station analysis. Table 4.13 shows a comparison of the various 1:20 and 1:100 year instantaneous flow estimates by OTTHYMO and the statistical estimates.

TABLE 4.12
RAINFALL DISTRIBUTIONS (percent distribution, %)

Interval (hrs.)	24 hour SCS Type II	12 hour AES	Interval (min)	1 hour AES
1	1	1	5	1
2	1	1	10	4
3	1	5	15	10
4	1	11	20	19
5	2	20	25	24
6	2	25	30	15
7	2	18	35	11
8	2	10	40	7
9	3	5	45	5
10	4	2	50	2
	6	1	55	1
11	45	1	60	1
12	9			
13	4			
14	3			
15	3			
16	2			
17	2			
18	2			
19	1			
20	1			
21	1			
22	1			
23	1			
24	1			

TABLE 4.13
COMPARISON OF INSTANTANEOUS PEAK FLOW ESTIMATES (M^3/S)

			100 year flows					20 year flows				
Location	Location	Area (sq km)	A	B	C	D	E	A	B	C	D	E
Carbonear	Powell's Bk to outlet	8	24.6	29.9	13.4	14.0	25.2	18.2	22.4	10.2	10.2	18.6
	Island Pond Bk to outlet	37.8	46.5	65.2	63.5	66.2	54.2	36.6	50.7	48.0	48.0	38.4
Hant's Harbour	Shorts Bk to outlet	7.9	8.2	70.1	13.3	13.8	17.7	6.4	47.8	10.0	10.0	12.6
	Halfway Bk to outlet	19.1	19.6	32.3	32.1	33.4	23.1	15.2	25.7	24.3	24.3	14.4
Heart's Delight	Heart's Delight Bk to outlet	44.1	32.2	48.6	74.1	77.2	47.3	26.1	39.5	56.0	56.0	31.2
	Brook #1 to outlet	5.5	10.2	15.3	9.2	9.6	11.2	7.8	12.0	7.0	7.0	7.9
	Brook #2 to outlet	2	2.8	40.5	3.4	3.5	3.3	2.2	26.3	2.5	2.5	2.4
Salmon Cove	Salmon Cove R to study area	52.4	60.1	110.1	88.0	91.7	88.3	44.4	83.0	66.5	66.5	61.2
Victoria	Salmon Cove R to study area	40.8	55.6	94.4	68.5	71.4	89.5	41	70.9	51.8	51.8	64.6
Whitbourne	Hodges R to Bethunes Pd	48.4	50.1	57.6	81.3	84.7	59.2	38.8	46.3	61.5	61.5	42.6
Winterton	Western Pond Bk to outlet	13	13.9	25.9	21.8	22.8	22.3	10.6	20.5	16.5	16.5	15

NOTES:

A - Regional Regression Analysis (1986 equations)
B - Regional Regression Analysis (1990 equations)
C - Averaged Regional Flow from hydro metric stations:
  Come by Chance River near Goobies (02ZH002).
  Northeast Pond River at Northeast Pond (02ZM006).
  Northwest Brook at Northwest Pond (02ZN001).
D - Averaged Single Station Analysis from other stations:
  Come By Chance River near Goobies (02ZH002).
  Northeast Pond River at Northeast Pond (02ZM006).
  Northwest Brook at Northwest Pond (02ZN001).
E - OTTHYMO simulation

The regression equation parameters, the OTTHYMO input parameters and the suitability of applying the average unit flows to the various watersheds were reviewed to evaluate the discrepancies in the flow estimates produced by the various methods. Since the flow values produced by applying the averaged Regional Flow from the nearby hydrometric stations and the averaged single station analysis were in very close agreement, they have been referred to as one set of values called average unit flow application values.

The OTTHYMO flow estimates were considered to be better estimates for each of the 11 study areas. Generally, this was because the flow values either agreed with the other estimates, were an average of the other estimates or because there was more doubt associated with the other flow estimates. There was more certainty associated with the OTTHYMO models since the model accounts for the response of each catchment, pond and river segment to the storm and could be checked for reasonableness through comparisons with the other OTTHYMO models and the statistical methods.

As well, the OTTHYMO model accounts for the catchment's physical features and their characteristics while the statistical methods were based on other catchments which might not have the same physical features as close as the ones in this study. For example, the mean annual runoff (MAR) values for the three hydrometric stations used to develop flow estimates from the average unit flow application are greater than the mean annual runoff values for the watersheds under study.

<u>Powells Brook - Carbonear:</u>

The flows obtained from both the 1984 and 1990 regression equations for Powells Brook were in close agreement with the flows obtained with the OTTHYMO models. The application of the average unit flow rates from the three hydrometric stations produced considerably lower flow values. This was believed to be a result of the low area controlled by lakes and swamps (ACLS) value for Powells Brook compared to the three hydrometric station values. The higher OTTHYMO values were recommended for computation of the flood profiles.

Island Pond Brook - Carbonear:

The flows resulting from the 1984 regression equation for Island Pond Brook were slightly lower in value to the flows resulting from the OTTHYMO model. The 1990 equation produced values similar to the flows from the average unit flow application which produced slightly higher values than the OTTHYMO values. However, the area covered by lakes and swamps factor for the 1990 regression equation was out of range. It was recommended that the OTTHYMO values be used for the flood profiles since they are an average of the various other flow estimates.

Shorts Brook - Hant's Harbour:

The regression equation produced the lowest flow values and the OTTHYMO model produced the greatest values. There was less confidence in the values resulting from the 1984 regression equation since both the shape factor and the area controlled by lakes and swamps fell outside of the applicable limits for the full equation. Thus, the two step back equation was used with the upper 95% confidence limits being close to the OTTHYMO values. There was also considerable doubt associated with the 1990 equation since the values for the area controlled by lakes and swamps and the area covered by lakes and swamps were out of range and the resulting flow values were unreasonable. The lower values resulting from the average unit flow application were believed to be a result of the low area controlled by lakes and swamps value for Shorts Brook compared to the three hydrometric station values, as was believed to be the case for Powells Brook. The OTTHYMO values were recommended for these reasons.

Halfway Brook - Hant's Harbour:

The situation for Halfway Brook was similar to Island Pond Brook and, again, the OTTHYMO values were recommended.

Heart's Delight Brook - Heart's Delight:

A considerable difference in flow estimates resulted between the average unit flow application and the 1984 regression equation application with the latter being lower. The OTTHYMO estimates were close to the average of these two estimates and were in close agreement with the flows resulting from the 1990 regression equation. There appeared to be no reason to doubt one method more than another and, thus, the average values produced with the OTTHYMO model were recommended.

Brook #1 - Heart's Delight:

All flow estimates were in close agreement. The OTTHYMO estimates were recommended.

Brook #2 - Heart's Delight:

All flow estimates were in close agreement, except those resulting from the 1990 regression equation. This was due to the fact that all of the input parameters fell outside of the applicable range for use in this equation. The OTTHYMO estimates were recommended.

Salmon Cove River - Salmon Cove:

The 1984 regression equation produced the lowest flow estimates for Salmon Cove and the 1990 equation produced the highest values. The OTTHYMO values were an average of the two regression equation values and were in close agreement with the values produced with the average unit flow application. Thus the OTTHYMO flow estimates were recommended.

Salmon Cove River - Victoria:

The OTTHYMO model used to estimate the flows for Victoria was part of the model for Salmon Cove. The OTTHYMO estimates were greater than the average unit flow estimates and the 1984 regression values, but less than the 1990 regression values. Since the model for Victoria was a part of the model for Salmon Cove, it was believed that if the model provided likely flow estimates for Salmon Cove, it would also do so for Victoria. Since the OTTHYMO flow estimates were recommended for Salmon Cove, they were also recommended for Victoria.

Hodges River - Whitbourne:

A considerable difference in flow estimates resulted between the averaged unit flow application (averaged Regional Flow and averaged single station flow) and the direct Regional 1984 regression equation application with the latter being lower. The OTTHYMO estimates were close to the average of the direct two other estimates and were in close agreement with the 1990 regression values. The average values produced with the OTTHYMO model were recommended.

Western Pond Brook - Winterton:

The direct application of the 1984 Regional regression equation produced the lowest flow values and the 1990 regression values were the highest. The average unit flow application produced average values along with the OTTHYMO estimates. Again, there appeared to be no reason to doubt one method more than another and, thus, the average values produced with the OTTHYMO model were recommended.

4.6 Conclusions

Without adequate flow records for a particular catchment area, the use of OTTHYMO to derive return period flow values is generally more accurate than using statistical methods derived by analysing other catchment areas. The OTTHYMO model is based on the physical features and characteristics of the particular catchment area, whereas the statistical methods are developed based on other catchments which may not have similar physical features.

The recommended final instantaneous peak flow values from the OTTHYMO models are presented in Table 4.14. Although, the models could not be completely calibrated, the results were compared with flow estimates from various statistical methods in order to increase the confidence in the model results. These flow values were used in the hydraulic (HEC-2) models of each study area to derive the 1:20 year and 1:100 year floodlines. The crest gauge information collected during the course of this study was used to calibrate the HEC-2 models.

TABLE 4.14
RECOMMENDED OTTHYMO PEAK DISCHARGE VALUES

Location	Description	100 year Qp (m^3/s)	20 year Qp (m^3/s)
Carbonear	Powell's Brook to outlet	25.2	18.6
	Island Pond Brook to outlet	54.2	38.4
Hant's Harbour	Shorts Brook to outlet	17.7	12.6
	Halfway Brook to outlet	23.1	14.4
Heart's Delight	Heart's Delight Brook to outlet	47.3	31.2
	(south) brook to outlet	11.2	7.9
	(north) brook to outlet	3.3	2.4
Salmon Cove	Salmon Cove River to study area	88.3	61.2
Victoria	Salmon Cove River to study area	89.5	64.6
Whitbourne	Hodges River to Bethumes Pond	59.2	42.6
Winterton	Western Brook Pond Brook to outlet	22.3	15.0

5.0 HYDRAULIC INVESTIGATIONS

5.1 Introduction

Hydraulic models were developed for each of the study areas in order to derive the 1:20 and 1:100 year flood levels and to analyze ice jams near structures. The BOSS HEC-2 model (BOSS Corporation, 1988-1992) was chosen since it has been used extensively for these purposes. It is based upon an optimized version of the U.S. Army Corps of Engineers Hydrologic Engineering Center water surface profile computation model HEC-2. The HEC-2 program uses the standard step procedure to compute changes in water surface elevation between adjacent cross-sections. Computations start at one end of the river being studied and progress cross-section by cross-section to the other end. At bridge and culvert locations, where flow mechanics are more complex, other methods are used to determine the change in surface elevation.

The program calculates water surface profiles for steady or gradually varied flow in natural or man-made channels and both subcritical and supercritical profiles can be computed. The program can account for backwater created by bridges, culverts, weirs, and other floodplain structures. The assumptions made in developing the model include:

- Flow is steady or gradually varied;
- Flow is one-dimensional; velocity components in directions other than the direction of flow are not accounted for;
- River channels have small slopes (less than 1:10). The pressure head is represented by the water depth measured vertically downward and not normal to the sloping water surface;
- Friction slope is assumed constant between adjacent cross-sections.
- Rigid flow boundaries are assumed; and,
- Profile computations are not allowed to pass through critical depth. Separate analyses must be performed for subcritical and supercritical profiles.

The features added to the HEC-2 program by the BOSS Corporation include data entry routines that are designed to save time and to increase productivity by preventing errors due to simple typing mistakes.

Recently, a study on the Trout River on the west side of the Island of Newfoundland successfully used the Hec-2 model to develop the 1:20 and 1:100 year floodlines (Department of Environment and Lands, 1990).

5.2 Hydraulic Model Structure and Input Data

The surveyed cross sections of the low flow channel bathymetry in the study areas were extended into the floodplain approximately perpendicular to the flow direction using the 1:2500 topographic mapping. Checks were made to ensure that each section adequately described the channel valley geometry and flow-carrying capacity. Where necessary, ineffective flow areas of the floodplain, such as indents in the valley floor, were removed from the cross section geometry; refer to Figure's 3.1 to 3.9.

The location of the cross sections along each stream were measured in meters along the channel centerlines starting from the outlets to the ocean, except for Whitbourne for which cross sections were measured from Bethunes Pond, Salmon Cove from Salmon Cove Pond and Victoria for which the first cross section was assigned a chainage of 100. The right edge of the brook (looking downstream) at low flow (that shown on the maps) for each cross section was given a reference number of 1000 when entered into the model. Horizontal distances to the right of the channel center (looking downstream) were added to the reference number, while distances to the left were subtracted. The channel lengths between cross sections were surveyed along the channel centerline. The left and right overbank flow lengths were determined using the 1:2500 topographic mapping. These lengths represent the anticipated path of the center of mass of the overbank flow.

5.2.1 Hydraulic Structures

A total of 24 structures were deemed to have hydraulic significance and were included in the models. Table 5.1 lists the location of each structure. Photographs of each structure were taken and have been included on the Structural Data Sheets in the Technical Appendices.

The special bridge method was used to calculate the hydraulic losses through all bridges to ensure that the models were applicable for all flow rates. This method requires four cross sections, which correspond to sections at the upstream and downstream faces of the bridge and sections located far enough upstream and downstream not to be affected by the bridge.

The special culvert method was used for all culverts. However, this method requires that the culvert be specified using either a diameter or rectangular dimensions and may not accurately model the flow through irregularly shaped openings of culverts. An arched culvert is an example of an irregularly shaped culvert. These were modelled by approximating a diameter to maintain the same opening area. The special culvert method has the same cross section location requirements as the special bridge method.

5.2.2 Hydraulic Parameters

The required hydraulic parameters include various loss coefficients used to calculate losses due to flow expansion and contraction, flow obstructions such as bridges and culverts and friction losses. The expansion and contraction loss coefficients consist of values for the gradual contraction and expansion of flow experienced along the entirety of the channel reach under study and for more abrupt changes at bridges and culverts. The values used in all models for the gradual contraction and expansion coefficients were 0.1 and 0.3, respectively. The values used for the abrupt changes at bridges and culverts are typical values. These are presented in Tables 5.1 along with the other required loss coefficients.

5.2.3 Manning's Roughness Coefficients

Manning's roughness coefficients (n) were used in the model to account for the friction losses. Ideally, Manning's n values are used to calibrate HEC-2 models to recorded historical flood events. However, there have not been any flow gauges on any of the streams under study. Thus, information obtained from the crest gauge and velocity measurements taken during the course of this study were used to select the initial Manning's n values.

The initial Manning's roughness coefficients for the channel reaches and overbank areas were chosen based on a variety of factors. Factors considered for the overbank areas were the type and density of vegetation. The channel coefficients were determined based on the meander pattern, paving material, slope and presence of woody debris and vegetation. These factors were determined with the aid of photographs and field observations. The initial values of the roughness coefficients were obtained from tables relating descriptions of channel reaches and overbank areas to the corresponding roughness values (Chow, 1959).

TABLE 5.1
SUMMARY OF HYDRAULIC STRUCTURES AND COEFFICIENTS (UNCALIBRATED)

								FWHA CHART		CULVERT	BRIDGE
	Brook	Town/Location	Type	Manning's n	Ce	Cc	Cw	Num	Scale	Ent. Loss	Total Loss
C11	Island Pond	Carbonear/Outlet	Bridge		0.5	0.3	1.7				1.5
C12	Island Pond	Carbonear/d/s Salmon Cove Pond	Bridge (train trestle)		0.5	0.3	1.7				1.5
C13	Island Pond	Carbonear/d/s Salmon Cove Pond	Bridge (train trestle)		0.5	0.3	1.7				1.5
C14	Island Pond	Carbonear/Pond Side Rd. & Cross Rd.	Bridge		0.5	0.3	1.7				1.5
C15	Island Pond	Carbonear/C.N.B. Highway	Culvert	0.023	0.8	0.5	1.7	2	3	0.8	
CP1	Powell's	Carbonear/Lower South Side Rd.	Bridge		0.5	0.3	1.7				1.5
CP2	Powell's	Carbonear/u/s Lower South Side Rd.	Bridge (train trestle)		0.5	0.3	1.7				1.5
CP3	Powell's	Carbonear/Highroad South	Bridge		0.5	0.3	1.7				1.5
CP4	Powell's	Carbonear/Cinema	Culverts	0.023	0.8	0.5	1.7	2	3	0.8	
CP5	Powell's	Carbonear/Industrial Loop @ NF Power	Culverts	0.025	0.8	0.5	1.7	2	3	0.8	
CP6	Powell's	Carbonear/Industrial @ Penny Transport	Culverts	0.023	0.8	0.5	1.7	2	3	0.8	
HH1	Halfway	Hant's Harbour	Bridge		0.5	0.3	1.7				1.5
HS1	Shorts	Hant's Harbour	Bridge		0.5	0.3	1.7				1.5
HS2	Shorts	Hant's Harbour	Culverts	0.024	0.8	0.5	1.7	2	3	0.8	
HD1	Heart's Delight	Heart's Delight	Bridge		0.5	0.3	1.7				1.5
HU1	Brook # 2	Heart's Delight	Culvert	0.023	0.8	0.5	1.7	2	3	0.8	
HU2	Brook # 2	Heart's Delight	Culvert	0.023	0.8	0.5	1.7	2	3	0.8	
HL1	Brook # 1	Heart's Delight	Culvert		0.8	0.5	1.7	2	3	0.8	
SS1	Salmon Cove	Salmon Cove	Bridge		0.5	0.3	1.7				1.5
VS1	Salmon Cove	Victoria/C.N.B. Highway	Bridge		0.5	0.3	1.7				1.5
WH1	Hodges River	Whitbourne	Culvert	0.023	0.8	0.5	1.7	2	3	0.8	
WH2	Hodges River	Whitbourne	Culvert	0.023	0.8	0.5	1.7	2	3	0.8	
WW1	Western Pond	Winterton	Culvert	0.023	0.8	0.5	1.7	2	3	0.8	
WW2	Western Pond	Winterton	Culvert	0.023	0.8	0.5	1.7	2	3	0.8	

NOTES:
Ce = Expansion loss coefficient
Cc = Contraction loss coefficient
Cw = Roadway weir loss coefficient
Ent. = Entrance loss coefficient
FWHA = Federal Highway Administration
NUM & Scale = Required parameters for inlet control equations (FWHA, 1985)

The descriptions and corresponding values used in the models are shown in Table 5.2.

The values of the channel roughness coefficients were based on bankfull stage. The roughness coefficients for certain types of overbank areas may vary due to flood stage. Examples would be areas containing high grass or row crops. As the flood elevation increases, the friction loss is reduced as the grass or crop becomes flattened. Since these conditions were not found to exist along the study reaches, no adjustments were needed to be made to the Manning's n values for the overbank areas for the various flow conditions.

Table 5.2
Values of the Roughness Coefficient n (Chow, 1959)

Channel and Overbank Description	n
Clean, straight, no rifts or deep pools, some stones and weeds	0.035
Clean, winding, some pools and shoals	0.04
Same as above, but some weeds and stones	0.045
Same as above, but more stones	0.050
Sluggish reaches, weedy, deep pools	0.07
Overbank Areas:	
Short grass or cultivated area without crops	0.03
High grass or mature row crops	0.035
Mature field crops	0.04
Scattered brush, heavy weeds	0.05
Light brush and trees	0.06
Medium to dense brush or medium dense trees	0.10

5.2.4 Flow Input

The recommended peak 1:20 and 1:100 year design flows resulting from the hydrologic analyses (Table 4.8) were used in the Hec-2 models for each study area to facilitate the calculation of the 1:20 and 1:100 year flood elevations.

5.2.5 Starting Water Surface Elevations

Starting water elevations for each of the study areas were determined using the slope-area method (Manning's flow equation) to find the normal depth of flow. For those study areas with the first cross section located close to the outlet, the normal flow elevations were then compared with the mean ocean levels at the outlets to determine which was higher. Thirty-one years (1962-1993) of monthly means and peak instantaneous sea level values for St. John's Harbour were obtained from the Department of Fisheries and Oceans. From this the overall mean was found to be -0.01 MGD. Correction factors of minus 0.091 m and minus 0.015 m were obtained for Harbour Grace and Heart's Content, respectively. The correction factor for Harbour Grace was assumed to be applicable for Carbonear as was Heart's Content for Heart's Delight, Winterton and Hant's Harbour. In all cases, the normal flow elevations were found to be higher than the mean ocean levels and were, thus, used as the starting water elevations.

In addition to the mean, the highest ever recorded water level of 2.5 m (1.69 m GD) for St. John's was used as the starting water elevation in those models that had a lower normal starting elevation. The effects of this are described in section 5.5.3.

5.3 Hydraulic Model Calibration

The hydraulic models were calibrated using the stage-discharge curves obtained from each of the crest gauges installed within the study areas. Table 5.3 presents the crest gauge

Table 5.3
Crest Gauge and Discharge Measurement Data

Location	Date	Discharge (cms)	Water Level (MSL)
Carbonear - Island Pond Brook	18-Nov	1.133	1.030
cross section no. 396	22-Dec	1.220	1.180
	18-Feb	0.764	0.880
	13-Mar	3.201	1.330
Carbonear - Powell's Brook	18-Nov	0.220	6.057
cross section no. 130	22-Dec	0.173	6.087
	15-Feb	0.125	6.027
	13-Mar	0.334	6.157
Hant's Harbour - Halfway Brook	21-Nov	0.320	1.725
cross section no. 244	22-Dec	0.915	1.855
	17-Feb	frozen	frozen
	13-Mar	1.782	1.865
Hant's Harbour - Shorts Brook	21-Nov	0.064	0.021
cross section no. 74	22-Dec	0.259	0.011
	17-Feb	0.624	-0.079
	13-Mar	0.609	-0.109
Heart's Delight - Heart's Delight Brook	23-Nov	0.896	1.364
cross section no. 218	22-Dec	1.372	1.404
	20-Feb	0.828	1.364
	13-Mar	4.172	1.684
Salmon Cove - Salmon Cove River	19-Nov	1.238	8.024
cross section no. 431	22-Dec	1.699	8.074
	17-Feb	0.960	8.084
	13-Mar	3.486	8.114
Victoria - Salmon Cove River	18-Nov	1.284	42.365
cross section no. 247	22-Dec	frozen	43.015
	17-Feb	0.484	42.255
	13-Mar	2.042	42.485
Whitbourne - Hodges River	14-Dec	3.490	56.230
cross section no. 314	22-Dec	2.570	56.000
	21-Feb	0.684	55.830
Winterton - Western Pond Brook	22-Nov	0.356	19.424
cross section no. 642	22-Dec	0.786	19.524
	20-Feb	0.151	19.404
	13-Mar	1.115	19.824

FIGURE 5.4 – FLOOD INFORMATION MAP (VICTORIA)

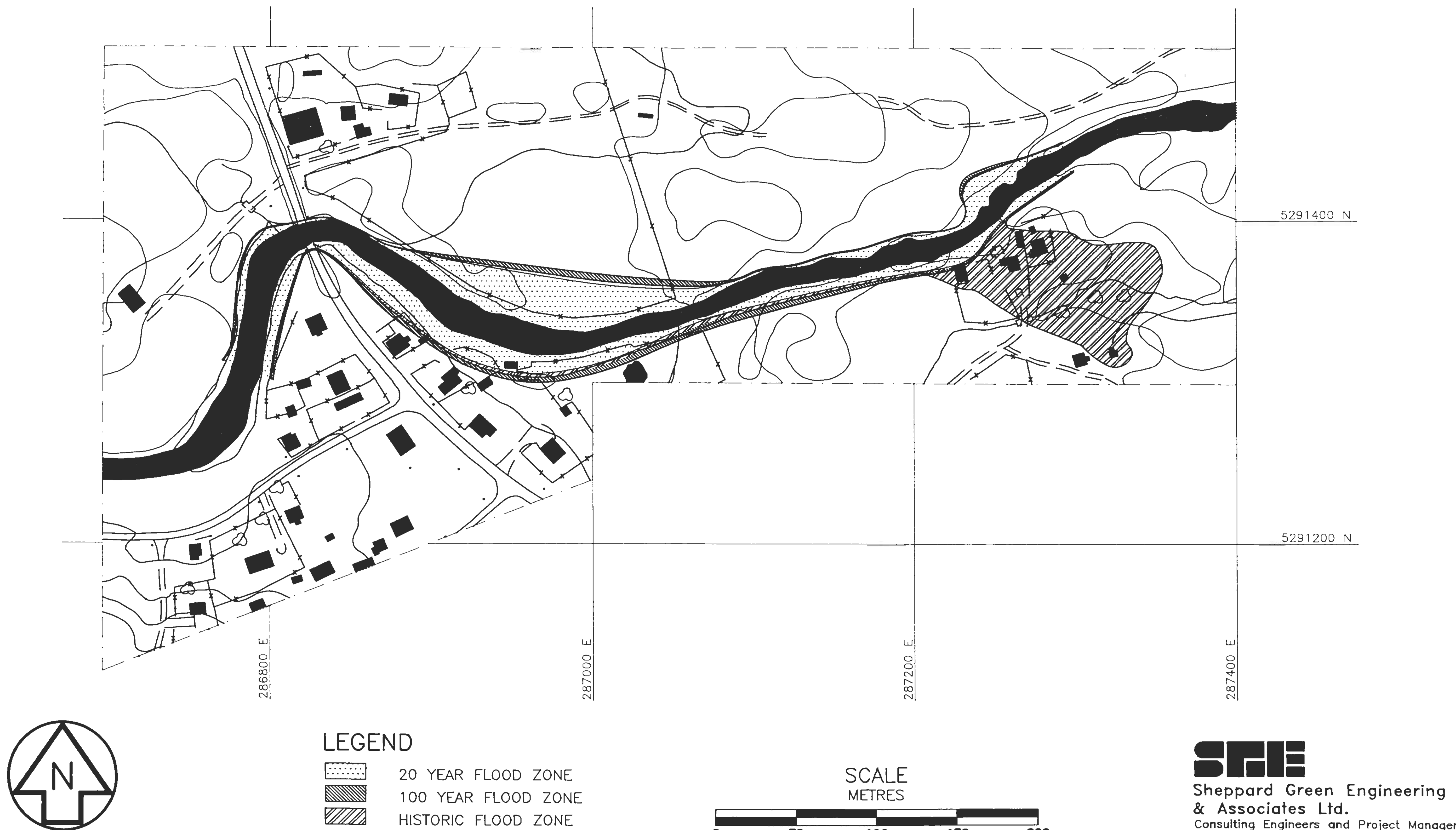
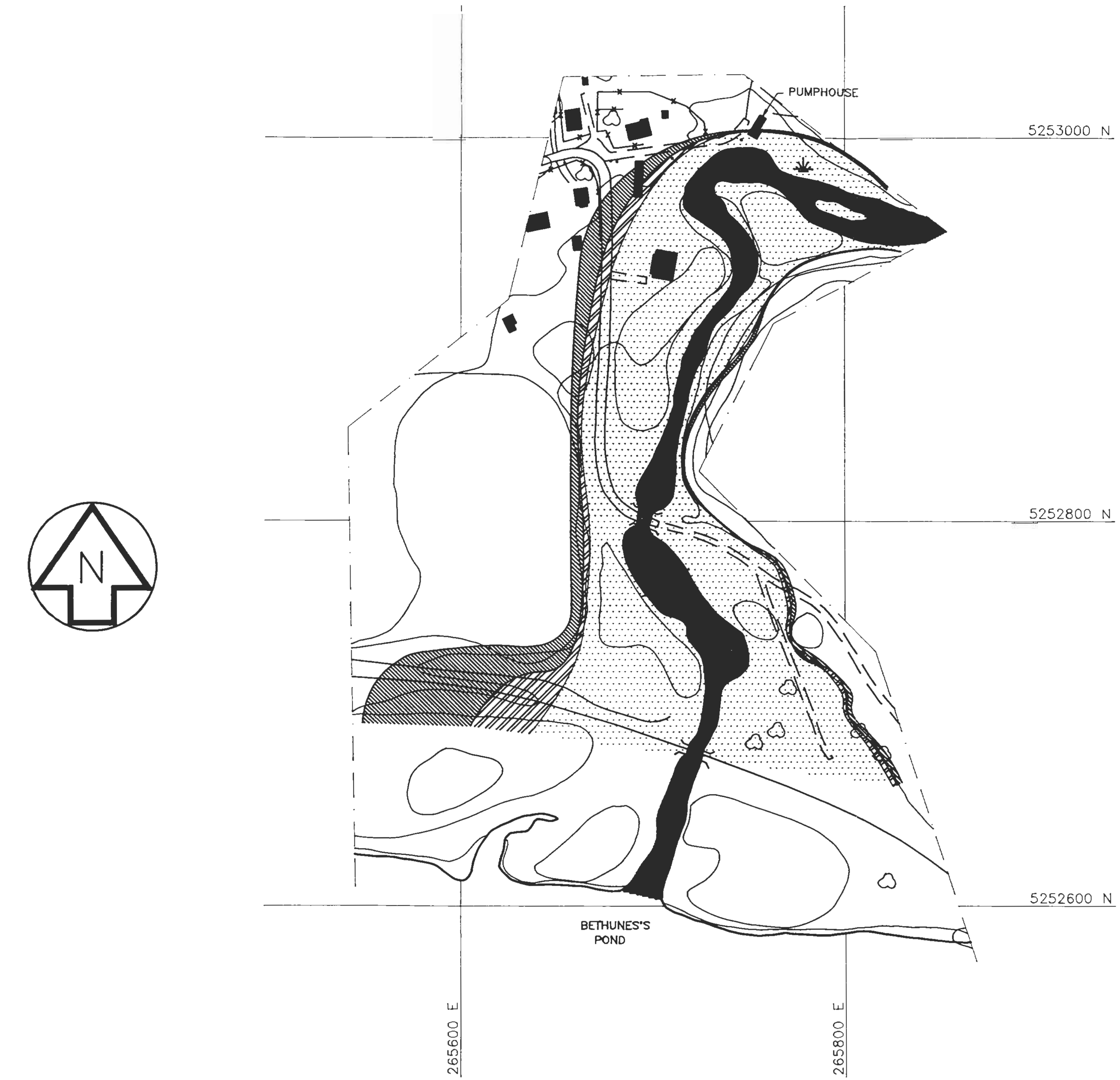


FIGURE 5.6
FLOOD INFORMATION MAP
(WHITBOURNE)



LEGEND

- 20 YEAR FLOOD ZONE
- 100 YEAR FLOOD ZONE
- HISTORIC FLOOD ZONE

SCALE
METRES
0 50 100 150 200

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FIGURE 5.7 – FLOOD INFORMATION MAP (HEARTS DELIGHT)

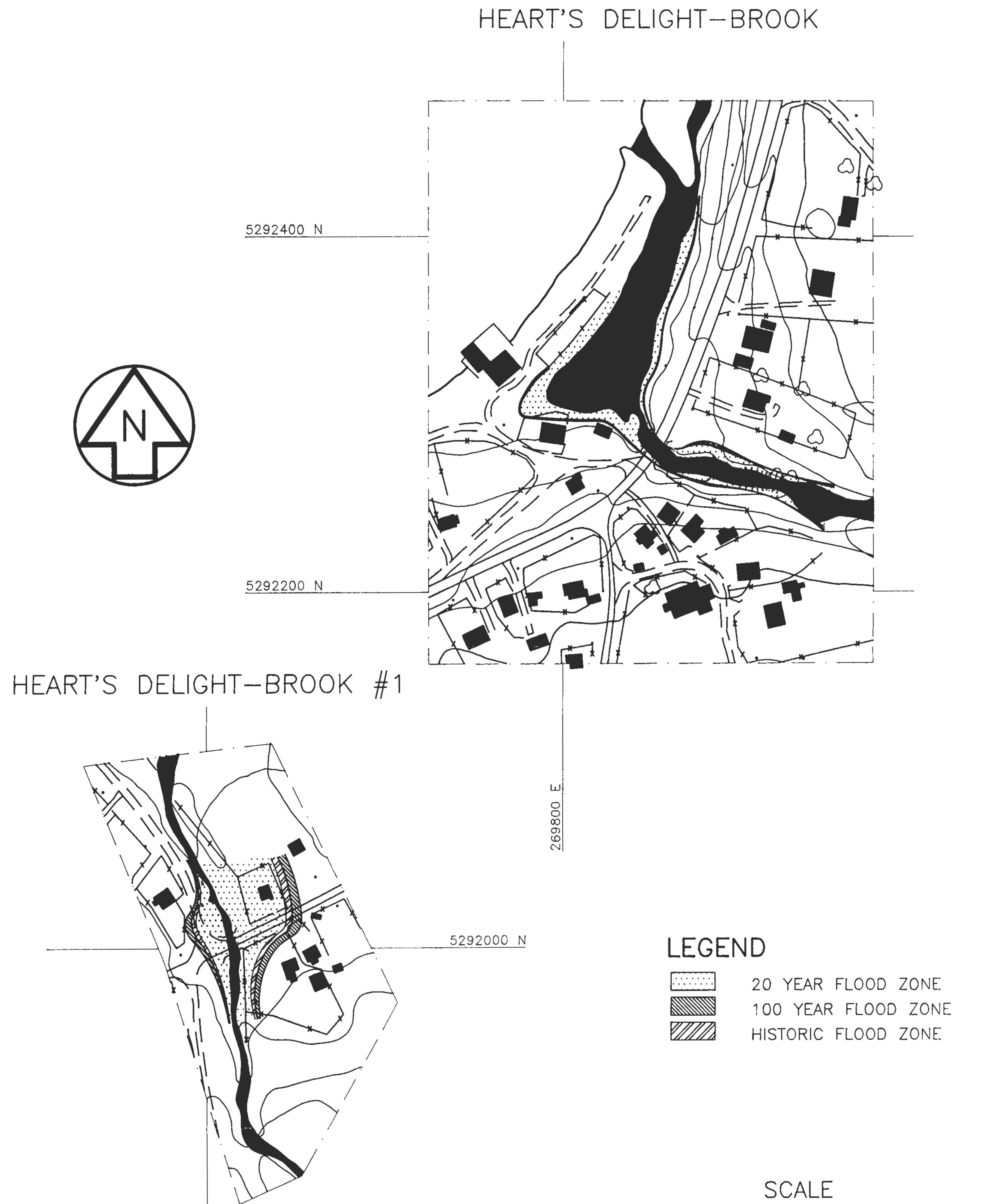

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FIGURE 5.8 – FLOOD INFORMATION MAP (HEARTS DELIGHT)

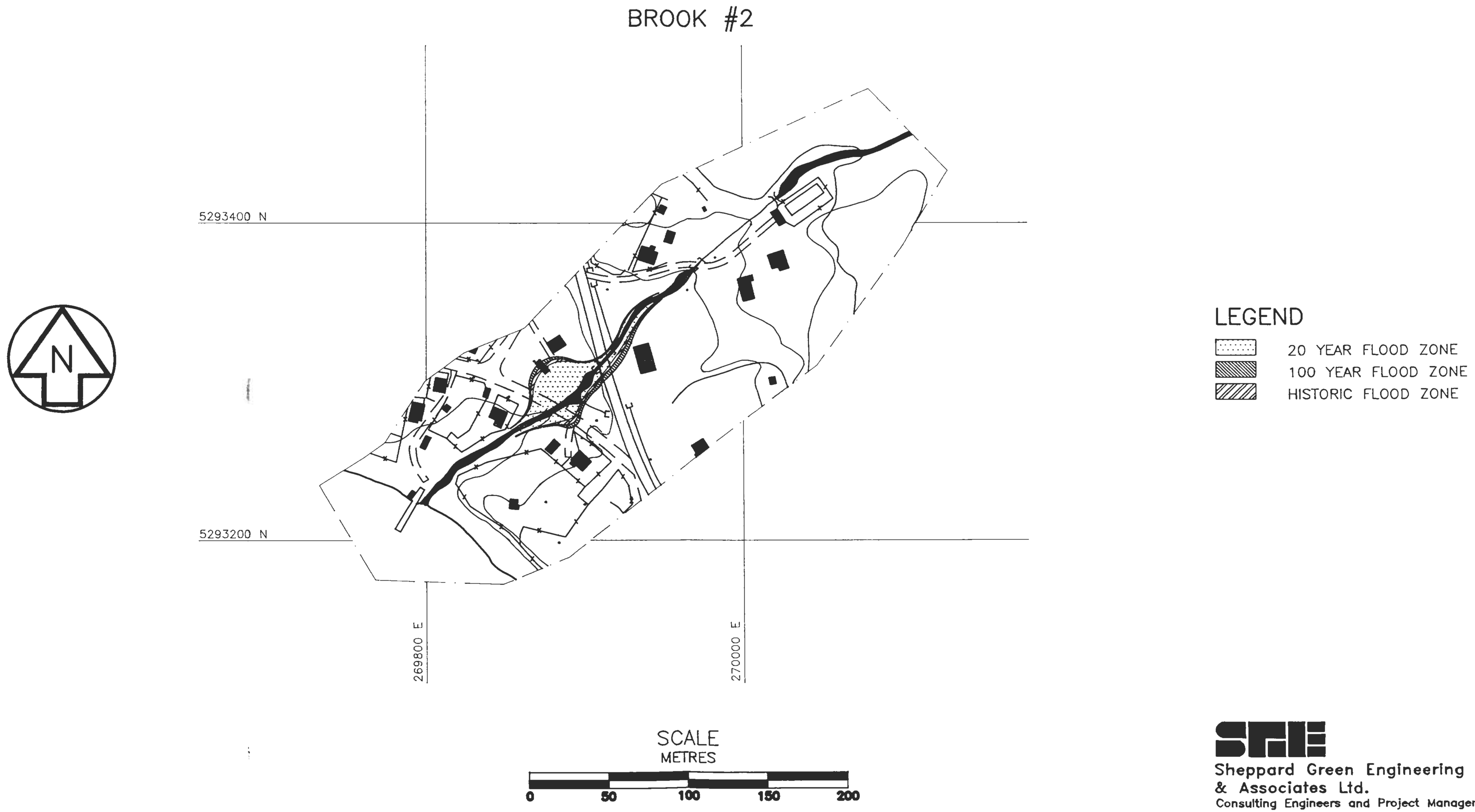
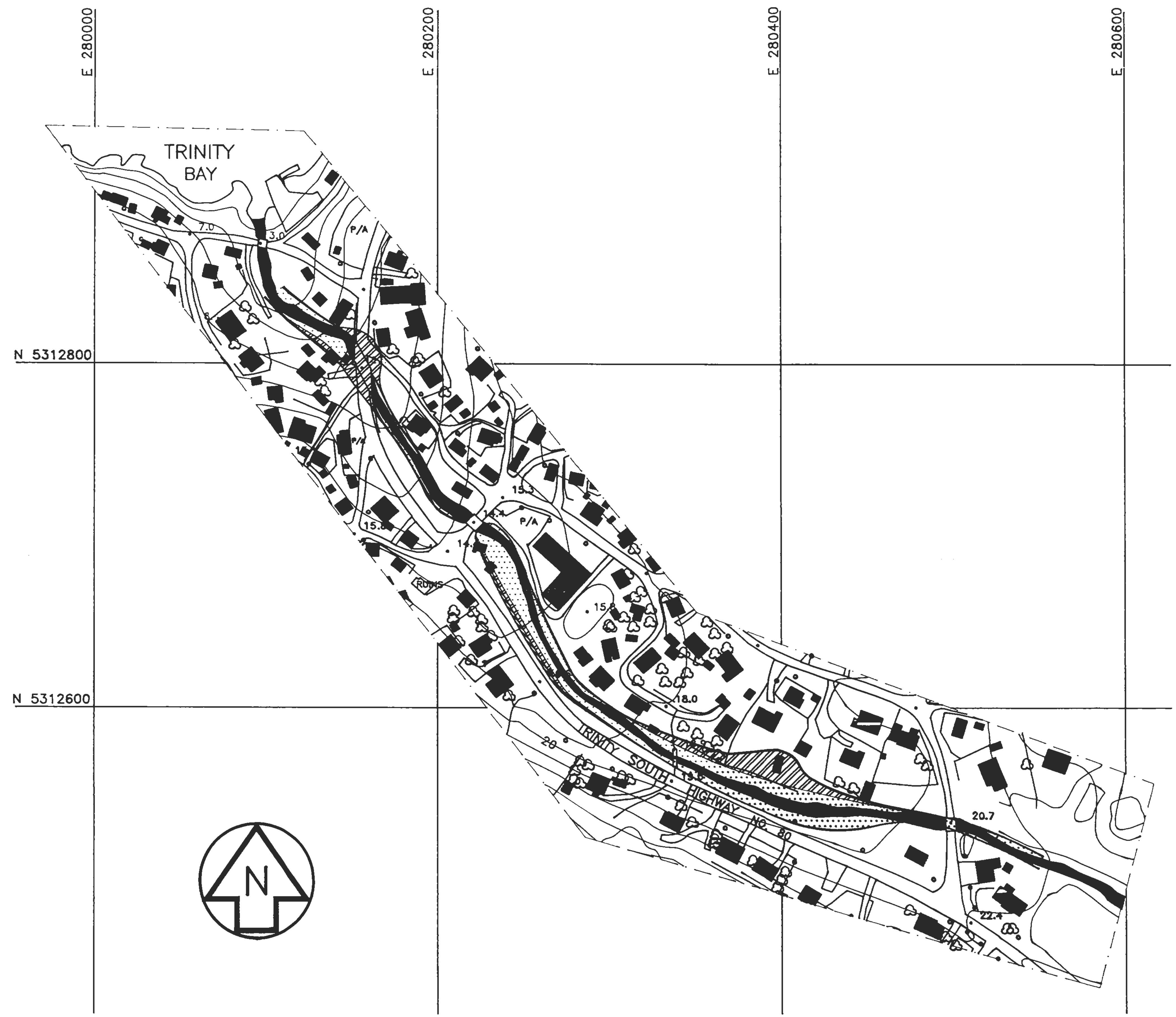


FIGURE 5.9
FLOOD INFORMATION MAP (WINTERTON)



LEGEND

20 YEAR FLOOD ZONE

100 YEAR FLOOD ZONE

HISTORIC FLOOD ZONE

SCALE
METRES

0 50 100 150 200

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FIGURE 5.10 – FLOOD INFORMATION MAP (HANT'S HARBOUR)

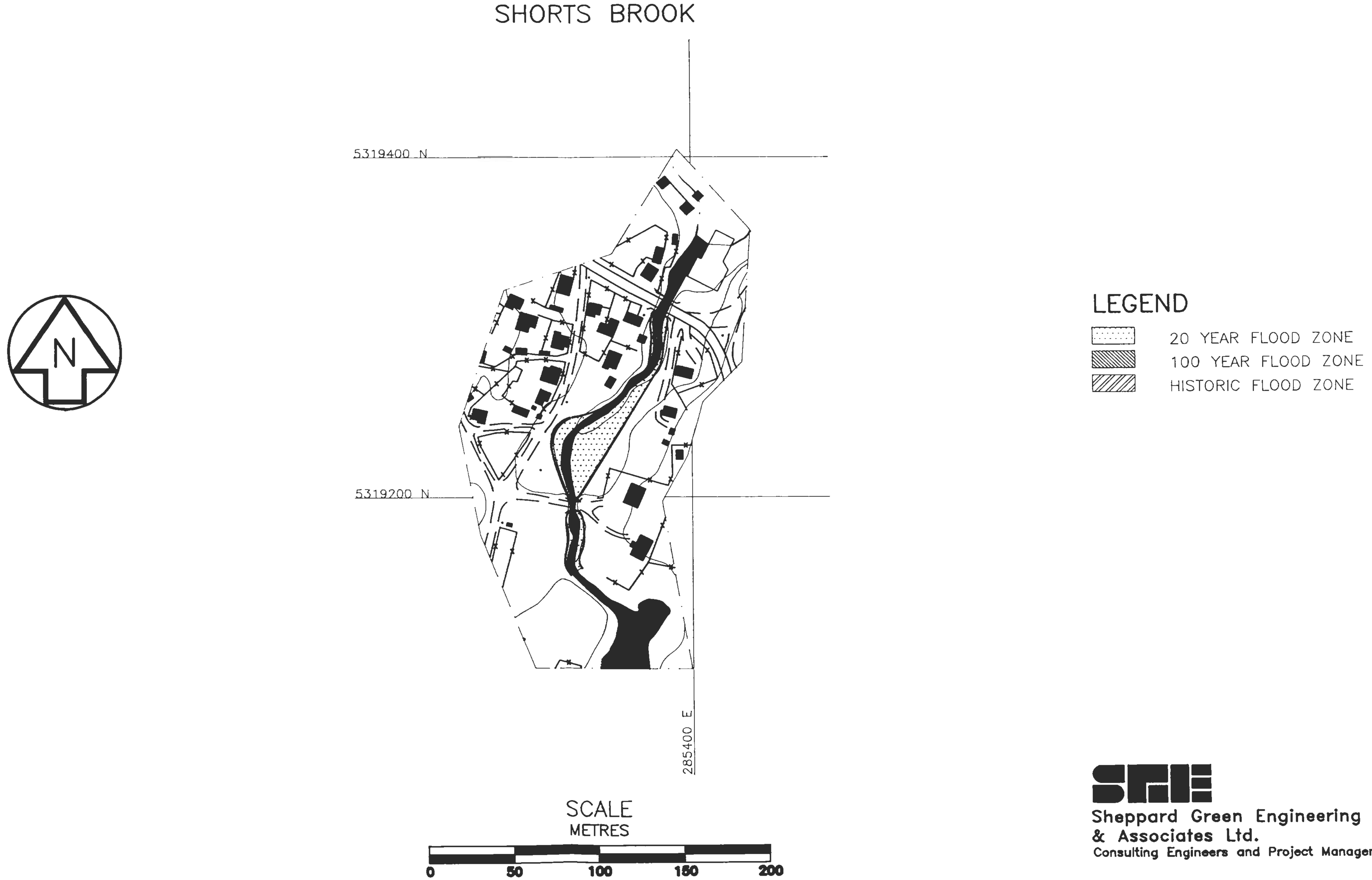

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FIGURE 5.11 – FLOOD INFORMATION MAP (HANT'S HARBOUR)

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information including the dates visited, measured flows, and water levels, that was collected. over the course of this study. The HEC-2 models were run with a series of flows in the range of that determined in the field at the time of crest gauge measurement. The resulting elevations at the crest gauge locations in the models were plotted with the corresponding flows and compared to the field data. Changes were then made to the Manning's friction factors in each of the HEC-2 models until the stage-discharge plots matched the field data points.

The expansion and contraction loss coefficients and the coefficients accounting for losses due to bridges and culverts were not found to have a significant effect on the water elevations and were, therefore, not adjusted during the calibration.

Table 5.4 shows the final Manning's n values used to calibrate the HEC-2 models. Plots of the HEC-2 stage versus discharge for the study areas along with the field data points used for calibration have been included in Appendix G of the Technical Appendices.

For each of the study areas only the channel friction factors could be calibrated since only channel flow measurements were obtainable at the crest gauge locations during the course of this study. Although the overbank values were initially based on Table 5.2, they were increased correspondingly to maintain a reasonable difference in roughness between the low flow channel and overbanks. A discussion of the calibration results is presented in the following sections.

Table 5.4
Final Manning's Coefficients of Friction

Study Area:	**Channel n values:**	**Overbank n values:**
Carbonear - Island Pond Brook	0.055	0.060
Carbonear - Powells Brook	0.060	0.065
Hant's Harbour - Halfway Brook	0.040	0.050
Hant's Harbour - Shorts Brook	0.045	0.050
Heart's Delight - Heart's Delight Brook	0.065	0.070
Heart's Delight - (north) brook	0.060	0.070
Heart's Delight - (south) brook	0.060	0.070
Salmon Cove - Salmon Cove River	0.035	0.050
Victoria - Salmon Cove River	0.060	0.090
Whitbourne - Hodges River	0.070	0.080
Winterton - Western Pond Brook	0.075	0.080

<u>Carbonear - Island Pond Brook:</u>

The value of 0.055 for the channel friction factor is reasonable for Island Pond Brook since this value represents a clean channel with some pools and shoals and a large number of stones. The overbank value of 0.060 reflects the friction due to light brush and trees. The study area, located in the town of Carbonear, has very few trees. However, brush is located along this reach.

<u>Carbonear - Powells Brook:</u>

The values of 0.060 and 0.065 for the channel and overbank friction factors, respectively, are very close to those for Island Pond Brook. The values are slightly higher for Powells Brook due to a greater number of stones and boulders lining the channel and more brush located along the study reach.

Victoria - Salmon Cove River:

The channel friction factor value of 0.060 is greater than the value for the study area in Salmon Cove due to a large meander in the channel and a very low slope making the reach more sluggish. The overbank value of 0.090 represents an area with medium to dense trees.

Salmon Cove - Salmon Cove River:

The calibrated friction factor value of 0.035 for the channel was the lowest value out of all of the study areas. This represents a clean, straight channel with no rifts or deep pools, but with stones lining the channel. This description appears reasonable for the study reach. The overbank value of 0.05 represents an area with light brush.

Whitbourne - Hodges River:

The study reach in Whitbourne has the lowest slope of any of the other study reaches which explains why the channel friction factor value of 0.070 is reasonable. The overbank area for Whitbourne is very similar to that for Victoria, except that there are a few open areas. Therefore, the value of 0.080 is reasonable.

Heart's Delight - Heart's Delight Brook:

The study reach along Heart's Delight Brook was found to be very similar to the study area along Powells Brook. The values of 0.065 and 0.070 for the channel and overbank friction factors, respectively, are slightly higher than the values for Powells Brook due to a greater number of stones and boulders lining the channel and thicker brush in the overbank areas.

Heart's Delight - Brook # 2:

A crest gauge was not set up on this brook. Therefore, the HEC-2 model could not be calibrated. Instead, the friction factors were determined by referring to the other study reaches with similar characteristics. The value of 0.060 for the channel friction factor was chosen since this represents a clean channel with some pools and shoals and a large number of stones. The overbank value of 0.070 represents an area with medium to dense brush.

Heart's Delight - Brook #1:

A crest gauge was not set up on this brook. Therefore, the HEC-2 model could not be calibrated. Instead, the friction factors were determined by referring to the other study reaches with similar characteristics. The value of 0.060 for the channel friction factor was chosen since the reach under study had characteristics very similar to the brook north of Heart's Delight Brook in Heart's Delight. The overbank value of 0.050 represents an area with light brush.

Winterton - Western Pond Brook:

Although, according to Chow, friction factors generally increase with decreasing slope, field studies (Rosgen, 1992) have indicated that very steep channel reaches can also have high friction factors. This would explain the channel friction factor of 0.075 for Winterton even though the slope of the Western Pond Brook is as high as 0.05 m/m and the channel bottom contains a large number of boulders. The overbank value of 0.08 represents the friction due to dense brush.

Hant's Harbour - Halfway Brook:

The study area on Halfway Brook is located at a straight and clean section of the channel in an open area with heavy weeds. The values of 0.040 and 0.050 for the channel and overbank friction factors, respectively, are reasonable.

Hant's Harbour - Shorts Brook:

The water level and flow measurements taken at the crest gauge location on Shorts Brook indicated that the site was affected by ocean water levels. As a result, this information could not be used to calibrate the HEC-2 model. The values of 0.045 and 0.050 for the channel and overbank friction factors, respectively, were determined by referring to the values for Halfway Brook. Often channels located in close proximity share common characteristics. The overbank areas for the two reaches appeared very similar. The channel friction factor for Shorts Brook was set slightly higher than that for Halfway Brook due to the presence of more stones.

5.4 Discussion of Calibrated Hydraulic Models and Open Water Floodlevels

The calibrated HEC-2 models for each study area were used to compute the 1:20 and 1:100 year flood levels and to determine areas of concern with respect to the flood flows. Plots of the 1:100 year flood elevations at each cross section in the model are provided in Appendix H along with the 1:100 year flood profiles and summary profiles which compare the 1:20 and 1:100 flood elevations. Refer to Figure 5.1 and 5.2 as examples of the output.

FIGURE 5.1 - CROSS SECTION 308m (SALMON COVE RIVER, SALMON COVE)

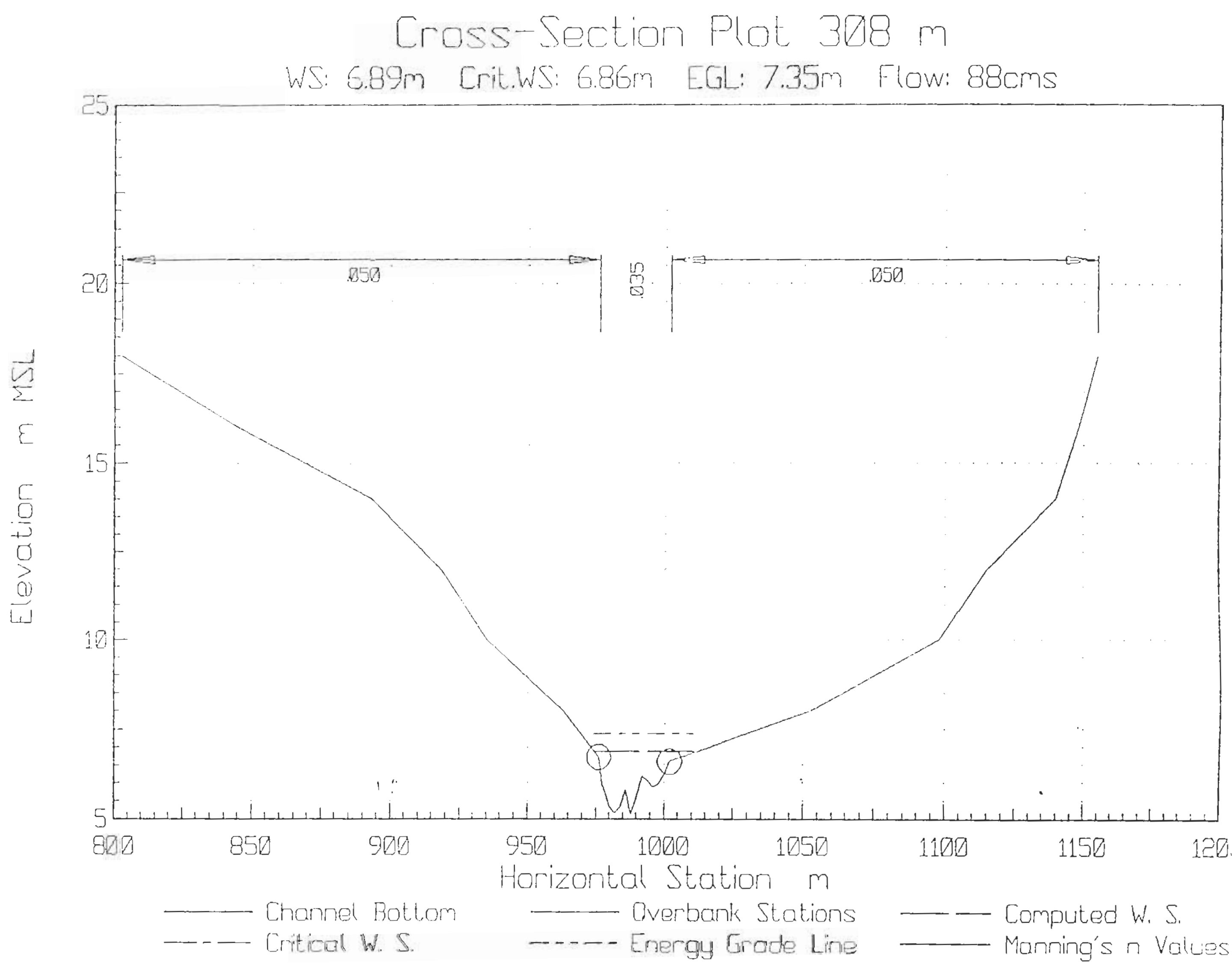
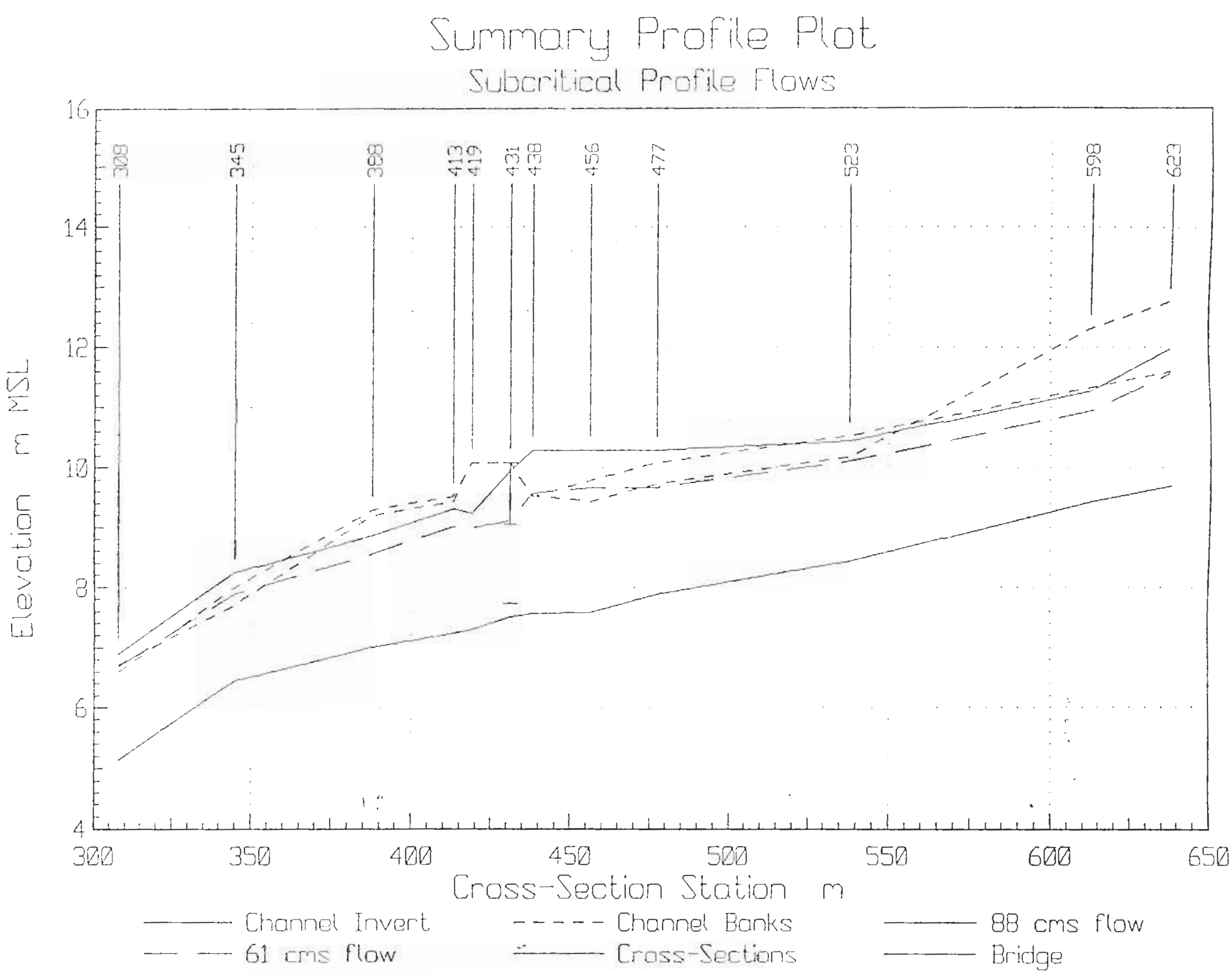


FIGURE 5.2 - SUMMARY PROFILE PLOT (SALMON COVE RIVER, SALMON COVE)



Island Pond Brook - Carbonear:

The 1:100 year flow of 54.2 m^3/s through Island Pond Brook resulted in a water level 21 cm below the low chord of the furthest downstream bridge. Pressure and weir flow resulted at the first train trestle. The pressure flow was 45.3 m^3/s and the weir flow was 8.9 m^3/s. A pressure flow of 30 m^3/s and a weir flow of 24.2 m^3/s resulted at the second train trestle. These train trestles appear to cause extensive flooding problems. Pressure flow and some weir flow resulted at the upstream road bridge. The pressure flow was 51 m^3/s and the weir flow was 4.2 m^3/s. This bridge does not appear to be a great flow obstruction.

The total opening area of the three culverts, (two arched and one circular), was approximated by specifying a diameter for each culvert in the model and using the Special Culvert Routine. This was necessary since the model does not handle different sized or non-circular culverts. Since these culverts did not cause a great deal of backwater for the 1:100 year flows, the assumption was made that this approximation would not significantly affect flood levels. The assumption was tested by specifying various culvert diameters and checking the water elevations upstream of the culverts. There were no significant changes to these elevations.

It became evident that the 1:100 year flow would overtop Valley Road. To be conservative, the loss of flow as a result of this spill was not accounted for in the model and was assumed not to significantly affect flood levels. The path of spill is indicated on the floodline map. In general, the spill will flow to the left of the brook and re-enter one of the tributaries of the brook. Since there are no buildings or structures in the path of the spill, there are no flooding concerns at this location.

Powells Brook - Carbonear:

The 1:100 year flow of 25.2 m^3/s did not overtop the road bridges and the train trestle. Pressure and weir flow occurred at all culverts, but there was minimal weir flow at the culverts by the Carbonear Cinema (cross section no. 536). The pressure flow at the culverts by the cinema was 23.4 m^3/s and the weir flow was 1.8 m^3/s. Weir flow did not occur for the 1:20 year flow. Pressure flow of 18.4 m^3/s and weir flow of 6.8 m^3/s resulted at the culverts by Newfoundland Power and pressure flow of 20.0 m^3/s and weir flow of 5.2 m^3/s resulted at the culverts by Penny's Transport. The culverts by Newfoundland Power are an obvious flow obstruction due to the low gradient of the culverts and low inverts compared to the channel inverts. Water is backed up considerably at both the culverts near Newfoundland Power and the Carbonear Cinema.

Salmon Cove River - Victoria:

The 1:100 year flow of 89.5 m3/s resulted in a water level 0.25 m below the low chord of bridge under study. The possibility of overbank flow between sections 100 and 583 was eliminated by removing ineffective flow areas. A fairly large tributary of Salmon Cove River enters the system just downstream of the study area and has a 1:100 year flow of 22.6 m^3/s. Flooding of this tributary may be of concern, however, the flood elevations were not determined since this was not part of this study.

Salmon Cove River - Salmon Cove:

The 1:100 year flow of 88.3 m^3/s resulted in a pressure flow of 82.3 m^3/s and weir flow of 5 m^3/s at the bridge under study. Backwater from the bridge resulted in an area upstream that was flooded. Flooding also occurred a short distance downstream of the bridge where a tributary enters the system.

Hodges River - Whitbourne:

A problem was found with the 1:2500 mapping for Whitbourne. It was suspected that the contour intervals on the map were marked 2.5 m lower than they should have been. In order to match the surveyed cross sections, the contour intervals on the map were assumed to be 2.5 m higher than shown. When the surveyed cross sections were extended into the floodplain, the new topographic elevations were used.

The 1:100 year flow of 59.2 m^3/s resulted in a pressure flow of 54.2 m^3/s and a weir flow of 5.0 m^3/s at the train trestle. Only pressure flow resulted through the road bridge. Although the opening areas under the road bridge and the train trestle are close in value, the slope is lower under the train trestle. The width of the train trestle was increased from 11.2 m to 12 m in order to get the trapezoidal approximation in the HEC-2 model to equal the actual area. The encroachments at the road bridge were accounted for by modifying the upstream and downstream cross sections since adding encroachment points would have eliminated effective flow areas around the bridge.

Heart's Delight Brook - Heart's Delight:

The 1:100 year flow of 47.3 m^3/s resulted in a 0.44 m below the bridge low chord. Flooding in this study area is most likely a result of high tide levels and ice jams at the outlet.

Brook # 2 - Heart's Delight:

The 1:100 year flow of 3.3 m^3/s resulted in pressure and weir flow at the downstream culvert. The water level did not reach the upstream culvert obvert. The culverts are approximately the same size, but there is a higher gradient on the upstream culvert and the distance from the top of the road to the invert is greater on the upstream culvert than the downstream culvert. The pressure and weir flow at the downstream culvert was 2.7 m^3/s and 0.6 m^3/s, respectively.

Brook # 1 - Heart's Delight:

The area of the arched culverts were approximated by specifying a diameter and using the Special Culvert routine. The 1:100 year flow of 11.2 m^3/s was divided into 4.4 m^3/s of pressure flow and 6.8 m^3/s of weir flow at the culverts. A tributary enters the system close to the study area. The flow from this area was included in the flood flows, however, the water surface elevations for this tributary were not determined since it this was not part of this study.

Western Pond Brook - Winterton:

The width of the furthest downstream bridge was adjusted to account for the approximate 45° skew angle. The 1:100 year flow of 22.3 m^3/s resulted in a water level 1.0 m below the upstream end low chord of the furthest downstream bridge. Pressure flow only resulted at the two other upstream bridges. This brook is very steep resulting in critical depth being crossed several times in the HEC-2 model. Flooding in this brook is not severe.

Halfway Brook - Hant's Harbour:

The 1:100 year flow of 23.1 m^3/s resulted in pressure flow through the bridge under study. The water was backed up considerably behind the bridge causing flooding in the same area where flooding due to ice jams has been reported.

Shorts Brook - Hant's Harbour:

The 1:100 year flow of 17.7 m^3/s resulted in a water level 0.60 m below the low chord of the downstream bridge.

The Split Flow procedure was used to model the multiple differing sized culverts with differing invert elevations on Shorts Brook. Separate models were developed for the culverts. One contained the two smaller circular culverts and the other contained the larger arched culvert which was approximated by an equivalent diameter. The 1:20 and 1:100 year flow was arbitrarily split between these two models in varying proportions and the various resulting water surface elevations upstream of the culverts were determined. A discharge versus water surface elevation rating curve was developed for each model at the upstream cross section (no. 194) from which a total discharge curve was obtained by summing together the discharges for each model at common water surface elevations. The water surface elevations for the 1:20 and 1:100 year flows were then obtained from this curve.

5.5 Sensitivity Analyses

The sensitivity of each hydraulic model was evaluated with respect to discharge, Manning's n and starting water elevations. These parameters have been found to be the most sensitive parameters in many HEC-2 models (Huber and Bedient, 1992). In each case, the parameter of interest was varied while holding all others constant. The 1:100 year flow values were increased and decreased by 10% in each model. A Manning's multiplier factor was used in the models to change the Manning's friction coefficients to plus and minus 20%. The starting water elevations used in the models were increased to the highest ever recorded in St. John's, after correction factors were applied, for those study areas close to the outlets. For the other study areas, the starting elevations calculated with the Manning's equation were increased and decreased by 0.5 m. The sensitivity of the models to ice cover and ice jam conditions was also tested in addition to the models' sensitivity to bridge and culvert structures located within the study reaches.

The relative importance and sensitivity of each model variable was established using the calibrated models and the 1:100 year flow developed as part of this investigation.

5.5.1 Sensitivity to Peak Discharge

The effect of increasing and decreasing the peak 1:100 year discharge on the flood elevations in each study reach is presented in Table 5.5.

TABLE 5.5
Hydraulic Model Sensitivity to Peak Discharge

Study Area	Average Difference (m)	Range for plus 10% (m)	Range for minus 10% (m)
Island Pond Brook - Carbonear	0.15	0.02 to 0.42	0.02 to 0.34
Powell's Brook - Carbonear	0.10	0.01 to 0.46	0.02 to 0.35
Halfway Brook - Hant's Harbour	0.08	0.05 to 0.10	0.05 to 0.10
Shorts Brook - Hant's Harbour	0.07	0.03 to 0.08	0.06 to 0.14
Heart's Delight Brook - Heart's Delight	0.09	0.05 to 0.13	0.07 to 0.12
Brook north of Heart's Delight Brook	0.05	0.03 to 0.07	0.01 to 0.08
Brook south of Heart's Delight Brook	0.06	0.02 to 0.11	0.03 to 0.13
Salmon Cove River - Salmon Cove	0.12	0.06 to 0.20	0.06 to 0.22
Salmon Cove River - Victoria	0.11	0.07 to 0.14	0.08 to 0.14
Hodges River - Whitbourne	0.10	0.01 to 0.19	0.01 to 0.27
Western Pond Brook - Winterton	0.07	0.04 to 0.10	0.04 to 0.10

Notes:
1. The sensitivity to peak discharge was performed using the 100 year flood flows.
2. The average difference includes changes due to both the plus and minus 10% cases and was determined by summing the differences at each cross section and dividing by the number of cross sections.

5.5.2 Sensitivity to Roughness Coefficient

The effects of increasing and decreasing the Manning's friction coefficient (n) values by 20% on the flood elevations for each study area are summarized in Table 5.6.

TABLE 5.6
Hydraulic Model Sensitivity to Manning's n (plus and minus 20%)

Study Area	Average Difference (m)	Range for plus 20% (m)	Range for minus 20% (m)
Island Pond Brook - Carbonear	0.09	0.01 to 0.58	0.00 to 0.44
Powell's Brook - Carbonear	0.08	0.00 to 0.45	0.00 to 0.40
Halfway Brook - Hant's Harbour	0.10	0.00 to 0.12	0.00 to 0.71
Shorts Brook - Hant's Harbour	0.08	0.00 to 0.14	0.00 to 0.15
Heart's Delight Brook - Heart's Delight	0.15	0.00 to 0.19	0.00 to 0.22
Brook # 2 - Heart's Delight	0.02	0.00 to 0.07	0.00 to 0.09
Brook # 1 - Heart's Delight	0.03	0.00 to 0.08	0.00 to 0.08
Salmon Cove River - Salmon Cove	0.09	0.00 to 0.19	0.00 to 0.23
Salmon Cove River - Victoria	0.18	0.04 to 0.25	0.00 to 0.31
Hodges River - Whitbourne	0.09	0.04 to 0.15	0.05 to 0.18
Western Pond Brook - Winterton	0.08	0.00 to 0.18	0.05 to 0.22

Notes:
1. Island Pond Brook - The next highest differences for each case were found to be 0.17 and 0.16 m.
2. Halfway Brook - The difference of 0.71 m occurred at one cross section only. The next highest difference was 0.15 m for the -20% case.
3. Shorts Brook - The sensitivity was tested for the section downstream of the differing multiple culverts due to the complication of having used the Split Flow method to compute flood elevations u/s of the culverts.

5.5.3 Sensitivity to Starting Water Elevations

The effects of increasing and decreasing the starting water elevations in each model are presented in Table 5.7. In many cases, the HEC-2 model would not accept a 0.5 m decrease in starting water elevation because this would result in an elevation below the critical

Table 5.7
Hydraulic Model Sensitivity to Starting Water Elevations

Study Area	Starting Elevations Entered into Model	Effects of Starting Elevations on Upstream Levels
Island Pond Brook - Carbonear	normal: 0.93 m high: 1.60 m low: 0.43 m	 affects 700 m - upstream changes taper off gradually model accepted 0.6 m - no effect past section no. 40
Powell's Brook - Carbonear	normal: 2.49 m high: 2.99 m low: 1.99 m	 no effect past section no. 73 model accepted 2.37 m - no effect past first section
Halfway Brook - Hant's Harbour	normal: 0.79 m high: 1.70 m low: 0.29 m	 no effect past section no. 149 model accepted 0.63 m - no effect past section no. 149
Shorts Brook - Hant's Harbour	normal: 0.44 m high: 1.70 m low: -0.06 m	 no effect past section no. 82 model accepted 0.24 m - no effect past first section
Heart's Delight Brook - Heart's Delight	normal: 1.77 m high: 2.27 m low: 1.27 m	 no effect past section no. 241 - approx. 0.5 m increase until section no. 218 - 0.3 m increase at section 241 no effect past section no. 241
Brook # 2 - Heart's Delight	normal: 0.76 m high: 1.70 m low: 0.26 m	 no effect past section no. 48 model accepted 0.55 m - no effect beyond section no. 48
Brook # 1 - Heart's Delight	normal: 1.60 m high: 2.10 m low: 1.10 m	 no effect beyond section no. 368 model accepted 1.42 m - no effect past section no. 368
Salmon Cove River - Salmon Cove	normal: 6.89 m high: 7.39 m low: 6.39 m	 no effect beyond first cross section model accepted by 6.87 m - no effect past first section
Salmon Cove River - Victoria	normal: 44.70 m high: 45.20 m low: 44.20 m	 no effect beyond section no. 210 no effect beyond section no. 210
Hodges River - Whitbourne	normal: 56.61 m high: 45.20 m low: 56.11 m	 caused gradually tapering elevation increase of 0.5 m at first section to 0.18 m at last section no effect beyond section no. 73
Western Pond Brook - Winterton	normal: 2.52 m high: 3.02 m low: 2.02 m	 no effect beyond section no. 53 no changes - model would not accept any decrease

Notes:
1. The sensitivity to starting water elevations was performed using the 100 year flood flows.
2. In many cases the HEC-2 model would not accept the 0.5 m decrease entered into the models because this would result in a depth lower than the critical depth. When this occurred, the effects of the lowest elevation accepted was analyzed (critical depth).
3. Cross section distances were measured from the outlets or downstream ponds.

depth. The model automatically assumes critical depth given a supercritical elevation and, thus, the model's sensitivity to the critical depth as the starting elevation was tested instead. The elevations along many of the study reaches did not change beyond the first cross section, or soon thereafter, with a decrease or increase in starting water elevations. The exceptions were Island Pond Brook for which 700 m of the study area was affected by the high starting water elevation of 1.60 m, and Hodges River for which the entire study reach was affected by the increase in starting elevation to 57.11 m. In addition, about half of the study reach on Heart's Delight Brook was affected by both the high and low starting water elevations.
Few of the computed elevations for the 1:100 year flows for the study areas were changed in response to the changes made in the starting water elevations.

5.5.4 Ice Jam Analysis

Both ice cover and ice jams have caused flooding during flows with peaks less than the 1:20 and 1:100 year peaks. The extent of this flooding has been documented in several reports over recent years. The purpose of this section is to determine the sensitivity of the flood levels to ice cover and ice jam conditions.

Ice jams are often an integral part of a river ice breakup process and the progression of breakup requires the formation and failure of ice jams at various points. Some sites in a river are prone to regular ice jam formations with little variation in jam location. Often these sites are at locations where there are changes in the water surface profile or conveyance due to channel constrictions, bends, or downstream reservoirs.

In order to determine potential ice jam locations, stabilities of ice covers along the study reaches were estimated. Areas where the ice cover was found to be stable under high flows were judged to be potential ice jam locations since the upstream break up ice may have more of a tendency to jam as it attempts to pass by these locations. The ice cover stability was estimated using stability factors based on criteria developed by Pariset (1966) which have been incorporated into the HEC-2 model. These criteria were found to be suitable for analysis of cohesionless ice covered wide rivers greater than 3.5 m deep (Calkins, 1978). Unfortunately, the reaches under study are not very deep. However, the criteria may be used as a guide to indicate possible ice jam locations within the study reaches. Ice cover stability was assumed if the computed stability factors for the cross sections were close to those values indicating stability.

For all the study reaches, it was found that the maximum flows are most likely to occur when the streams are ice-free (Section 4.4). Flooding associated with ice conditions would occur with flows of lower magnitudes than the design flows for the open water conditions. In February of 1991, heavy rains and snowmelt produced a peak flow on Western Pond Brook that was estimated to have a 25 year reoccurrence interval. Thus, it is possible to have the previously computed peak 1:20 year flows for the study reaches during ice conditions. Since the determination of flows associated with ice conditions was not part of this study, the sensitivity of the HEC-2 models to ice cover and ice jam conditions were tested using the 1:20 year flows. However, it should be kept in mind that these flows are not as likely to happen under ice conditions and, thus, representing a combination of an even higher reoccurrence interval than 1:20 years. The flood elevations computed for this sensitivity analysis should not be taken as representative of typical ice related flood conditions, but represent more of a rare event or "worst case".

Initially, an ice cover thickness of 0.30 m along each study reach (within the channel and floodplain) and the 1:20 year flows were tested with the HEC-2 models to determine if flood elevations due to the these conditions would exceed the 1:100 year flood elevations. The stability factors, computed for each cross section along the reaches, were looked at to determine where ice jams could occur. These are in addition to those areas previously identified. If a likely location was near a culvert or a bridge, the opening area of the structure was reduced by 50% in an attempt to model the effect of an ice jam. If no structure was nearby, the thickness of the ice cover was increased at this location to model the ice jam effect. The sensitivity of the study reaches to ice cover and ice jam conditions could then be evaluated. Testing was done only on those reaches where ice related flooding has been observed in the past.

Observations made during the course of the study support the assumption of an ice cover of 0.30 m. Approximately 0.30 m of ice cover was observed in most of the study reaches, however, less ice cover was observed on Western Pond Brook and Powells Brook while more was observed on Halfway Brook and Salmon Cove River in Victoria. No flooding due to ice jams occurred during this study, but historical floods caused by ice jams support a 50% reduction of the opening area of bridge and culverts.

Island Pond Brook - Carbonear

The ice cover thickness of 0.30 m along with the 1:20 year flow produced flood elevations greater than the 1:100 year elevations by 0.20 to 0.40 m between section no.'s 631 and 1419. The stability factors did not indicate any likely areas for ice jams to occur.

Powells Brook - Carbonear

The flood elevations due to a 0.30 m ice cover and the 1:20 year flow exceeded the 1:100 year elevations by 0.20 m to 0.30 m throughout the reach. The stability factors indicated that ice jams could occur just upstream of the furthest downstream bridge and the furthest upstream bridge in the study area. The areas of these two bridges were reduced by 50% and this further increased the flood elevations by 0.36 m at section no. 40 (at the downstream bridge) to 0.15 m at section no. 73. The reduction of the upstream bridge opening area had little effect on the flood elevations. Flood elevations along Powells Brook do not appear to be greatly sensitive to ice conditions.

Salmon Cove River - Victoria:

The ice jamming that occurs approximately 500 m downstream of the bridge in the study area may be caused by a change in the water surface profile due to the channel becoming narrower and/or because Beaver Pond is located just downstream. The ice jam conditions at this location were modelled by increasing the ice cover thickness at cross section no.'s 210 and 385 from 0.30 m to 0.50 m. The flood elevations resulting from the 1:20 year flow exceeded the 1:100 year elevations by 0.50 m from section no.'s 100 to 478 and by an average of 0.25 m for the remaining sections. The stability factors did not indicate that the bridge was a likely place for an ice jam. Ice conditions appear to have a significant effect on the flood elevations and the extent of flooding downstream of the bridge.

Salmon Cove River - Salmon Cove

The ice cover thickness of 0.30 m along with the 1:20 year flow produced flood elevations greater than the 1:100 year elevations by 0.45 m from section no.'s 308 to 419 and from section no. 523 to 598. The elevations were close to the 1:100 year elevations from section no.'s 431 to 477 and section no. 623. The stability factors did not indicate possible ice jam locations.

Hodges River - Whitbourne

The response of the HEC-2 model to an ice cover thickness of 0.30 m and the 1:20 year flow was to increase flood elevations beyond the 1:100 year flood elevations by 0.25 m from section no.'s 50 to 75, and by 0.10 m to 0.15 m for the remaining sections. The Pariset stability factors indicated possible ice jam locations upstream of both the train trestle and the road bridge. With the opening areas of these reduced by 50%, the flood elevations increased beyond the

1:100 year elevations by 0.60 to 0.70 m from sections no.'s 50 to 209. The elevations were lower than the 1:100 year elevations beyond section 221. The train trestle appears to have more of an effect on the flood elevations during ice conditions than the road bridge, however, neither bridge has a significant effect on the open water flood elevations.

Heart's Delight Brook - Heart's Delight

The flood elevations due to 0.30 m of ice cover and the 1:20 year flow exceeded the 1:100 year elevations by an average of 0.15 m along the study reach. The stability factors indicated that an ice jam could occur at the entrance to the bridge. After reducing the bridge opening area by 50%, the flood elevations exceeded the 1:100 year elevations by 0.83 m at section no. 218 down to 0.16 m at section no. 299. The open water surface profile did not appear to be significantly affected by this bridge, however, ice jamming due to the bridge may be problematic.

Western Pond Brook - Winterton

With an ice cover thickness of 0.30 m, the 1:20 year flow resulted in flood elevations exceeding the 1:100 year flood elevations by 0.10 to 0.20 m throughout the study reach. The Pariset stability factors indicated possible ice jam locations upstream of the two most upstream bridges in the study reach. The opening areas of these were reduced by 50% by moving the abutments inwards and adjusting the encroachments. The resulting flood elevations exceeded the 1:100 year elevations by 1 m at the further downstream bridge (section no. 251), by 0.50 m up to section no. 431 (just upstream of the most upstream bridge), and by 0.10 m at section no. 469. These bridges were previously determined not to have a significant effect on the open water surface profiles, but may be problematic during ice conditions.

Halfway Brook - Hant's Harbour

The ice cover thickness of 0.30 m along with the 1:20 year flow produced flood elevations greater than the 1:100 year elevations by 0.35 m at section no. 48 down to 0.10 m at section no. 199. The flood elevations beyond section no. 199 were lower than the 1:100 year elevations. The stability factors indicated that ice jam conditions could occur anywhere upstream of the bridge within the study reach. The opening area of the bridge was reduced by 50%, but this had little effect on the flood elevations. The area was further reduced to 25% of the existing area which caused the elevation to exceed the 1:100 year elevation at the bridge entrance by 1.0 m and by 0.80 m upstream to the last cross section. This bridge was previously determined to have a fairly significant effect on the upstream open water flood elevations in the area of a few residents and ice jamming appears likely in this area.

Conclusions

A combination of ice cover with a thickness of 0.3 m and the 1:20 year flows will result in the flood elevations exceeding the 1:100 year elevations with open water conditions by approximately 0.2 to 0.4 m along the study reaches. Using the HEC-2 models, it was determined that the communities which are relatively sensitive to ice jams are Winterton, Hant's Harbour, Heart's Delight, and Victoria, but Whitbourne, Salmon Cove, and Carbonear are not. The sections where ice jams resulted in significantly increasing the flood elevations are:

- Winterton - Section between the two upstream bridges;
- Hant's Harbour - Section upstream of bridge on Halfway Brook;
- Heart's Delight - Section upstream of bridge on Heart's Delight Brook; and,
- Victoria - Section 400 m downstream of bridge.

The alternatives for mitigating ice jam problems may include increasing the spans of bridges, widening narrow channel sections, and constructing berms. Care must be taken to ensure that a solution of one problem does not create another in the same location or cause upstream and/or downstream problems. Feasibility studies would be required to adequately address all the factors involved in making such changes to mitigate flooding due to ice jams.

5.5.5 Sensitivity Analysis of Flood Levels to Bridge and Culvert Structures

The sensitivity of the 1:100 year open water floodlevels to the structures in each of the study reaches was determined with the use of the HEC-2 models. The HEC-2 models for the study reaches were run with the 1:100 year flows after increasing the spans of the existing bridges and/or the diameters of existing culverts and making the required adjustments to the encroachments. For those reaches with more than one structure, the dimensions of each structure were increased one at a time and analyzed individually. In some cases, structures were removed from the models.

Enlarging the opening areas of bridges and culverts may significantly reduce upstream flood elevations. This, in turn, may cause downstream flood elevations to increase due to a reduction in upstream attenuation of peak flows. Undersized bridges and culverts will cause flood waters to rise until sufficient weir flow occurs over the roadway. The flood water stored upstream of the structure may be sufficient to cause some attenuation of the peak flow value which would then reduce elevations through and downstream of the structure. This flow attenuation was not considered while computing the flood elevations with the HEC-2 models. Thus, no downstream effects were observed as a result of the enlarged structures in the HEC-2 models. Large decreases in the 1:100 year flood levels as a result of a newly sized structure suggested that the existing structure is undersized and that the area may benefit by its replacement. Recommended sizes of new structures were not provided since this is beyond the scope of this study.

Island Pond Brook - Carbonear:

The span of the furthest downstream bridge in the study area was increased by 2 m in the HEC-2 model and the encroachments were adjusted accordingly. This increased the bridge opening area by approximately 20% and resulted in decreasing the 100 year flood elevations by 0.18 m at cross section no. 41 (just upstream of the bridge), up to 0.26 m at section no. 48, and 0.06 m at section no. 55 (just downstream of the second train trestle). This bridge appears to have a significant impact on the flood elevations, but only for a short distance(up to the second train trestle).

The furthest downstream train trestle was removed from the model. This resulted in decreasing the flood elevations by 0.50 m from the existing entrance section for this trestle to just downstream of the second train trestle. The elevations were reduced by 0.25 m at the entrance to the second train trestle and upstream a short distance.

The second train trestle was also removed from the model. This reduced the flood elevation at section no. 63 (existing entrance section) by 0.50 m and reduced the upstream elevations by 0.20 to 0.30 m until section no. 631 where no changes were observed. The removal of this train trestle would have a significant impact on reducing the flood elevations for a large area upstream.

Cumulative effects on the flood elevations were determined for the case where both train trestles are removed, however, the results did not differ greatly from the elevations reductions resulting from the individual analysis.

The span of the road bridge at the intersection of Pond Side Road and Cross Road was increased by 2 m, and the encroachments adjusted accordingly, which resulted in increasing the opening area by approximately 20%. This resulted in flood level decreases between 0.06 m and 0.14 m for a distance of 100 m upstream of the bridge. These are not significant elevation changes, especially when compared to the elevation decreases resulting from the removal of the downstream train trestle.

The diameters of the three culverts at the C.N.B. Highway were increased by 1 m which lowered the flood levels for approximately 50 m upstream by 0.25 m.

Powells Brook - Carbonear:

The span of the bridge at Lower Southside Road was increased by 2 m, increasing the opening area by 40%. This reduced the flood levels at the bridge entrance and for a distance of 40 m upstream by between 0.08 m and 0.32 m. The flood elevations in this area are not greatly sensitive to this bridge's opening area.

Increasing the span of the train trestle had very little effect on upstream flood levels.

The span of the bridge at Highroad South was extended by 2 m, increasing the opening area by approximately 40%. The upstream flood elevations decreased by 0.39 m at the bridge entrance (section no. 201), and 0.64 m at section no. 206. No changes were observed at section no. 366, however, one may assume that the flood elevations would be decreased along part of the brook between section no.'s 206 and 366. There is only one building located in this area that could benefit from the replacement of this bridge.

The sizes of the culverts near the Carbonear Cinema were increased from an approximated diameter of 1.35 m to 1.85 m. This resulted in significant flood elevation reductions for a distance of over 150 m upstream. The elevation reduction at the entrance to the culverts (section no. 536) was 0.70 m. The elevation reduction tapered off to 0.18 m at section no. 703 and no changes were observed at section no. 803.

No significant elevation reductions were observed after increasing the diameters of the culverts located near NF Power and Penny's Transport. There were no cumulative effects of the structures on the flood levels along the study area.

Salmon Cove River - Victoria:

The span of the bridge in the study area in Victoria was also increased by 4 m, increasing the opening area by almost 25%. The flood elevations did not decrease significantly. The resulting elevation decreases were 0.04 m at section no. 611, 0.16 m at section no. 636, and down to 0.02 m at section no. 701.

Salmon Cove River - Salmon Cove:

The span of the bridge in Salmon Cove was extended by 4 m, increasing the opening area by almost 30%. This resulted in flood elevation decreases of 0.12 m just upstream of the bridge (section no. 431), up to 0.35 m at section no. 438, and tapering off to 0.05 m at section no. 523.

Hodges River - Whitbourne:

The span of the train trestle was extended by 2 m, increasing the opening area by 15%. This resulted in reducing the flood elevations by 0.32 m at the trestle entrance, by 0.10 m for 200 m upstream, and by 0.05 m for the remainder of the upstream study reach. Increasing the span of the road bridge did not alter these elevations significantly. Neither of these structures appear to affect flood levels significantly.

Heart's Delight Brook - Heart's Delight:

Increasing the span of the bridge on Heart's Delight Brook, resulting in an increase in the opening area of over 20%, had no significant effect.

Brook #1 - Heart's Delight:

The effect of the two culverts on the flood elevations were analyzed by increasing their diameters from 0.95 m to 1.2 m. This resulted in an average elevation decrease of 0.10 m for a distance of approximately 50 m upstream.

Brook #2 - Heart's Delight:

Both culverts on this brook were analyzed by increasing the approximated diameters of 1.35 m to 1.7 m. This reduced the flood elevation at the downstream culvert by 0.21 m and by less than 0.04 m for a distance of 20 m upstream. The flood elevations at the upstream culvert were reduced by 0.27 m at the culvert entrance (section no. 157), tapering off to 0.06 m at section no. 203.

Western Pond Brook - Winterton:

All three bridges in the study area on Western Pond Brook were analyzed, individually, by increasing their spans by 2 m and observing the effects on the flood elevations. No elevation decreases resulted until the most upstream bridge was analyzed. This is due to the fact that the flow in the region of the two downstream bridges is critical, or below critical and assumed to be critical in the HEC-2 model, because of the steep gradient.

The span increase of 2 m increased the opening area of the most upstream bridge by 30%. The flood elevation decreases observed were 0.16 m at the bridge (section no. 421), and less than 0.06 m for approximately 20 m upstream.

Halfway Brook - Hant's Harbour:

The span of the bridge on Halfway Brook was increased by 2 m, increasing the opening area by almost 50%. This resulted in elevation decreases of 0.68 m at the bridge entrance (section no. 212), and tapering off to 0.17 m by the last section no. 327. Replacement of this bridge may be beneficial in reducing flood elevations near the upstream residences.

Shorts Brook - Hant's Harbour:

The bridge span on Shorts Brook was increased by 2 m, increasing the opening area by approximately 35%. This resulted in flood elevation decreases of 0.26 m at the bridge entrance (section no. 58), 0.33 m at section no. 62, and down to 0.20 m at section no. 82.
The three multiple sized culverts were analyzed by giving all three culverts a diameter of 2 m. This reduced the flood elevations by 0.48 m at the culvert entrance (section no. 194), 0.40 at section no.'s 199 and 207, and 0.21 m at the last section no. 244.

5.6 Conclusions of Hydraulic Analyses

The HEC-2 model was successfully used to compute the 1:20 and 1:100 year flood profiles along each of the study areas. Each model, except Shorts Brook and Brooks #1 and #2 in Heart's Delight, was successfully calibrated with the crest gauge and velocity measurement data taken in the field. Shorts Brook could not be calibrated because the location of the crest gauge was affected by ocean levels. Brooks #1 and #2 in Heart's Delight could not be calibrated since crest gauges were not set up. In these cases, references with other study reaches were used to select the appropriate model parameters.

The 1:100 year flood profiles were, in general, equally sensitive to variations in the peak discharges and the roughness coefficients. All of the 1:100 year flood profiles, except that for Whitbourne, were affected by changes in starting water elevations for only short distances.

A combination of ice cover with a thickness of 0.30 m and the 1:20 year flood flows will result in the flood levels exceeding the 1:100 year flood elevations under open water conditions by 0.2 to 0.4 m along the various study reaches.

5.7 1:20 and 1:100 Year Floodline Descriptions

For all study areas, flood risk maps have been produced. The 1:2500 scale base mapping was digitized and thematic layering completed. The 1:20, 1:100 and historical floodlines have been added; refer to Figure 5.3 to 5.12. Tables 5.8 to 5.18 present the 1:20 and 1:100 year flood elevations at each cross section along the study reaches. Note that Figure 5.3 - Flood Information Map (Carbonear) is included in Appendix I of the Technical Appendices.

Table 5.8 - Carbonear Flood Elevations (Island Pond Brook) (m MSL)

Section	1:20	1:100	Historical
25	0.58	0.93	0.73
27	0.60	0.92	0.76
40	0.80	1.16	9.98
41	0.98	1.48	1.24
42	0.99	1.48	1.23
43	0.97	1.43	1.21
48	1.26	2.12	1.71
50	1.36	2.12	1.77
54	1.42	2.14	1.76
55	1.32	2.11	1.72
63	1.79	2.73	1.72
67	1.88	2.73	2.31
200	2.17	2.78	2.28
333	2.22	2.80	2.32
340	2.15	2.70	2.41
351	2.30	2.88	2.57
360	2.61	3.23	2.94
396	2.68	3.26	3.01
451	2.99	3.38	3.09
631	4.70	4.80	4.74
691	5.69	5.97	5.85
749	6.60	6.84	6.72
786	6.98	7.15	7.06
876	8.07	8.24	8.15
964	8.69	8.91	8.78
1044	9.24	9.44	9.33
1134	10.80	10.97	10.79
1184	11.31	11.58	11.42
1242	12.03	12.33	12.16
1299	12.34	12.48	12.42
1356	13.53	13.68	13.61

Table 5.9 - Carbonear Flood Elevations (Powells Brook) (m MSL)

Section	1:20	1:100	Historical
33	2.29	2.49	2.39
40	2.49	2.75	2.62
54	3.68	3.98	3.84
62	3.86	4.10	3.98
73	3.89	4.14	4.02
91	4.70	4.99	4.85
96	5.37	5.70	5.53
106	6.14	6.35	6.25
113	6.43	6.69	6.56
125	6.69	6.92	6.80
140	7.21	7.39	7.30
151	7.55	7.71	7.63
179	7.75	7.89	7.82
184	7.81	8.07	7.94
201	8.95	9.35	9.15
206	9.29	9.98	9.64
366	10.70	10.86	10.78
456	12.00	12.18	12.06
481	12.18	12.38	12.06
536	14.75	15.33	12.27
563	14.75	15.33	15.04
613	14.83	15.36	15.10
703	15.16	15.47	15.32
803	16.27	16.43	16.35
921	18.28	18.42	18.35
942	18.46	18.60	18.53
967	18.78	18.93	18.85
993	20.29	20.05	20.17
1013	20.31	20.11	20.21
1091	20.61	20.80	20.70
1131	22.59	22.80	22.69
1163	23.96	24.15	24.05
1189	24.99	25.15	24.07
1217	25.80	26.05	25.92
1247	26.22	26.46	

Table 5.10 - Victoria Flood Elevations (m MSL)

Section	1:20	1:100	Historical
100	44.32	44.70	44.51
105	44.34	44.72	44.53
210	45.07	45.43	45.25
385	46.01	46.40	46.20
478	46.54	46.85	46.69
583	48.49	48.76	48.62
600	48.88	49.13	49.00
611	49.46	49.83	49.64
636	49.76	50.17	49.96
701	50.56	50.83	50.69
751	51.02	51.26	51.14
821	51.74	51.99	51.86
891	52.21	52.48	52.34

Table 5.11 - Salmon Cove Flood Elevations (m MSL)

Section	1:20	1:100	Historical
308	6.68	6.89	6.79
345	7.89	8.25	8.07
388	8.55	8.85	8.70
413	9.01	9.31	9.16
419	8.99	9.22	9.10
431	9.10	9.94	9.52
438	9.58	10.30	9.94
456	9.66	10.29	9.97
477	9.67	10.28	9.98
523	10.11	10.45	10.28
598	10.95	11.27	11.11
623	11.56	11.98	11.77

Table 5.12 - Whitbourne Flood Elevations (m MSL)

Section	1:20	1:100	Historical
50	56.48	56.61	56.54
73	56.58	56.71	56.64
75	56.52	56.58	56.55
85	56.64	57.01	56.82
88	56.81	57.05	56.93
128	56.85	57.07	56.96
185	56.87	57.09	56.98
205	56.87	57.10	56.98
209	56.82	57.04	56.88
221	57.04	57.54	57.29
226	57.14	57.59	57.36
314	57.40	57.74	57.57
434	57.59	57.91	57.75
514	57.67	57.96	57.82

Table 5.13 - Heart's Delight Flood Elevations (Heart's Delight Brook) (m MSL)

Section	1:20	1:100	Historical
193	1.40	1.77	1.58
197	1.44	1.80	1.62
214	1.78	2.19	1.98
218	1.91	2.36	2.13
241	2.30	2.70	2.50
299	3.33	3.60	3.49
326	5.28	5.56	5.42

Table 5.14 - Heart's Delight Flood Elevations (Brook #1) (m MSL)

Section	1:20	1:100	Historical
343	1.47	1.60	1.53
368	1.85	1.96	1.90
403	2.35	2.39	2.37
407	2.58	2.81	2.69
421	3.42	3.60	3.51
425	3.44	3.61	3.52
465	3.38	3.54	3.46

Table 5.15 - Heart's Delight Flood Elevations (Brook #2) (m MSL)

Section	1:20	1:100	Historical
5	0.65	0.76	0.70
48	1.30	1.40	1.35
73	1.70	1.80	1.75
96	2.31	2.39	2.35
99	2.69	2.83	2.76
114	3.33	3.49	3.41
117	3.43	3.54	3.48
135	3.44	3.54	3.49
138	3.44	3.55	3.5
157	4.33	4.65	4.49
160	4.43	4.77	4.60
203	4.63	4.83	4.73

Table 5.16 - Winterton Flood Elevations (m MSL)

Section	1:20	1:100	Historical
53	2.32	2.52	2.42
79	5.24	5.52	5.41
103	5.97	6.33	6.25
108	5.98	6.35	6.24
131	6.41	6.61	7.21
167	8.06	8.33	8.42
227	11.68	11.88	11.72
236	12.10	12.34	12.22
248	12.71	12.97	13.33
251	13.09	13.41	13.26
318	14.63	14.98	14.75
378	16.18	16.40	16.29
404	18.00	18.15	18.07
419	18.53	18.68	18.61
421	18.58	18.82	18.79
428	19.28	19.53	19.47
431	19.67	19.98	19.96
469	20.52	20.83	20.81

Table 5.17 - Hant's Harbour Flood Elevations (Halfway Brook)(m MSL)

Section	1:20	1:100	Historical
48	0.53	0.79	0.33
112	1.19	1.42	0.34
149	1.66	1.93	0.84
194	2.30	2.83	2.55
199	2.41	2.94	2.67
212	3.19	3.97	3.58
217	3.48	3.97	3.73
244	3.56	4.03	3.67
289	3.54	4.01	3.80
327	3.62	4.07	3.88

Table 5.18 - Hant's Harbour Flood Elevations (Short's Brook) (m MSL)

Section	1:20	1:100	Historical
40	0.22	0.44	0.36
45	0.24	0.45	0.34
58	0.71	0.97	0.80
62	0.85	1.15	1.02
82	0.93	1.23	1.08
147	1.83	2.03	1.94
179	2.46	2.69	2.59
184	2.44	2.64	2.53
194	3.96	4.45	2.19
199	4.02	4.49	4.25
207	4.01	4.48	4.24
244	3.98	4.38	4.18

6.0 REMEDIAL MEASURES

6.1 General

The Federal government started the Flood Damage Reduction Program in 1975 to identify flood risk areas and discourage development in these areas. However, since attractive building sites are often located near lakes, rivers or coastal beaches, development has continued. It is important to understand that no floodproofing methods will totally protect property from the effects of severe flooding. The best protection is to avoid development in the floodplain or floodplain fringe; ie., the fringes of a floodplain where flood waters flow are shallow with reduced velocity.

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 characteriatics.

6.1.1 Non-Structual Measures

The most common non-structural measure is the implementation of Land Use Regulations or Development Control. Land use regulations are enacted to control the development of land within the floodplain. These regulations based on floodplain mapping, can be incorporated into municipal plans and zoning bylaws to identify flood risk areas and what type of structures can be built.

Watershed management practises include the management of activities that can increase the magnitude of flooding. These activties include a wide range of land use practises ranging from agriculture and forestry to urban expansion.

Flood forecasting systems provide accurate information on the areal extent of flooding, when it will occur and the depth of water for critical locations. Other potential measures are residential redevelopment, tax policies to discourage development on the floodplain, and warning signs showing past flood heights.

In addition to the non-structural measures discussed above, the development of public awareness about the flooding problem will minimize potential loss. All activity in the floodplain and much of the activity in the drainage area should be interpreted in terms of its potential impact on the flood problem.

6.1.2 Structural Measures

Structural measures such as dykes, channel improvements and diversions are often employed at great expense to control flood waters. In addition to these measures, modifications to bridge and culvert installations can be a cost effective means of changing flood characteristics.

Dykes are embankments built to protect low-lying areas from inundation and are the most commonly uses structures to protect Canadian communities. They alter only high flows of water by restraining entry to the low-lying areas. The degree of protection provided by dykes depends on their height and construction. High water levels increase pressure against the dykes, accelerate their erosion, hasten their saturation and damage due to under-seepage. Any of these incidents can result in dyke failure. The reliability of a dyke system is contingent upon its continued inspection and maintenance.

Channel improvements may include realignment to eliminate oxbows and sharp bends, dredging, removal of debris, installation of weirs or drop structures, and provision of bank protection. Each of these measures contributes to one or more of the following:

- the stabilization of the river course;
- increases in gradient and flow velocity;
- an enlargement of channel capacity;
- the reduction of bank or channel erosion.

A diversion involves the redirection of part or all of a river's flow around a particular location. It offers a reliable and positive degree of flood control through reduction of flow at the flood prone area. The development of a diversion may require the construction of a dam or the installation of inlet and outlet control structures. Excavation of a diversion channel or the improvement of an existing channel is usually necessary. (Environment Canada, 1993).

Floodproofing is any combination of structural or non-structural changes to buildings or utilities that reduce or eliminate damage caused by floods. Preventative floodproofing measures are more economical than corrective ones because they are applied during the construction of the building. However, corrective measures applied to an existing structure are still a viable means of reducing flood damages. Dry floodproofing is preferred by most property owners because the contents of the building are kept dry and there is no need for clean up. Wet floodproofing minimizes potential damage by allowing water into the building; having water inside and outside the building equalizes the water pressure on the walls and floors, and in most cases, results in less structural damage.

Permanent floodproofing measures are more effective in reducing flood damages in areas prone to frequent or flash flooding. Always in place, these meaures should also be considered for any flood prone area, particularly if there is no flood forcasting service or warning system. Basic technique include raising the foundation above the flood level using fill or supports and surrounding the building with floodproof concrete walls or earth berms.

Contingency floodproofing measures are best suited to areas where the depth or risk of flooding is not too great. Basic techniques include the installation of watertight barriers around doors and windows.

Emergency floodproofing measures are most 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 with readily available materials to build dykes or barriers against rising water. 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, floodproofing is only recommended for the 1:100 flood zone.

6.2 Potential Remedial Measures

The non-structural measures presented in Section 6.2.1 are recommended for all communities in the study. Given their nature, no cost estimate has been assigned to the implementation of these measures.

6.2.1 Non-Structural Measures

Floodplain regulations should be implemented to restrict future development and thus reduce the potential for continued increases in flood damages. A two-zone floodway flood-fringe concept is recommended for all communities where zoning regulations would prohibit future development in the high hazard areas. Development may be permitted in the flood-fringe areas, depending on the degree of hazard and the implementation of flood proofing measures to protect these developments.

The most common non-structural measure is the implementation of Land Use Regulations or Development Control. Land Use Regulations can be enacted to control the development of and within the floodplain. These regulations, based on floodplain mapping, should be incorporated into municipal plans and zoning bylaws to identify flood risk areas and what type of structures can be built. These documents should identify the floodplain and/or the flood fringe on a suitable map.

The zoning bylaws should reflect wise use of the floodplain. The construction of structures in the floodway should be limited to private or public works that, by the nature of their service such as marinas or water plants, that cannot be located elsewhere. The construction of these facilities should include the use of floodproofing techniques. Recreational uses within the floodplain are acceptable.

Within the floodplain fringe, the zoning bylaws should require that structures be floodproofed if located within the 1:100 flood zone. The bylaws should established the acceptable floodproofing requirements. Other flood fringe zoning bylaws should control:

- the construction of a building for which the entrance or exit is restricted by flood waters;
- the manufacture, storage disposal and/or consumption of hazardous substances that would pose a health risk if released during a flood event;
- the construction of institutional buildings such as hospitals, nursing homes, and schools for which flooding could pose a significant threat to inhabitants involved in an emergency evacuation;
- the placement of police, fire, ambulance and electrical substation services that may be impaired by flooding or failure of floodproofing.

6.2.2 Structural Measures

Structural remedial measures have been identified for each study reach and a preliminary cost estimate associated with each is given in Table 6.1.

Carbonear (Island Pond Brook)

The effective flow area of the concrete bridge at the outlet of Island Pond Brook has been significantly reduced due to rock/aggregate accumulation. The section of the brook immediately under and downstream of the bridge can be dredged to increase effective flow area.

Removal of the train trestles at the outlet of Carbonear Pond will reduce flood levels in Carbonear Pond as detailed in Section 5.5.5.

Carbonear (Powell's Brook)

The removal and replacement of the culverts located on Powell's Drive adjacent to the Carbonear Cinema; (see Section 5.5.5.) will eliminate flooding of the cinema parking lot and reduce the potential for weir flow over Powell's Drive.

Armour stone protection placed on the river bend immediately upstream of the train trestle will reduce erosion. A rock dyke at this location will contain high flows from overtopping and flooding adjacent residential properties.

Victoria - Salmon Cove River

The removal of large bedrock outcrops in the river adjacent to Mr. Clyde Antle's property will reduce the potential for ice jamming. A dyke constructed from Mr. Antle's property, for a distance of approximately 200 m upstream will contain high flows.

Salmon Cove - Salmon Cove River

Increasing the span and lower chord elevation of the concrete bridge on Riverdale Crescent will increase effective flow area and reduce flood levels as detailed in section 5.5.5.

Whitbourne - Hodges River

A dyke, constructed from the pumphouse to the new road bridge, will contain high flows. Increasing the effective flow area of the existing man-made channel which diverts water from flowing past the pumphouse will improve channel flow.

Heart's Delight

With respect to Brook #1, replacing the existing double culvert installation with two (2) 1200 mm diameter culverts will reduce the potential for flood damage to nearby residences.

With respect to Heart's Delight Brook, armour stone placed perpendicular to the shoreline and west of the brook outlet will serve to collect migrating beach material and reduce the frequency and/or severity of the blockage at the outlet of the brook.

With respect to Brook #2, replacing the existing culvert with a larger diameter structure will reduce the potential for road overtopping.

Winterton - Western Pond Brook

The concrete bridge replacement on Route 80 in 1992 has, by all accounts, eliminated the flooding problem at this location.

Hant's Harbour

With respect to Halfway Brook, a new hydraulic structure designed to pass the 1:100 year flow will reduce potential flooding.

With respect to Shorts Brook, the existing triple culvert installation should be replaced with a structure (s) to pass the 1:100 year flow.

6.3 Cost Estimates

A summary of the estimated construction costs for potential structural remedial measures is given in Table 6.1 while location plans are given in Figure 6.1 to 6.10.

TABLE 6.1 - SUMMARY

SUMMARY OF COSTS ESTIMATES FOR STRUCTURAL REMEDIAL MEASURES

Location	DESCRIPTION	MEASURE OF UNIT	QUANTITY	UNIT PRICE	PRICE	SUBTOTAL	CONTINGENCY @ 25%	ESTIMATED TOTAL CONSTRUCTION COST
CARBONEAR								
Island Pond Brook	Dredging	m3	100	$40.00	$4,000.00			
	Demolition & Removal of 2 Train Trestles	L.S.	UNIT	$30,000.00	$30,000.00	$34,000.00	$8,500.00	$44,880.00
Powells Brook	1. Remove Existing Arch Culverts	L.S.	UNIT	$10,000.00	$10,000.00			
	Supply and Install 2000mm Dia. Culverts	lm	60	$360.00	$21,600.00	$31,600.00	$7,900.00	$41,712.00
	2. Supply and Placement of 0.5 m3 Armour Stone	m3	30	$70.00	$2,100.00			
	Rock Dyke @ 1.0m High and 20.0m Long	m3	40	$0.00	$4,000.00	$6,100.00	$6,100.00	$12,627.00
VICTORIA								
Salmon Cove River	Removal of Rock	m3	15	$150.00	$2,250.00			
	Rock Dyke @ 2.0m High and 200.0m Long	m3	600	$100.00	$60,000.00	$62,250.00	$15,562.50	$82,170.00
WHITBOURNE								
Hodges River	Channel Excavation	m3	150	$40.00	$6,000.00			
	Rock Dyke @ 2.0m High and 200.0m Long	m3	600	$100.00	$60,000.00	$66,000.00	$16,500.00	$87,120.00
HEART'S DELIGHT								
Heart's Delight Brook	Supply & Placement od 1.0 m3 Armour Stone Protection	m3	175	$130.00	$22,750.00	$22,750.00	$5,687.50	$30,030.00
Brook #1	Remove Two(2) Existing 0.95 m Dia Culverts	L.S.	UNIT	$750.00	$750.00			
	Supply and Install Two(2) 1200 mm Dia. Culverts	lm	24	$230.00	$5,520.00			
Brook #2	Remove Existing 1350 mm Dia. Culvert	L.S.	UNIT	$450.00	$450.00			
	Supply and Install 1800 mm Dia. Culvert	lm	16	$325.00	$5,200.00	$11,920.00	$2,980.00	$15,734.40
HANTS HARBOUR								
Halfway Brook	Demolish and Remove Existing Road Bridge	L.S.	UNIT	$10,000.00	$10,000.00			
	Construct New Bridge	L.S.	UNIT	$30,000.00	$30,000.00	$40,000.00	$10,000.00	$52,800.00
Shorts Brook	Remove Existing Culverts	L.S.	UNIT	$1,250.00	$1,250.00			
	Supply and Install Three(3) 2000 mm Dia. Culverts	lm	18	$360.00	$6,480.00	$7,730.00	$1,932.50	$10,203.60

NOTE: 1. Applicable taxes extra.

FIGURE 6.1 - ISLAND POND BROOK (CARBONEAR) REMEDIAL MEASURES

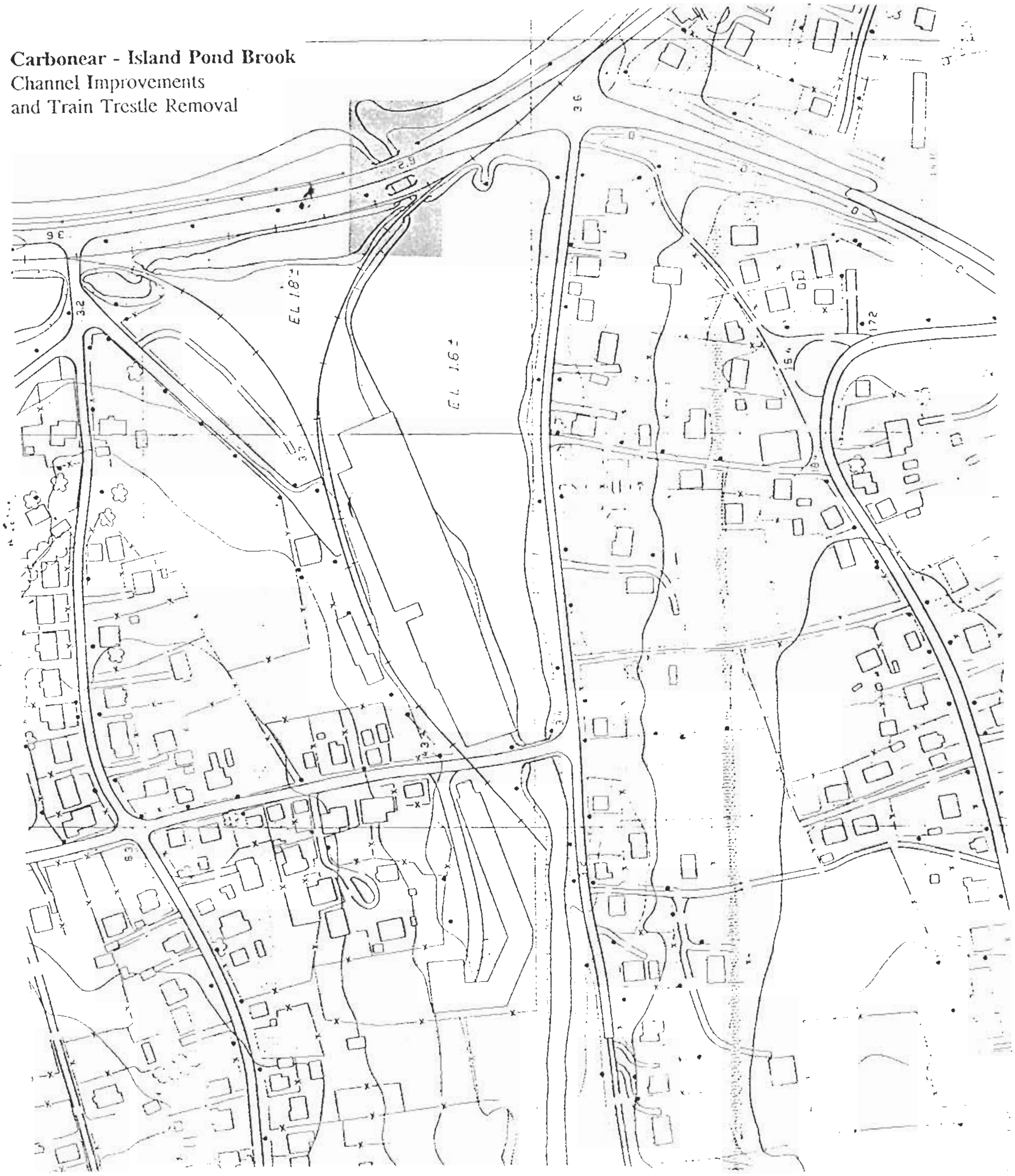


FIGURE 6.2 - POWELLS BROOK (CARBONEAR) REMEDIAL MEASURES

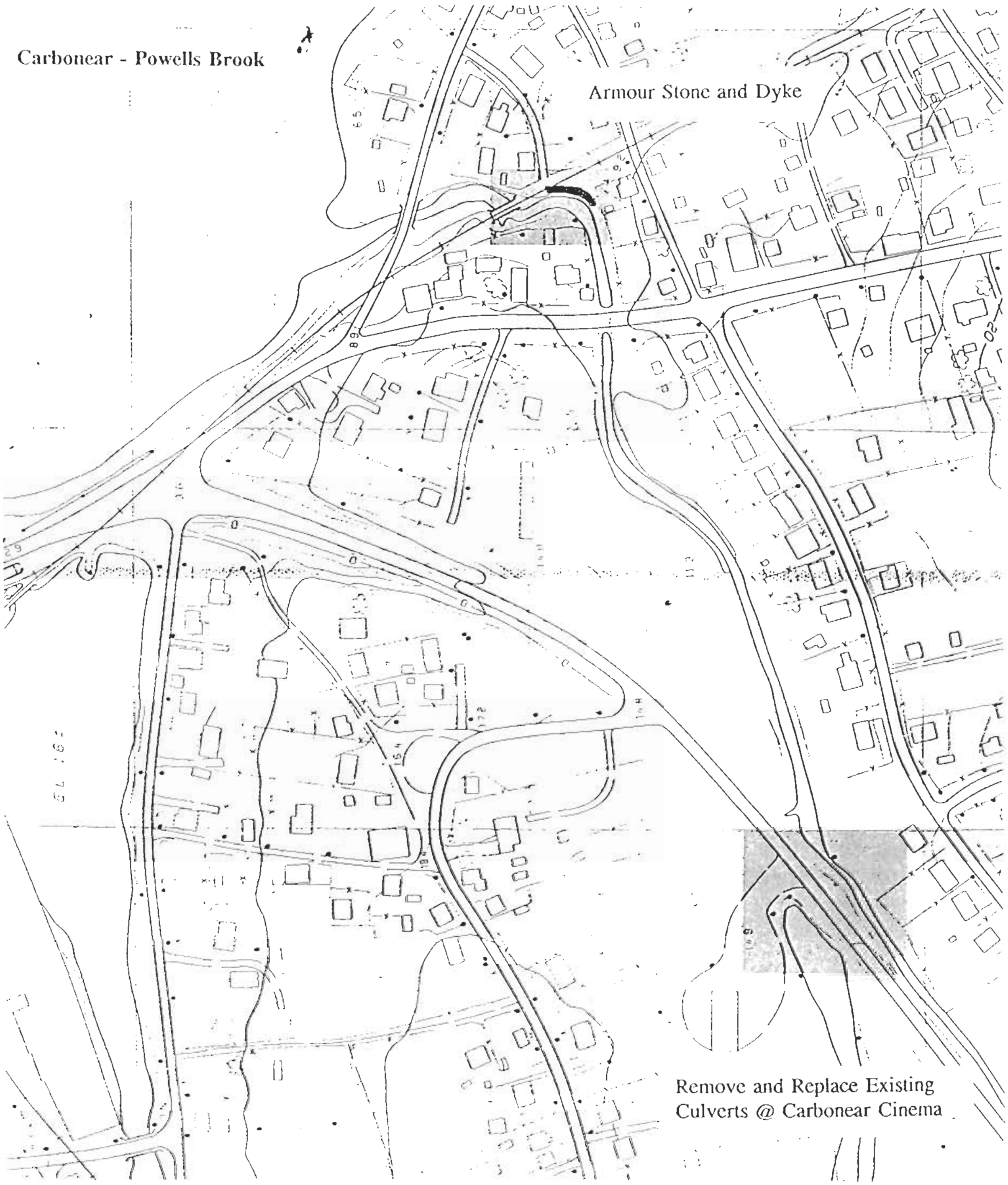


FIGURE 6.3 - SALMON COVE RIVER (VICTORIA) REMEDIAL MEASURES

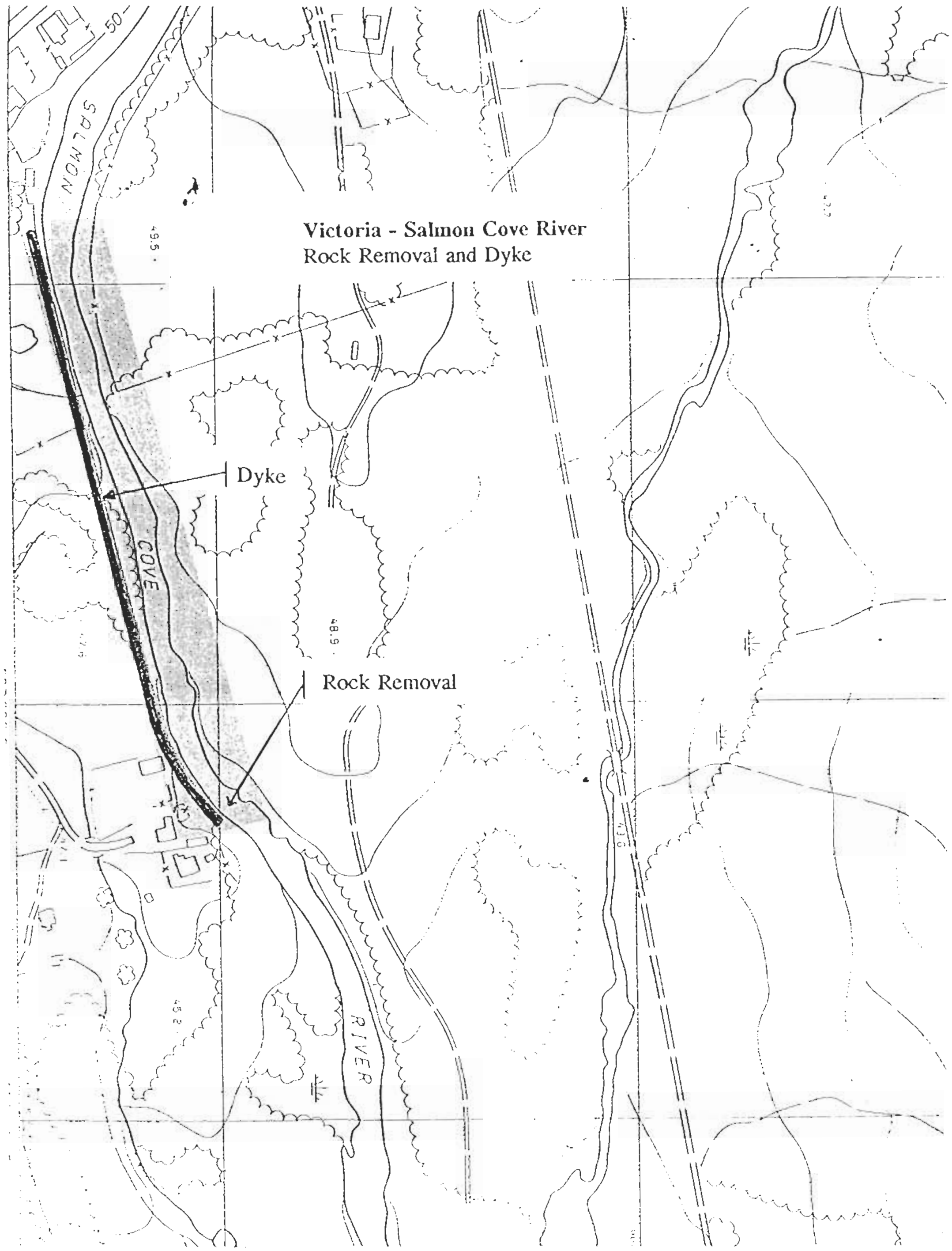


FIGURE 6.4 - SALMON COVE RIVER (SALMON COVE) REMEDIAL MEASURES

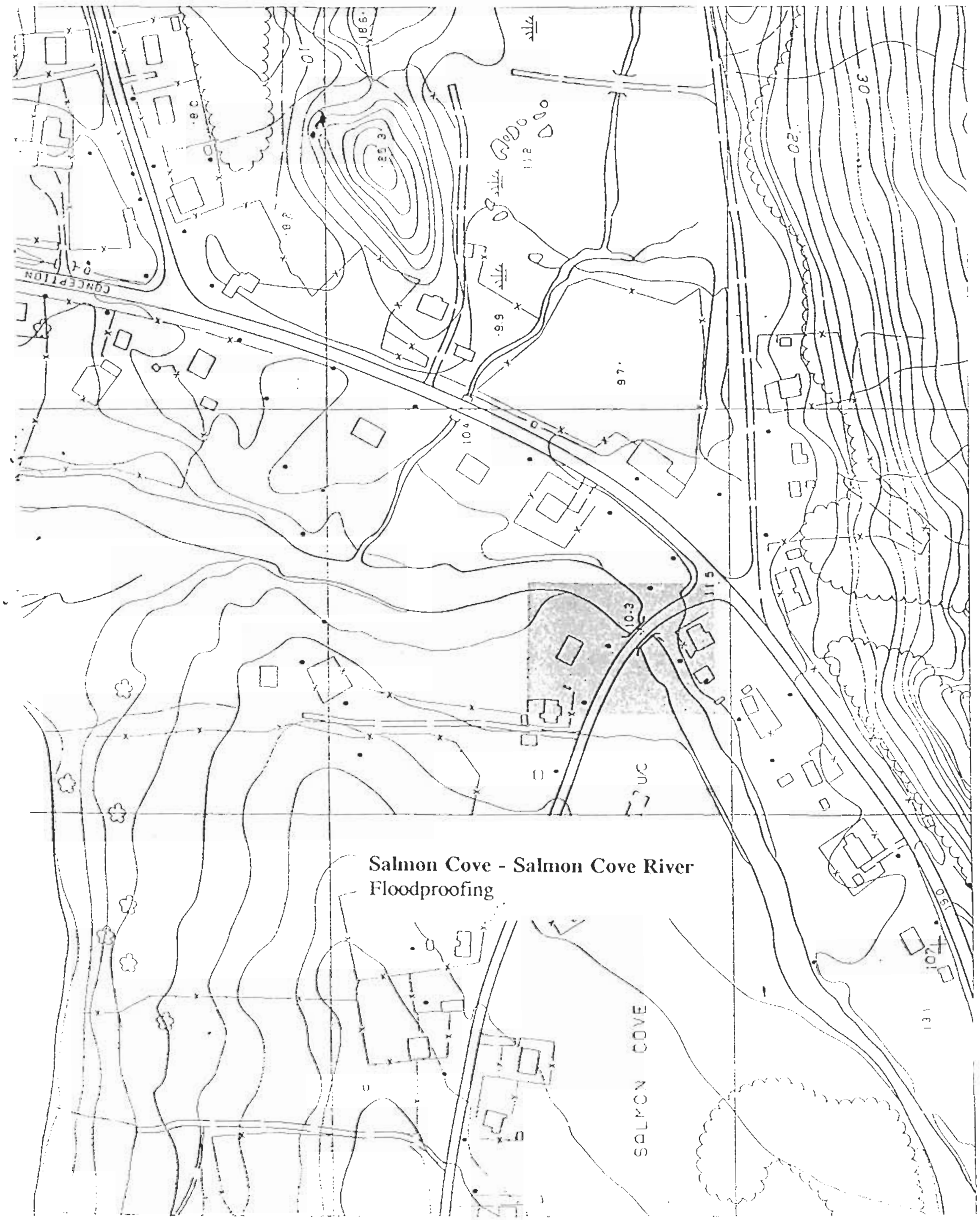


FIGURE 6.5 - HODGES RIVER (WHITBOURNE) REMEDIAL MEASURES

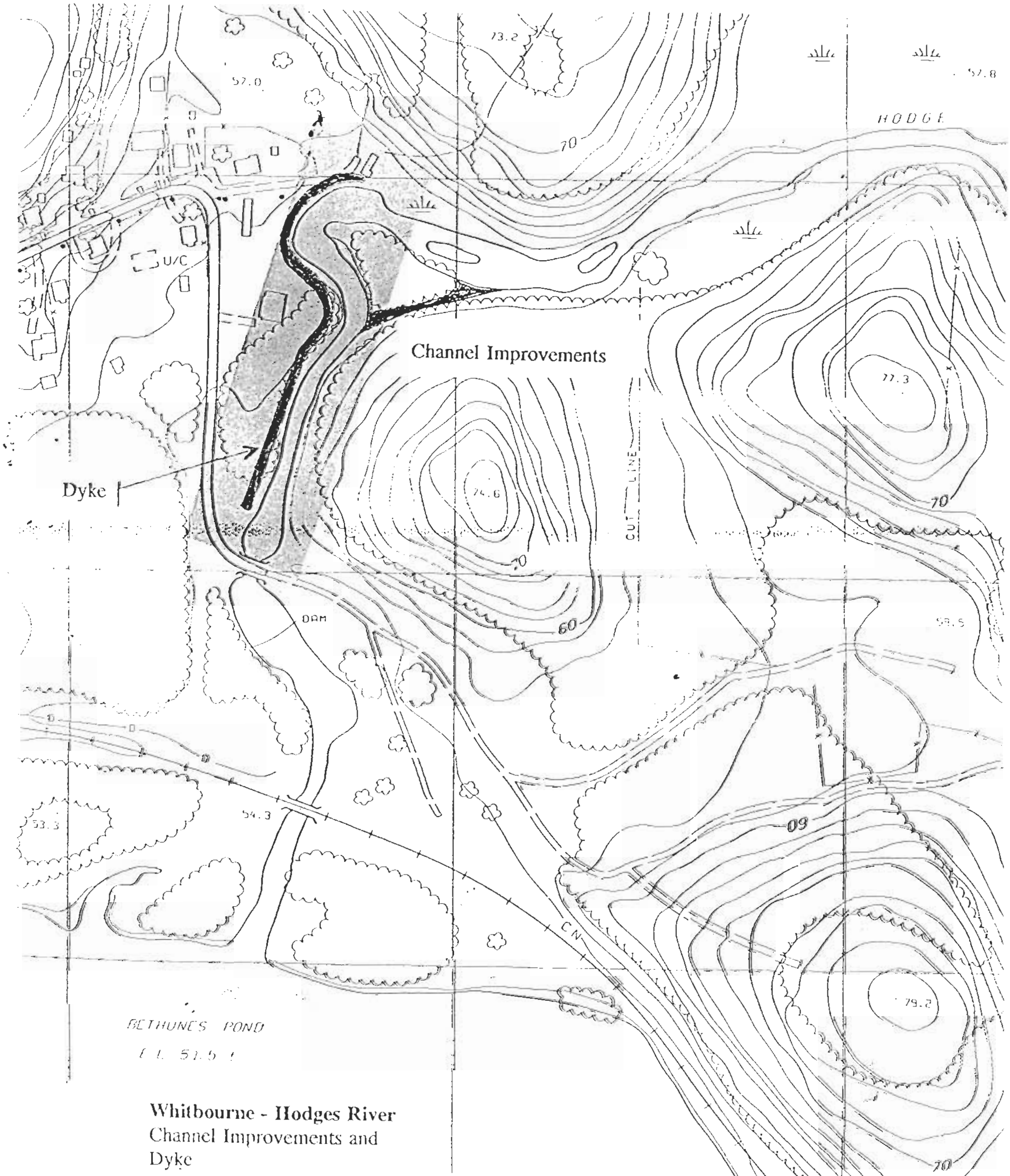


FIGURE 6.6 - HEARTS DELIGHT BROOK (HEARTS DELIGHT) REMEDIAL MEASURES

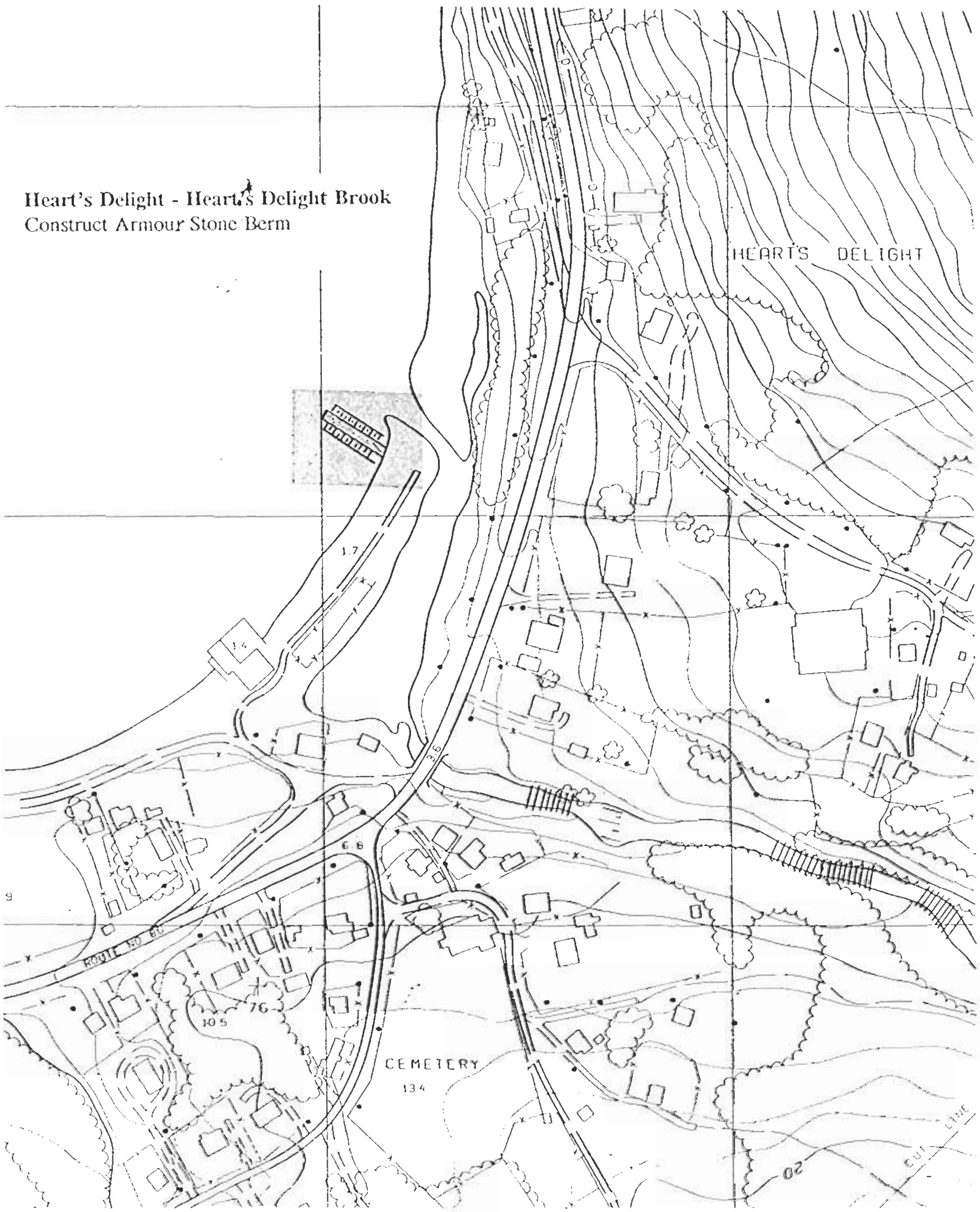


FIGURE 6.7 - BROOK #1 (HEARTS DELIGHT) REMEDIAL MEASURES

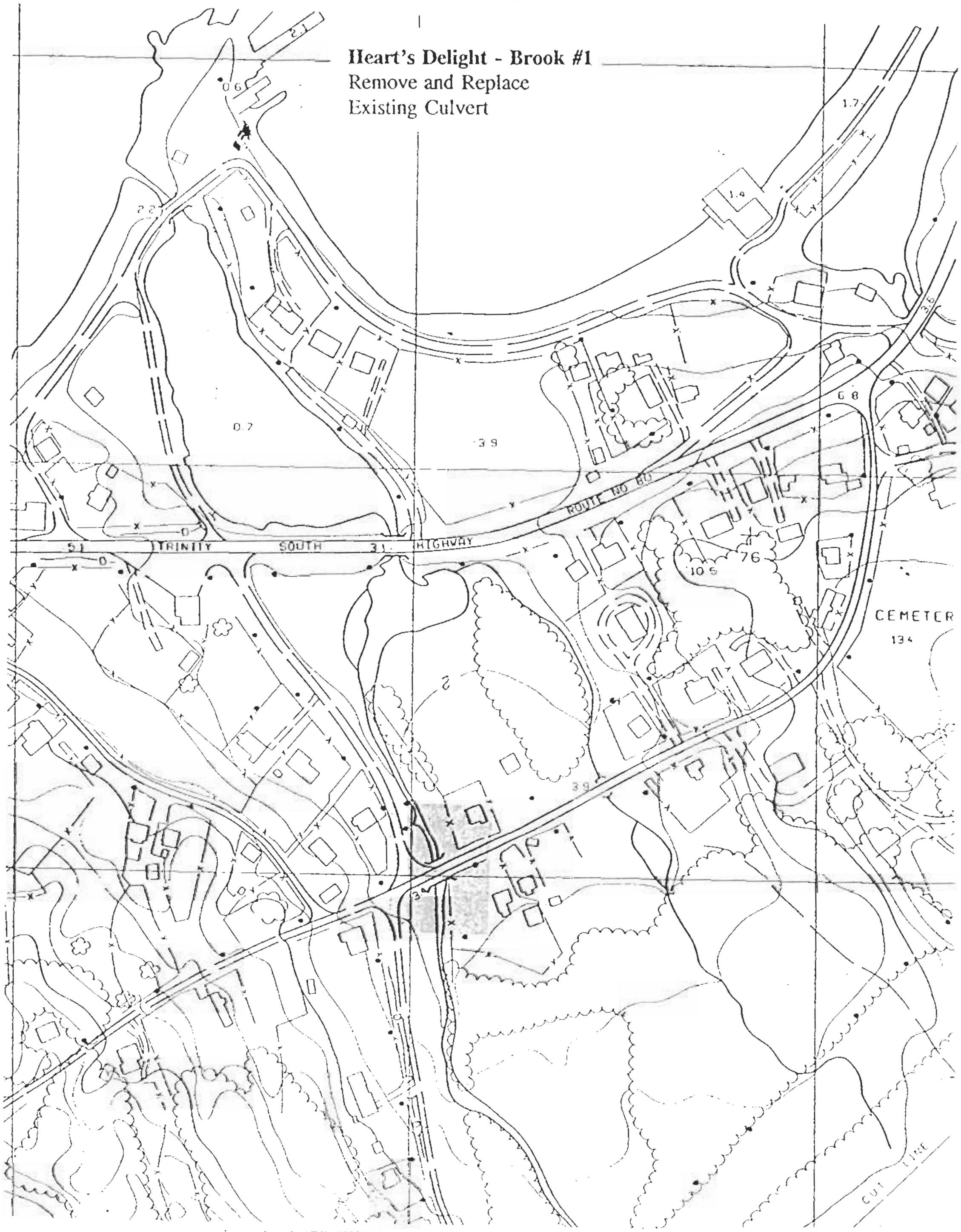


FIGURE 6.8 - BROOK #2 (HEARTS DELIGHT) REMEDIAL MEASURES

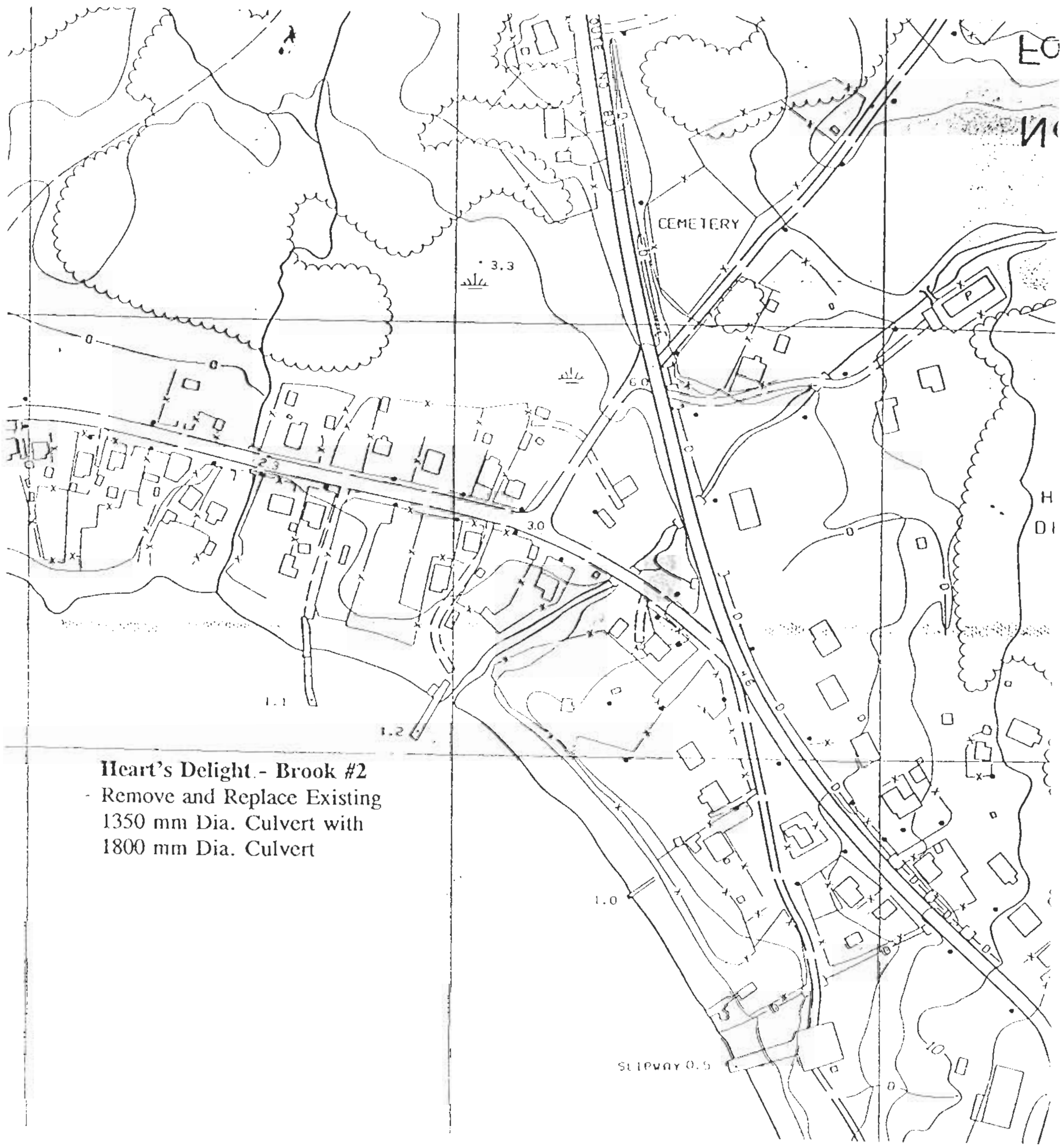


FIGURE 6.9-HALFWAY BROOK (HANT'S HARBOUR) REMEDIAL MEASURES

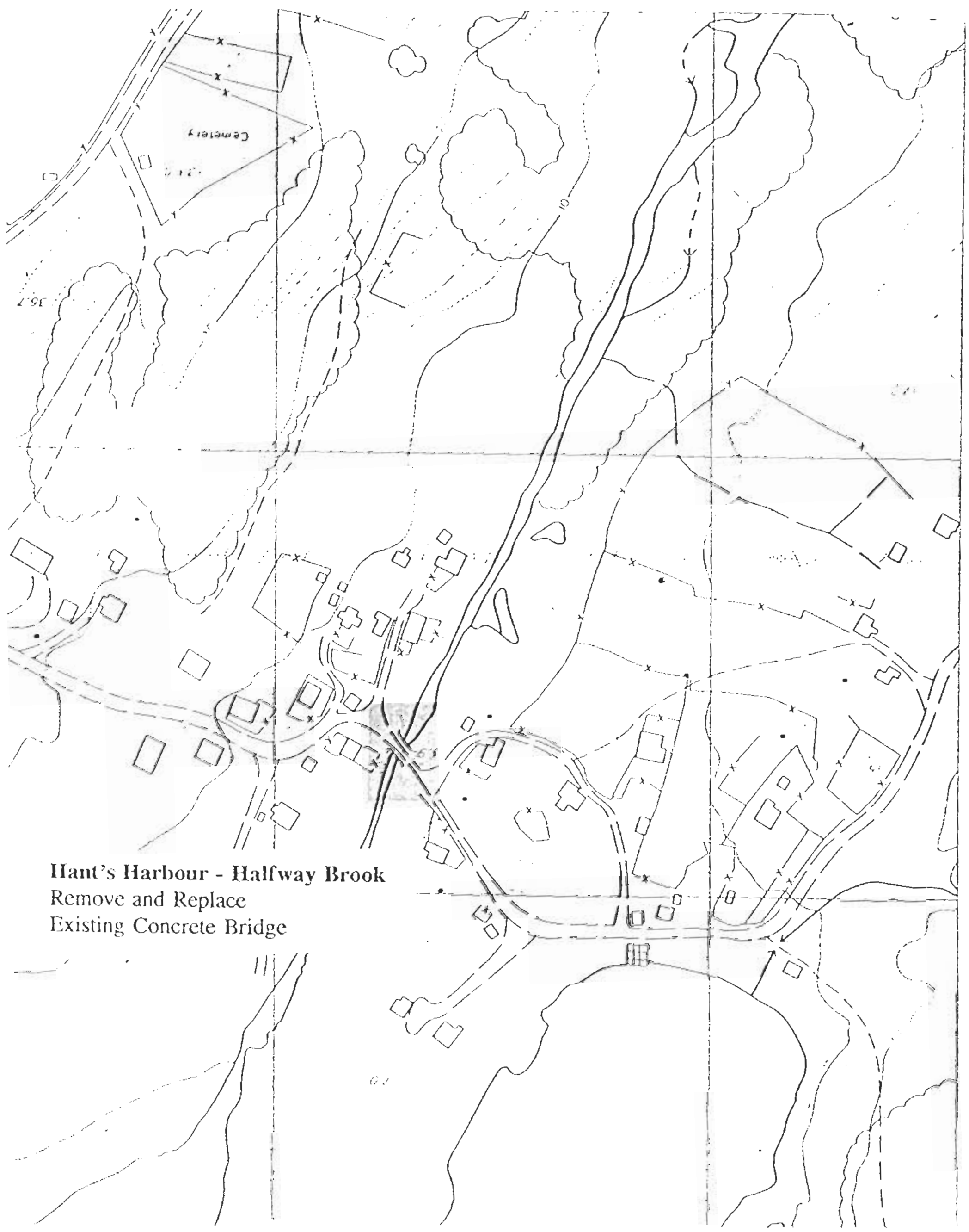
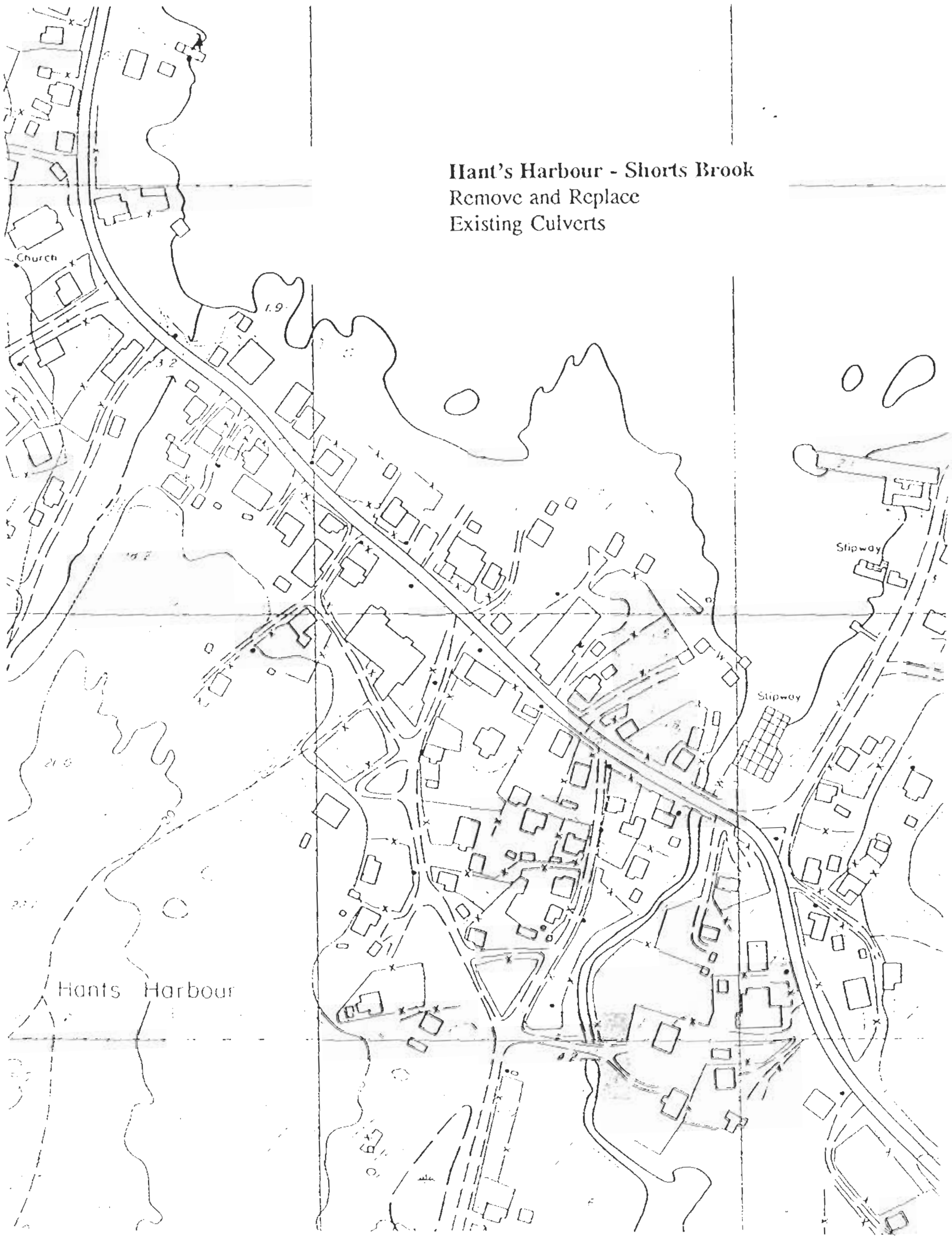


FIGURE 6.10-SHORT'S BROOK (HANT'S HARBOUR) REMEDIAL MEASURES



6.4 Recommended Remedial Measures

In areas of potential future development, regulations should be implemented to restrict development and reduce the potential for continued increase in flood damages. A two-zone floodway - flood fringe concept is recommended that would prohibit future development in high hazard areas. However, depending on the degree of hazard and the implementation of floodproofing measures, development may be permitted in floodplan fringe areas.

Within the floodplain fringe, zoning by-laws should be implemented to ensure that structures within the 1:100 flood zone are floodproofed. The bylaws should establish the floodproofing requirements.

An annual program to remove debris from sensitive hydraulic structures and/or sections of rivers subject to ice/debris jams should be implemented throughut the study area.

The section of Island Pond Brook downstream of the concrete bridge at the outlet of Carbonear Pond should be monitored and dredged on a regular basis to ensure adequate effective flow area.

The train trestles at the outlet of Carbonear Pond should be removed.

The culverts on Powell's Drive adjacent to Carbonear Cinema should be replaced.

Armour stone protection should be placed on the river bend immediately upstream of the train trestle on Powell's Brook.

The bedrock outcrops in Salmon Cove River (Victoria) adjacent to the Antle property should be removed.

The Penny residence adjacent to the concrete bridge on Salmon Cove River (Salmon Cove) should be floodproofed.

The Town of Whitbourne's water supply pump house should be floodproofed through the construction of a concrete or earth berm. Consideration should be given to increasing the elevation of this structure.

The Philip residence (Whitbourne) should be floodproofed by the construction of berms or fill.

The culverts on Brook #1 and Brook #2 in Heart's Delight should be replaced.

The Mercer residence located near the outlet of Heart's Delight Brook should be floodproofed by increasing the elevation of the structure.

The culverts on Shorts Brook (Hant's Harbour) should be replaced.

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