South Nation River Conservation Authority

Environment Canada / Environnement Canada

Ministry of Natural Resources / Ministère des Richesses naturelles
Ontario

CANADA/ONTARIO FLOOD DAMAGE REDUCTION PROGRAM

Flood Plain Mapping
Bear Brook
Township of Cumberland

EGA

ECOS GARATECH ASSOCIATES LTD.
Consulting Engineers

EGA

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File No.: 8520-15-011

December 23, 1991

South Nation River
Conservation Authority
15 Union Street
Berwick, Ontario
K0C 1G0

Attention: Mr. D. O'Grady
General Manager

Reference: Bear Brook
Flood Plain Mapping Study

Gentlemen:

We are pleased to submit herewith our Report documenting the hydrologic and hydraulic analyses undertaken for the Bear Brook Watershed in the Township of Cumberland.

The Report and Flood Risk Maps define the areas along Bear Brook, Elian Reginbald Drain, Bearbrook Drain, McWilliams Drain, Shaws Creek and McKinnons Creek which are subject to flooding under the Regulatory (100 year) flood.

The Report also presents hydrotechnical assessments of the applicability of adopting the Two-Zone Concept along Bear Brook.

We trust that the results contained in this Report will assist the Authority in establishing a comprehensive water management program for the Bear Brook Watershed and we would be pleased to assist and participate in the implementation and compilation of these programs.

Yours very truly,

EGA Consultants

REGISTERED PROFESSIONAL ENGINEER
P. S. H. LIM
PROVINCE OF ONTARIO

Peter S.H. Lim, P.Eng.
Project Engineer

PSHL/l

Encl.

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CANADA/ONTARIO FLOOD DAMAGE REDUCTION PROGRAM

FLOOD PLAIN MAPPING

BEAR BROOK

TOWNSHIP OF CUMBERLAND

DECEMBER 1991

Prepared by:

EGA CONSULTANTS
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SUMMARY

In June of 1987, the South Nation River Conservation Authority retained EGA Consultants (Ecos Garatech Associates Ltd.) to undertake a flood and fill line delineation, as well as an assessment of the applicability of adopting the Two-Zone Concept along Bear Brook within the Township of Cumberland.

Bear Brook originates in the City of Gloucester, and flows in an easterly direction to its outlet at the South Nation River near Ettyville. This study deals with the portion of the watercourse and five of its tributaries located within the Township of Cumberland. The first cross-section (Chainage 0.0) along the main channel commenced in Clarence Township, about 1500 metres downstream of the Cumberland Township boundary. The tributaries include Elian Reginbald Drain and its tributaries, Bearbrook Drain, McWilliams Drain and its tributaries, Shaws Creek, and McKinnons Creek and its tributaries.

The hydrologic and hydraulic systems have been analyzed with the aid of the CFA, VUH, HYMO and HEC-2 computer programs. The Regulatory flood (100 year) and fill lines have been plotted on the Conservation Authority's twelve Flood Risk Maps. All land lying within the flood and fill line delineation is considered to be susceptible to flooding and subject to erosion and potential slope failure. It therefore has been recommended that the Conservation Authority, in cooperation with the Ministry of Natural Resources and Cumberland Township, prepare Official Plan Policies and Zoning By-Laws covering the regulation of Bear Brook and its tributaries within Cumberland Township, in accordance with the Provincial objectives of water management.

There are only 29 buildings located within the Regulatory flood plain of the Bear Brook and its Tributaries. The maximum depth of flooding is 0.7 metres suggesting that a flood proofing program could eliminate the majority of flood damages to existing buildings subject to flooding under the 100 year flood event.

Based on the findings of the hydrotechnical assessments and evaluations of the applicability of adopting the Two-Zone Concept in three areas identified by the Project Team, it is recommended that the Conservation Authority not consider the implementation of the Two-Zone Concept along Bear Brook in the Township of Cumberland, at least until further more detailed hydrologic and hydraulic analysis have been completed.

PREFACE

Under the Terms of Reference established by the South Nation River Conservation Authority, this report documents the findings and conclusions of the hydrologic and hydraulic analyses, and the review of the Two-Zone Concept undertaken for Bear Brook within the Township of Cumberland.

ACKNOWLEDGEMENTS

EGA Consultants is grateful to the following organizations for their support and co-operation throughout the course of the study:

South Nation River Conservation Authority

Ontario Ministry of Natural Resources (Eastern Region)

Water Planning and Management Branch, Environment Canada

Atmospheric Environment Service, Environment Canada

Water Resources Branch, Environment Canada

Township of Cumberland

McNeely Engineering

Ontario Institute of Pedology

STUDY TEAM MEMBERS

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1.0 INTRODUCTION

1.1 STUDY OBJECTIVE

In view of the potential encroachment along Bear Brook, the South Nation River Conservation Authority retained EGA Consultants (Ecos Garatech Associates Ltd.) to undertake a Flood Plain Mapping Study of Bear Brook and its major tributaries (Elian Reginbald Drain, Bearbrook Drain, McWilliams Drain, Shaws Creek and McKinnons Creek) located within the Township of Cumberland.

The principal objective of the study was the delineation of the Regulatory flood plain and associated fill line for approximately 85.3 km along Bear Brook and its major tributaries. To accomplish this objective, detailed hydrotechnical analyses were undertaken.

In addition, the applicability of adopting the Two-Zone Concept at specific locations along Bear Brook was evaluated.

1.2 BACKGROUND INFORMATION

1.2.1 Watershed Description

The Bear Brook watershed (see Figure 3.1) drains an area of about 458 sq km. The watershed, located within the Regional Municipality of Ottawa-Carleton and Counties of Prescott and Russell, has its origin in the City of Gloucester, just south of the City of Ottawa. The main channel meanders in an easterly direction to discharge into the South Nation River near Ettyville. Numerous creeks and municipal drains, like Black Creek, North Indian Creek, South Indian Creek, Elian Reginbald Drain, Bearbrook Drain, McWilliams Drain, Shaws Creek and McKinnons Creek, to name a few, flow either northerly or southerly from both sides to meet Bear Brook. From west to east, the watershed measures some 43 km across. The total length of the main channel extends some 73 km, with a vertical fall of about 31.5 m, producing an equivalent slope of 0.00043 m/m. Negative slopes are evident in some areas of the main channel.

The basin, which forms part of the South Nation River system, is bounded to the north by the Ottawa River system, to the south by the Castor River system, to the east by the South Nation River watershed and to the west by the Rideau River system. There are no large towns in the watershed. The main centres include Bourget, Vars, Limoges, Ettyville, Hammond, Cheney, Bearbrook, Sarsfield, Navan, Notre-Dame-des-Champs and Carlsbad Springs.

The watershed lies within the physiographic region of the Ottawa Valley Clay Plains. East of Ottawa, the area is floored with clay and silt, and bordered by sand plains. An elevated clay plain is evident around Sarsfield and Chartrand. In South Plantagenet, Clarence and Cumberland townships, the main valley is occupied by the South Nation River and Bear Brook. In the City of Gloucester the area is drained by Green's Creek. These streams do not provide complete drainage of the valley, and consequently it is occupied by the Mer Bleue bog.

The soils in the watershed consists of fine sandy loam, silty loam, loam, clay, clay loam, sandy clay, silty clay, fine sand and sand, which were deposited by the Champlain Sea.

The land uses in the Bear Brook watershed consist of forested and agricultural lands.

1.2.2 Study Area

The extent of the study included the main channel of Bear Brook and five of its tributaries located within the Cumberland Township boundary. The hydraulic models were constructed for approximately 24.7 km along Bear Brook, 15.2 km along Elian Reginbald Drain, 3.4 km along Bearbrook Drain, 10.1 km along McWilliams Drain, 10.0 km along Shaws Creek and 21.8 km along McKinnons Creek.

Along Bear Brook, the first cross-section (Chainage 0.0) was located in Lot 21, Concession 10 in Clarence Township, about 1500 metres downstream of the Cumberland Township boundary. The uppermost cross-section was located about 40 metres upstream of the Township boundary.

1.2.3 Previous Investigations

In 1978, Crysler & Lathem Ltd. completed a flood plain mapping study of Bear Brook for the Conservation Authority. The report entitled "Bear Brook Floodplain Study" was prepared.

1.3 STUDY PROCEDURES

The Flood Plain Mapping Study of Bear Brook within the Township of Cumberland generally followed the procedures as described below.

1.3.1 Data Collection

During the initial stages of the study, pertinent background information, pertaining to plans, reports, topographic maps, and drawings were obtained from the following organizations:

(1) The Conservation Authority provided information relating to reports, flood plain maps, drawings and air photographs.

(2) The Township of Cumberland provided information pertaining to Official Land Use Plan.

(3) McNeely Engineering provided drawings of Municipal Drains.

The background information was reviewed, and, wherever applicable, utilized in the study.

1.3.2 Field Surveys

During the months of August and November of 1987, EGA Consultants conducted the following field surveys and reconnaissance:

(1) Ground photography and reconnaissance to assist in the analysis of the river systems pertaining to valley sections and hydraulic friction values through various channel reaches.

(2) Field surveys to supplement the digital cross-sections.

(3) Field surveys to determine the dimensions of all structures crossing the watercourses.

(4) Field surveys to verify the horizontal and vertical accuracy of the mapping.

The fairdrawn mapping and the photogrammetric photo interpretation of digital cross-sections were completed in February of 1988 by Airmap Limited.

The results of the field surveys to verify the accuracy of the mapping are described in the Report on Inspection of Horizontal and Vertical Accuracy for Selected 1:5000 Scale Mapping, completed by EGA Consultants in December 1987, and the Report was subsequently approved by the Department of Energy, Mines and Resources.

1.3.3 Flood Plain Mapping

The following methodologies were used to determine the peak flood flows for the various return frequency events:

(1) Synthetic Unit Hydrograph Method and Single Station Frequency Analysis.

(2) Delisle River Regional Flood Frequency Method.

(3) Regional Regression Method.

(4) Watershed Classification Method.

(5) Index Flood Method.

The results of the five methodologies were compared and the peak flows were subsequently selected and utilized in the hydraulic component of the study.

In order to be consistent with the 100 year water surface profile computed by Crysler & Lathem Ltd. in 1978, the slope-area method in the HEC-2 program was used to establish the starting water level elevation.

Water surface profiles were generated for the various flood events for Bear Brook, Elian Reginbald Drain, Bearbrook Drain, McWilliams Drain, Shaws Creek and McKinnons Creek hydraulic regimes.

The results of the hydrologic and hydraulic analyses, and the methodologies employed were subsequently approved by the Project Team. The Project Team comprised representatives from the Conservation Authority, the Ministry of Natural Resources (Eastern Region) and Environment Canada.

The flood plain resulting from the Regulatory flood (100 year) and the corresponding fill lines were plotted on the South Nation River Conservation Authority's twelve Flood Risk Maps.

1.3.4 Two-Zone Concept

Upon the delineation of the Regulatory flood plain, the Project Team instructed EGA Consultants to plot, where possible, the 1-metre, 2-metre and 3-metre depths of flooding on the twelve map sheets. The Project Team identified three areas for the assessment of the applicability of adopting the Two-Zone Concept by EGA Consultants. The three areas covered Map Sheets 62-24, 66-24, 62-28 and 66-28.

2.0 FLOOD PLAIN MANAGEMENT IN ONTARIO

2.1 GENERAL

In 1979 the Cabinet adopted Flood Plain Criteria and Policies for the purpose of implementing flood plain management throughout the Province. In 1988, the Province's Flood Plain Planning Policy Statement was issued under the Planning Act.

The stated objectives of these policies are:

(1) To prevent loss of life.

(2) To minimize property damage and social disruption.

(3) To encourage a coordinated approach to the use of land and management of water.

The first step in implementing the policies is the delineation of the Regulatory floodlines along waterways. In the case of the South Nation River Conservation Authority (within the Township of Cumberland), the magnitude and extent of the Regulatory flood is based on the 100 year flood.

Strict application of the Policy of no development in the Regulatory flood plain may be unduly restrictive to some communities, especially those with the majority of their existing development falling within the flood plain. The Provincial Policies therefore, provides the flexibility in recognizing special cases whereby, a "Two-Zone" approach or a "Special Policy" designation can be considered to permit controlled development within designated boundaries of the Regulatory flood plain.

2.2 PROVINCIAL ROLE

The Ministry of Natural Resources is responsible for providing the Policies, technical guidelines and financial assistance to the Conservation Authorities for the determination and delineation of flood plain lands.

The Ministry of Housing in cooperation with the Ministry of Natural Resources, is responsible for providing Policies and Procedures for the administration of the Planning Act, as it pertains to flood plain lands in order to comply with the Provincial objectives of flood plain management.

The Ministries of Natural Resources and Housing, in cooperation with the Conservation Authorities are responsible for the development of technical criteria and procedures for the selective application of the Two-Zone Floodway - Flood Fringe concept and for the designation of Special Policy areas.

The Ministries of Natural Resources, Housing and the Environment, in cooperation with the Conservation Authorities are responsible for the development of policies, technical guidelines and procedures for the preparation and approval of storm water management plans.

In addition to providing policies, criteria, procedures and guidance, the Ministries, where applicable, are responsible for the review and approval of requests, recommendations and proposals put forth by the Conservation Authorities and/or the Municipalities.

2.3 CONSERVATION AUTHORITIES ROLE

The Conservation Authorities are corporate bodies created under the Conservation Authorities Act by their constituent Municipalities.

The Authorities are responsible for the continued administration and implementation of the fill and construction regulations in accordance with the Conservation Authorities Act. This requires that the Authorities undertake studies in accordance with Provincial criteria, procedures and guidelines to define the extent of hazard lands within the Authorities jurisdictions.

For those areas where a municipality has prepared and adopted Official Plan Policies and Zoning By-Laws which comply with Provincial flood plain management objectives, the local Conservation Authority has the option of rescinding its construction regulations.

Aside from adopting and implementing regulations of a single zone Regulatory standard, whereby development is not permitted in the Regulatory flood plain, the Authority in cooperation with the Municipality may consider the following:

(1) <u>Two-Zone Floodway - Flood Fringe Concept</u>

The Authorities in cooperation with the Municipalities may apply this concept selectively, if appropriate, and should notify the Ministries of Natural Resources and Housing of the areas where this concept is to be applied, to ensure a coordinated approach during the plan review process.

(2) <u>Special Policy Status</u>

Special Policy status of selective areas must originate from a Municipality and take the form of an Official Plan Amendment. The proposal, if appropriate, would be recommended by the Conservation Authority, after assessing the impact of the proposal on upstream and downstream properties. Any request for Special Policy Status also requires approval of the Ministries of Natural Resources and Housing in consultation with the Ministry of the Environment.

The Conservation Authorities will delegate jurisdiction for urban storm water management over watersheds, regardless of size, to the Municipalities once the Municipalities have adopted storm water management plans which comply with the Provincial objectives of flood plain management, and in accordance with the procedural policies established by the Conservation Authorities and the Ministries of Natural Resources, Housing and the Environment.

The responsibilities of the Conservation Authorities, outlined previously, deal with the implementation of Provincial Policies as they pertain to flood plain management. It should not be overlooked that the Conservation Authorities are responsible for many other programs in the vast field of Conservation management, regulation and education, not to mention the corporate administration and financial responsibility to member Municipalities.

2.4 MUNICIPALITIES/TOWNSHIPS ROLE

The role of the Municipalities/Townships is of extreme importance. Not only are the Municipalities/Townships responsible for the birth of an Authority and the selection of members to the various Authorities' Boards thereby, setting Authority policies and objectives, but they are also financially supportive of the various conservation programs.

The Municipalities/Townships in consultation with the Authorities prepare Official Plan policies and Zoning By-Laws covering the regulation of flood plain lands in accordance with Provincial objectives of flood plain management.

As outlined in Section 2.3, the Municipalities/Townships in conjunction with the Authorities may consider, if applicable, a "Two-Zone" approach. The application may be made selectively and would require joint approval. In the case of the "Special Policy Status" of selective areas, the Municipalities/Townships are required to prepare an Official Plan Amendment which would include supporting information demonstrating the need for a deviation from the standard criteria. Such proposals should be submitted to the Ministry of Housing, the Ministry of Natural Resources and the local Conservation Authority for their consideration.

Furthermore, the Municipalities/Townships have jurisdiction of urban storm water development in drainage areas of less than 125 hectares. These areas are exempted from construction regulations prepared by the Authorities under the Conservation Authorities Act. In addition, the Municipalities/Townships may be delegated by the local Authority, the jurisdiction for urban storm water management over larger watersheds, regardless of size, once the Municipalities/Townships adopt storm water management plans which comply with the Provincial objectives of flood plain management and in accordance with Provincial procedural policies.

3.0 HYDROLOGY

3.1 GENERAL

Hydrology is the science that deals with the properties, distribution and circulation of water. The circuit of water movement from the atmosphere to the earth and back to the atmosphere through various stages or processes, as precipitation, interception, runoff, infiltration, percolation, storage, evaporation and transpiration is known as the hydrologic cycle.

In any hydrologic study, quantitative information on the rainfall runoff relationships of the study area is of prime importance. Natural precipitation varies greatly in time and space, and methods for quantifying it depend upon the technique employed for runoff estimation.

In applying any runoff estimation method, major considerations are:

(a) Regional climatological, hydro-physiographical, and geological differences.

(b) Differences in basin characteristics such as drainage area size and shape, channel length and slopes, potential storage, etc.

(c) Changing basin characteristics such as unregulated to regulated and land usage.

(d) Availability of data.

(e) Statistical significance of available data.

As defined by the Provincial Flood Plain Criteria, the Regulatory flood within the South Nation River Conservation Authority jurisdiction is the 100 year flood.

In general, the watershed of Bear Brook (See Figure 3.1), because of its shape and location, will produce maximum peak flows for the Regulatory flood (100 year) and the more frequent return events (eg. 10 year flood) as a result of a rainfall/snowmelt event or a snowmelt event. However, as the drainage area decreases, flooding may be related to high intensity rainfall events rather than snowmelt events.

Because of the preceding, the following five methodologies were used to determine the peak flows for the various return frequency events:

(1) Synthetic Unit Hydrograph Method (Ref. 1, 2, 3), and Single Station Frequency Analysis (Ref. 17).

(2) Delisle River Regional Flood Frequency Method (Ref. 18).

(3) Regional Regression Method (MNR) (Ref. 21).

(4) Watershed Classification Method (WCM) (Ref. 9).

(5) Index Flood Method (Environment Canada) (Ref. 11, 21).

Initial hydrologic analyses were undertaken, based on the above methodologies, for the 100 year flood event for the Bear Brook watershed. Following a meeting to discuss and compare the results of the initial hydrologic analyses, the Project Team came to the conclusion that:

(1) Since the Index Flood Method used all the recorded discharges, the 1979 and 1981 streamflow data recorded at Station No. 02LB008 should be included in the Single Station Frequency Analysis.

(2) A watershed parameter (B) value of 300 would be utilized in the HYMO model to generate peak flows, since the peak flows obtained from the preliminary hydrologic analyses compared favourably with the peak flows obtained by using the other four methodologies.

(3) Based on the items above, the VUH and HYMO models for the Bear Brook watershed would have to be calibrated in order to generate peak flows that would correspond to the 100 year flood flow determined under the Single Station Frequency Analysis.

(4) The total snowmelt amounts for the 1 to 7 day events estimated by AES should be distributed in accordance to the Keifer & Chu distribution, as outlined in the Technical Guidelines (Ref. 21), in order to determine the daily distributed incremental depth.

    In addition, the daily distributed incremental depth should be spatially distributed using the AES 30% Curve for Southern Ontario.

(5) The mean water equivalent of the snowpack recorded at Bear Brook and Bourget is about 100% higher than the mean water equivalent of the snowpack recorded at Bells Corners. Therefore, an increase by 50% to the snowpack amounts appears to be justifiable, in considering a snowmelt/rainfall event.

3.2 SYNTHETIC UNIT HYDROGRAPH METHOD

In catchments where no streamflow records are available, flood hydrographs are computed using the synthetic unit hydrographs. This procedure involves applying a design storm, determining rainfall-runoff relationships, and routing and summing the individual reach/basin hydrographs to various points of interest.

For this particular study, the modelling was undertaken to provide a reliable means to distribute flows to areas remote from the existing streamflow station near Bourget. The following were undertaken to determine the flood flows of the various return frequency events for the Bear Brook watershed:

(a) Snowmelt/Rainfall Simulations.

(b) Rainfall Simulations.

3.2.1 Snowmelt/Rainfall Simulations

In carrying out the snowmelt/rainfall simulations for the various return frequency events, the Climatological Applications Branch of Atmospheric Environment Service (AES) was retained by Ecos Garatech to determine the daily total free water input for the snowmelt season by computing the daily snow budget at Ottawa International Airport meteorological station. The snow budget computation procedures employed by AES (Ref. 19, 20) is summarized as follows:

(1) daily snowfall water equivalent is added to the accumulated snowpack.

(2) if the accumulated snowpack is non-zero, the potential snowmelt for the day is computed by a snowmelt model.

(3) if the potential melt is less than the accumulated snowpack water equivalent, then the actual melt for the day is equal to the potential melt and the snowpack is depleted by this amount.

(4) if the potential melt is greater than the accumulated snowpack, the actual melt for the day is set equal to the water equivalent of the snowpack which is then depleted to zero.

(5) the total free water input for the day is then the sum of the actual snowmelt plus any rainfall for the day.

The daily total free water computations were undertaken using the following five snow budget simulation models:

1) $SM = 0.0397\ (Ta - 27.6)$ (in/day)
2) $SM = (0.074 + 0.007\ R)(Ta - 32) + 0.05$ (in/day)
3) $SM = 3.0(Ta + TCA(((Tx - Tn)/8) + Tn))$ (mm/day)
4) $SM = 0.02\ (Tx - 32)$ (in/day)
5) $SM = 0.08\ (Ta - 32)$ (in/day)

where:
- SM = snowmelt amount (in)
- Ta = mean daily temperature (degrees F)
- Tx = maximum daily temperature (degrees F)
- Tn = minimum daily temperature (degrees F)
- $TCA = (Tn/4.4)$ such that $(0 <= TCA <= 1.5)$
- R = daily rainfall and/or snowfall (in)

The snowmelt amounts obtained from the above equations were used in the initial hydrologic analysis. During the course of the study, it had been revealed that the mean water equivalent of the snowpack recorded at Bear Brook and Bourget is about double that recorded at Bells Corner. As such, the snowmelt amounts were increased by 50%, which appears to be reasonable in considering snowmelt/rainfall events, in the final hydrologic analysis.

A) Station Selection

Ottawa International Airport station is the longest, and the closest meteorological station to the Bear Brook watershed. It has 46 years of recorded data from 1939 to 1982.

B) Snowmelt Model Selection

Of the above five snowmelt models, Model 4 was developed for climatological conditions in Southern Ontario. Potential snowmelt was computed based on maximum temperatures.

Model 5 is a modified version of Model 4, and it used mean temperatures for calculating potential snowmelt, as opposed to maximum temperatures.

Based on previous studies undertaken within the Eastern Region jurisdictions, it has been found that the melt rate coefficient was about 0.08 when mean temperatures were used in calculating potential snowmelt. As such, Model 5 was selected for the Bear Brook watershed.

3.2.2 Snowmelt Runoff Distribution

Although the AES analysis provided the total runoff volumes and the associated return frequency of the snowmelt events, it was necessary to establish the spatial distribution of runoff volume over time.

The SCS Type II 24-Hr rainfall distribution (Ref. 22) was initially used to create the spatial distribution for the snowmelt runoff volumes, to determine peak flows for 100, 50, 25, 10, 5 and 2 year 7-day snowmelt events. The SCS Type II distribution is given in Table 3.1.

For the final hydrologic analyses, the following were undertaken, in order to model the 100 year peak flow obtained from the Single Station Frequency Analysis:

(1) The total depths for the 100 year, 1 to 7 day snowmelt events were tabulated, and the daily incremental depths were obtained.

(2) Using the Keifer and Chu distribution (Ref. 21), the daily incremental depths were distributed accordingly (see Table 3.2).

(3) The daily distributed incremental depths were spatially distributed using the AES 30% Curve for Southern Ontario (See Table 3.3).

For snowmelt events, no aerial reductions were necessary.

3.2.3 Holtan Infiltration Model

The Holtan infiltration model is one of the subroutines in the Variable Unit Hydrograph (VUH) computer program (Ref. 25), which was developed by the Conservation Authorities and Water Management Branch of the Ontario Ministry of Natural Resources. The Holtan infiltration equation is:

$$f = GI * a * (SA ** 1.4) + fc$$

where: f = infiltration capacity (mm/hr)
GI = growth index of vegetative cover (% maturity)
a = coefficient representing infiltration capacity of available storage, i.e. index of surface-connected porosity (mm/hr per mm ** 1.4)
SA = available air void storage in the surface layer (A-horizon in the soil) (mm)
fc = constant rate of infiltration after prolonged wetting (mm/hr)

The other parameters required in the Holtan model are:

SMAX = maximum gravitational soil-water storage in the surface layer or A-horizon (mm)
DCON = maximum rate of percolation from the surface layer under saturated conditions (mm/hr)

In order to determine the parameters for the Holtan infiltration model, each primary hydrologic unit (reaches and basins) within the Bear Brook watershed was analyzed based on the type of soil (hydrologic soil groups), depth of surface layer and vegetative cover (land use).

A) Growth Index (GI)

The growth index represents the relative maturity of the vegetative cover. This parameter is dependent on the season, and varies from about 0.05 at seedling stage to a maximum value of 1.0 at full maturity.

In the initial stages of the study, it was assumed that all vegetation are grown and a value of 0.40 was assumed for the spring season. As a result of calibration, the GI value was modified to 0.20 for the final hydrologic analyses.

B) Coefficient of Infiltration Capacity (a)

The coefficient of infiltration capacity is evaluated at plant maturity and represents the fraction of the ground surface area occupied by plant stems or root crowns, i.e. the density of plant roots. The parameter is dependent on the land use.

Viessman, et al (Ref. 14) and Holtan, et al (Ref. 24) provide estimates of "a" for various vegetation of poor and good conditions (see Table 3.4).

The various land uses of each primary hydrologic unit (PHU) within the Bear Brook watershed were evaluated (Table 3.5). The area of open land was further divided into crop land and pasture in accordance to the ratio given in the Ministry of Transportation of Ontario (MTO) Bridge Hydraulics Manual (Ref. 9) under Carleton, Prescott and Russell Counties.

In order to estimate a weighted "a" value for each primary hydrologic unit, the following coefficients of infiltration capacity were used:

a)	Residential/Lake/Pond	0.05
b)	Crop Land	0.20
c)	Pasture	0.40
d)	Woods	0.80

C) <u>Soil-Water Storage (SA & SMAX)</u>

The value of SA represents the available air void storage in the soil surface layer (A-horizon) at any time. The parameter is dependent on the antecedent conditions of the soil.

The value of SA decreases in time as water infiltrates into the top soil. However, the value may sometimes increase due to evaporation, and whenever the seepage rate into a second deeper layer is higher than the rate of actual infiltration into the layer.

Basically, the value is equal to zero under saturated conditions. When the moisture approaches the wilting point (very dry condition), the value is equal to the total porosity (S) of the soil minus the moisture content at 15 bar tension.

The value of SMAX represents the maximum gravitational soil-water storage in the surface layer, and is dependent on the depth of the soil.

The SMAX value is estimated as the total porosity of the soil minus the moisture content at 0.3 bar tension.

The depths of the various soil types within each primary hydrologic unit were determined from the <u>Ontario Soil Survey Report No. 7 of Carleton County</u> (Ref. 5), <u>Report No. 33 of Russell & Prescott Counties</u> (Ref. 4), <u>Report No. 58 of the Regional Municipality of Ottawa-Carleton</u> (Ref. 6), and soils maps (Ref. 7, 8). The soil types were classified according to the MTO Bridge Hydraulics Manual (Ref. 9) and the SCS National Engineering Handbook on Hydrology (Ref. 1). The soil types, group classifications and depths are given in Table 3.6.

The hydrologic capacities of soil texture classes, as presented in Table 3.7, were utilized to determine the SMAX values for each primary hydrologic unit in the Bear Brook watershed.

The soil depths vary from about 76 mm to 279 mm. Collins & Moon (Ref. 23) states that larger soil layer depths should be used in summer and smaller layer depths in spring and fall. For this study, an initial soil depth of 150 mm was utilized under spring runoff conditions.

The initial SMAX values are given in Table 3.8 for each soil group of varying depths.

D) Constant Infiltration Rate (fc)

The constant rate of infiltration is dependent on the soil type. The estimated values of fc are given by Holtan et al, (Ref. 24). These values were subsequently used to determine the weighted fc values for each primary hydrologic unit of the Bear Brook watershed.

E) Maximum Rate of Percolation (DCON)

The percolation rate is controlled by the amount of soil-water present in gravity-drainable pores and the rate is dependent on the subsurface soil conditions.

It was assumed that the maximum rate of percolation at saturation, DCON, is equal to the constant rate of infiltration, fc.

F) Holtan Model Parameters

A summary of the parameters required to be utilized in the Holtan infiltration model are given in Table 3.9.

As a result of the initial calibration, the values for SMAX, SA, fc and DCON were re-established. The initial calibrated parameters for the Holtan Infiltration model are provided in Table 3.10.

Studies undertaken in the U.S. indicate growth index values (GI) of 0.3 and 0.1 for alfalfa and corn respectively, for the spring runoff period. Trial measurements of infiltrability of frozen and partially frozen loams by Collins and Moon, produced values of 0.2 mm/hr and 0.5 mm/hr respectively. Therefore, in the final calibration, the values for GI, fc and DCON were set to 0.20.

3.2.4 Watershed Parameters

The synthetic unit hydrograph program utilized in the simulation of flood flows within the Bear Brook watershed was HYMO.

The basic shape and dimensions of the HYMO unit hydrograph can be reasonably defined if the time to peak (Tp) and the recession constant (K) are known.

In ungauged watersheds Tp and K must be estimated. This is accomplished in the HYMO model by the following regression equations:

K = 27 (A**0.231)(S**-0.77)((L**2/A)**0.124)

Tp = 4.63(A**0.422)(S**-0.46)((L**2/A)**0.133)

where: A = watershed area (sq mi)
S = flood plain slope (ft/mi)
L = hydraulic length (mi)

Some of the watershed parameters for each primary hydrologic unit are given in Table 3.11.

The analysis of the time to peak of each primary hydrologic unit within the Bear Brook watershed was carried out based on the MTO, SCS and HYMO methodologies.

The MTO methodology utilizes the Bransby Williams equation (See Appendix A) to determine the time of concentration (tc) and the SCS method for the time to peak, the SCS methodology utilizes the Kirpich equation to obtain the time of concentration and subsequently the time to peak, and the HYMO methodology utilizes the above-given equation. The results of the time to peak analysis are presented in Table 3.12.

As a result of the comparison, the HYMO times to peak for the primary hydrologic units were selected to be utilized in the hydrologic model to determine the peak flows for the various return frequency events.

In order to determine reasonable recession constants (K) for the primary hydrologic units, the values of K for each sub-watershed, based on watershed parameters of B equal 250, 300, 350 and 400 were estimated. The results of the computations are provided in Table 3.13. Based on the review of the preliminary hydrologic results and the results of the other methodologies, a value of 300 was selected as the most representative watershed parameter (B).

3.2.5 Selection of Snowmelt Event

The following procedures and analyses were undertaken in order to select an appropriate snowmelt/rainfall event for the Bear Brook watershed.

(1) The snowmelt amounts for the 2-day, 5-day, 7-day and 10-day 100 year events were spatially distributed by applying the SCS Type II distribution.

(2) The distributed snowmelt amounts in conjunction with the Holtan model parameters were used as input data into the VUH program to simulate the excess overland runoff.

(3) The excess overland runoff obtained for the 2-day, 5-day, 7-day and 10-day 100 year events in the VUH model was then used together with the HYMO parameters to evaluate the peak flows for the Bear Brook watershed, in accordance to the watershed schematic presented in Figure 3.2.

Since the VUH model simulates the amount of infiltration and thus, the resultant runoff, a curve number (CN) value of 100 was used in the HYMO model, i.e. allowing all the excess runoff to flow off the land. Initial abstractions pertaining to depression storage and interception were not accounted for in the VUH and HYMO models. It was assumed that initial abstractions are minimal and that it would have occurred prior to the major portion of the spring snowmelt runoff.

Initial simulations were undertaken for the 2-day, 5-day, 7-day and 10-day 100 year snowmelt events for the Bear Brook watershed. Based on a review and comparison of the preliminary results with the other methodologies and the runoff records of the spring events recorded at Bourget, the 7-day snowmelt event was selected as being the most representative of the significant runoff period for the Bear Brook watershed.

3.2.6 Results

Based on the preceding, the peak flood flows for the 7-day 100 year snowmelt events were initially generated for the Bear Brook watershed, for B values ranging from 250 to 400, as provided in Table 3.14.

Upon undertaking the initial calibration, whereby the Holtan infiltration parameters were re-established, peak flood flows for the 2-day, 5-day, 7-day and 10-day 100 year snowmelt events were generated. The calibrated peak flood flows are given in Table 3.15.

Utilizing the 7-day snowmelt runoff amounts (50% increase) and distribution (Keifer & Chu and AES 30% Curve), and values of 0.2 for GI, fc and DCON, the peak flows for the 100, 50, 25, 10, 5 and 2 year frequency events were established. The results of the final simulations are provided in Table 3.16.

3.2.7 Subwatershed Breakdown

For comparison purposes, Subwatersheds 5 to 9 were further divided into sub-basins (See Figure 3.3). This was undertaken in order to generate peak flows at specific points of interest. The generated snowmelt related peak flows would be compared to the peak flows obtained from the rainfall simulations.

The schematics of Subwatersheds 5 to 9 are illustrated in Figures 3.4 to 3.8, respectively. Table 3.17 provides the HYMO sub-basin parameters.

A times to peak analysis was also carried out based on the MTO, SCS and HYMO methodologies. A summary of the times to peak analysis are given in Table 3.18. As a consistency, the HYMO times to peak were selected for use in the hydrologic models for Basins 5 to 9.

In order to determine reasonable recession constants (K) for the primary hydrologic units in Basins 5 to 9, values of the watershed parameter (B) ranging from 300 to 400 were used to estimate the recession constants. The computations are provided in Table 3.19.

Since watershed parameter (B) values of 300 was used in the analysis for the entire Bear Brook watershed, therefore, a B value of 300 was also selected for the subwatershed breakdown.

The calibrated Holtan infiltration parameters were utilized to generate the excess overland runoff from the VUH model. The excess runoff, in conjunction with the appropriate HYMO sub-basin parameters, were then used as input data in the HYMO models to determine peak flows at specific locations in Basins 5 to 9 for the 7-day 100, 50, 25, 10, 5 and 2 year snowmelt events. The simulated peak flows are given in Table 3.20. In addition, the routed peak flows of the 7-day 100 year snowmelt event are provided in Table 3.21.

3.2.8 Rainfall Analysis

3.2.8.1 Uncalibrated

As discussed earlier in this section, in smaller drainage areas (of less than 65 sq km) flooding may be related to high intensity rainfall events rather than snowmelt events, for the 100 year and the lower return frequency events. This is applicable to the following subwatersheds (basins):

1) Elian Reginbald Drain subwatershed (Basin 5), which drains by the Town of Leonard.

2) Bearbrook Drain subwatershed (Basin 6), which drains by the Town of Bearbrook.

3) McWilliams Drain subwatershed (Basin 7), which drains by the Town of Navan.

4) Shaws Creek subwatershed (Basin 8).

5) McKinnons Creek subwatershed (Basin 9).

As a result, the synthetic unit hydrograph technique was also employed to generate the peak flows for the 100, 50, 25, 10, 5 and 2 year storm events, in order to compare the results obtained from the snowmelt simulations for the above subwatersheds.

The upper sub-reaches of Basins 5 to 9 were sub-divided into sub-basin boundaries as illustrated in Figure 3.2. The HYMO watershed schematics for Basins 5 to 9 are illustrated in Figures 3.3 to 3.7, respectively.

The following sections outline the procedures and criteria used to determine the peak flood flows for the 100, 50, 25, 10, 5 and 2 year return frequency storm events.

A) <u>Soils, Land Use and Curve Numbers</u>

In order to determine the hydrologic soil cover complex numbers of the sub-basins of Basins 5 to 9, an analysis was undertaken for the soil types (Ref. 4, 5, 6, 7, 8), and the present and future land use classifications.

The soil types that occur within each sub-basin were classified with respect to their drainage characteristics and were subsequently, categorized into their respective hydrologic soil groups (See Table 3.6). The various hydrologic soil groups within each sub-basin are given in Table 3.22.

The various land use and treatment were classified on a storm runoff-producing basis. The greater the ability of the land use and treatment to increase total retention, the lower the classification. The various types of land use which occur within each sub-basin are presented in Table 3.23.

The soil groups and land use and treatment classes were combined into hydrologic soil cover complex numbers. The resulting soil cover complex numbers for each primary hydrologic unit in each basin are given in Table 3.24.

B) <u>Rainfall</u>

The data for the rainfall depths of the 100, 50, 25, 10, 5, and 2 year - 12 hour storm events were obtained from <u>Table D6</u> of the <u>Flood Plain Management in Ontario - Technical Guidelines</u> (Ref. 21) for Ottawa International Airport station. These values were confirmed by the tabulated data of rainfall intensity-duration-frequency values obtained from Atmospheric Environment Service (AES).

The 12 hour duration rainfall distribution for the various storm events is based on the 30% curve of the AES 12 hour storm distribution for Southern Ontario (Table D-8 of the Technical Guidelines).

The AES 30% - 12 hour distribution is given in Table 3.3, and the rainfall depths of the various storm events are provided in Table 3.25.

C) **Recession Constants, Times to Peak and Watershed Parameters**

The sub-basin parameters for each of the primary hydrologic units (PHU) of Basins 5 to 9 are given in Table 3.17. The analysis of the times to peak of each primary hydrologic unit in the basins was carried out based on the MTO, SCS and HYMO methodologies. The results of the times to peak analysis are presented in Table 3.18.

As a result of the comparison and in order to be consistent with the snowmelt analysis (whereby the times to peak were estimated and calibrated according to the HYMO equation), the HYMO times to peak were selected to be utilized in the rainfall hydrologic models to determine the peak flows for the various return frequency storm events.

The values of K for each sub-basin, based on watershed parameters of B equals 300, 350 and 400 were estimated. The results of the computations are provided in Table 3.19.

A B value of 300 was selected to be utilized in determining the peak flows of Basins 5 to 9 for the 100, 50, 25, 10, 5 and 2 year - 12 hour storm events.

D) **Results**

The final hydrologic parameters of Basins 5 to 9 were assembled as computer data input and analyzed utilizing the HYMO program.

The final peak flows for the various storm events for Basins 5 to 9 are provided in Table 3.26. In addition, the routed peak flows of the 100 year storm for Basins 5 to 9 are given in Table 3.27.

3.2.8.2 Calibrated

In addition, rainfall analysis was also carried out for the whole Bear Brook watershed.

Single station frequency analysis (see Section 3.7) was undertaken for the summer events. Based on the length of recorded data available, the 25 year storm peak flow (no specific type of distribution) was used for calibration.

The equivalent circular area of the Bear Brook watershed was determined. Utilizing the WMO areal reduction curve (Figure D-6 of Technical Guidelines), a 0.80 ratio was obtained. As such, the total rainfall depths (Table 3.25) for the various storm events were reduced by 20%. The total rainfall depths were distributed using the AES 30% Curve for Southern Ontario, for consistency.

Initially, an average CN value of 76 was used for the entire watershed to generate the 25 year storm peak flow at the Bourget streamflow station. The simulated 25 year storm peak flow was compared to the 25 year peak flow obtained by flood frequency analysis. By increasing the average CN value, the final value of 79 was utilized to generate the peak flows of the 100, 50, 25, 10, 5 and 2 year storm events, as given in Table 3.28. As noted, a final CN value of 79 compared quite favourably with those numbers obtained for Basins 5 to 9 (see Table 3.24).

3.3 DELISLE RIVER REGIONAL FLOOD FREQUENCY METHOD

In April 1983, a report entitled Delisle River Flood Plain Mapping and Water Management Study (Ref. 18) was prepared for the Raisin Region Conservation Authority whereby, a regional flood frequency analyses was undertaken to determine flood flows for the Delisle River watershed. The analyses, consisting of single station frequency analysis, homogeneity test and regional analysis, was performed to establish the relationships of flood flows with key watershed parameters such as drainage area, channel slope and channel length.

Twenty-two streamflow stations, with drainage areas ranging from 19 sq km to 3810 sq km were analyzed, out of which nine streamflow stations were selected and utilized. The following regression equations were established:

$$Q100 = 0.32\ DA + 7.62$$

$$Q50 = 0.30\ DA + 7.96$$

$$Q25 = 0.28\ DA + 7.97$$

$$Q10 = 0.26\ DA + 6.95$$

$$Q5 = 0.24\ DA + 5.62$$

where: Q100 = 100 year mean daily peak flow (cms)
Q50 = 50 year mean daily peak flow (cms)
Q25 = 25 year mean daily peak flow (cms)
Q10 = 10 year mean daily peak flow (cms)
Q5 = 5 year mean daily peak flow (cms)
DA = total drainage area upstream of a point of interest (sq km)

In addition, the watershed mean multiple factor of 1.122 was established to determine the maximum instantaneous peak flows from the mean daily peak flows.

The drainage area of the Bear Brook watershed was determined to be 458 sq km. Utilizing the above equations, the mean daily peak flow for the 100 year flood event was found to be 154.2 cms. Applying the watershed mean multiple factor of 1.122, the maximum instantaneous peak flow was determined to be 173.0 cms. The results of the computations are tabulated in Table 3.29.

3.4 REGIONAL REGRESSION METHOD

The Regional Regression Method was developed by the Conservation Authorities and Water Management Branch of the Ministry of Natural Resources. This method is fully described in the document entitled Flood Plain Management in Ontario - Technical Guidelines (Ref. 21).

Regression equations, in the general form:

$$Q = a0 * (DA ** a1) * (ACLS ** a2) * (MAR ** a3) * (EQSLP ** a4)$$

where: DA = drainage area (sq km)
ACLS = index of area controlled by lakes and swamps
EQSLP = equivalent slope (m/m)
MAR = mean annual runoff (mm)
MAS = mean annual snowfall (cm), and

a0, a1, a2, a3 and a4 are regression coefficients given in the Technical Guidelines, were utilized to determine the mean daily peak flows for the Bear Brook watershed for the 100, 50, 20, 10, 5 and 2 year flood events. The values of the parameters for the regression equation are provided in Table 3.30. The 25 year flood flow was then estimated from the probability plot. Applying a mean multiple factor of 1.122, the maximum instantaneous peak flows were obtained. The results of the mean daily and maximum instantaneous peak flows for the 100, 50, 25, 10, 5 and 2 year flood events are given in Table 3.30.

3.5 WATERSHED CLASSIFICATION METHOD

The Ministry of Transportation of Ontario (MTO) (Ref. 9) have developed a methodology for predicting flood discharges from medium and large drainage basins. This methodology, referred to as the "Watershed Classification Method (WCM)" accounts for both high runoff potential areas (using SCS curve numbers) and for low runoff potential areas typical of the Canadian Shield. Flood discharges are related to such hydrologic properties as drainage area, average slope, area of lakes and SCS curve numbers.

The Watershed Classification Method is a simple and convenient means of estimating design floods for natural watersheds in Ontario ranging in area from 25 sq km or less to about 100,000 sq km and for varying return frequency events.

The Watershed Classification Method is applied in six principal steps as follows:

(1) Base classification of the watershed is determined from the soils/land use curve number in southern and high-runoff northern basins, and from the retention factor in the Shield type basins.

(2) The base class is adjusted for minor watershed characteristics.

(3) The 25 year flood is derived from pre-determined runoff curves.

(4) The 25 year flood is adjusted to the required return frequency events.

(5) Any abnormal factors influencing the design flow are allowed for.

(6) The design flood estimate is checked by means of field and other data.

The Bear Brook watershed was considered as the Southern Type (B) basin. The base classification was therefore determined from the Soil/Land Use Curve Number chart. As the 25 year runoff curves represent the average basin characteristics of the gauging stations used to derive the curves, the base class had to be adjusted for specific characteristics of the Bear Brook watershed, such as, slope, shape and precipitation.

The general equation for the 25 year instantaneous peak flood flow is:

$$Q25 = CA^{0.75}$$

where:
- Q25 = 25 year instantaneous peak flood flow (cms)
- C = class coefficient
- A = basin area (sq km)

Conversion factors were then applied to the 25 year peak flood flow to establish peak flood flows for the other return frequency events.

The Watershed Classification Method was applied at the outlet of the Bear Brook watershed. The hydrologic properties and the adjustments applied to the base watershed class are summarized in Table 3.31. The results derived from this methodology for the various return frequency events are presented in Table 3.32.

3.6 INDEX FLOOD METHOD

S.M.A. Moin and M.A. Shaw of the Water Planning and Management Branch, Inland Waters Directorate, Environment Canada (Ref. 11) have developed a methodology for predicting flood discharges for varying watershed sizes in Ontario. This method is based on a regional flood frequency analysis carried out in Ontario on over 200 streamflow stations with 10 or more years of records. Ontario was divided into twelve hydrologic regions with roughly similar soils and physiographical characteristics. For each region a graphical relationship between the mean annual flood and drainage area was determined.

The Flood Index Method is applied in six principal steps as follows:

Step 1

The watershed area contributing to flow at the location in question.

Step 2

Determine the region in which the watershed is located by reference to Figure 5.1.1.

Step 3

Verify that the drainage area is within the range used to derive regional relationships. If not, continue with caution and verify the end result using another technique.

Step 4

Refer to Table 5.1.1 for the appropriate regional equation or to the appropriate plots of index flood versus drainage area (Figures 5.2.1 to 5.13.1) and estimate the index flood (Q2) using the drainage area determined in Step 1.

Step 5

Refer to the appropriate dimensionless frequency curve for the region shown in Figures 5.2.2 - 5.13.2 (Figures 5.14.1 to 5.14.2, if the Expected Probability Adjustment is used) and extract the ratio corresponding to the return period for the desired flood flow estimate.

Step 6

Compute the desired return period flood flow estimate as the product of the index flood estimate (Step 4) and the appropriate dimensionless frequency ratio derived from Step 5.

Step 7

Verify the above estimate using other techniques. If the station is located at or close to the boundary of a region, check the flow estimate using the regional relationships for the adjacent region.

The Bear Brook watershed was determined to be located in Region 2 from Figure 5.1.1. Utilizing Figure 5.3.1, the index flood flow (Q2) was found to be 119.0 cms. The dimensionless frequency ratio for the other flood events were estimated from Figure 5.3.2, and subsequently, the mean daily peak flows for the 100, 50, 25, 10, 5 and 2 year flood events were established. Applying the watershed mean multiple factor of 1.122, the maximum instantaneous peak flows were determined. The results of the mean daily and maximum instantaneous peak flows for the various return frequency flood events are provided in Table 3.33.

3.7 SINGLE STATION FREQUENCY ANALYSIS

The Consolidated Frequency Analysis Package (CFA88) was developed by the Water Resources Branch, Inland Waters Directorate, Environment Canada (Ref. 17).

The program includes the nonparametric screening of the input data sample for independence, trend, homogeneity and general randomness. The data can also be examined using several graphical displays. The presence of low and high outliers within the data set can be detected.

For a given station, the following eight combinations can be performed to undertake frequency analysis:

(a) The standard case; alone or in conjunction with
(b) Historic highs
(c) Historic highs and low outliers
(d) Historic highs, low outliers, and zeros
(e) Historic highs and zeros
(f) Low outliers
(g) Low outliers and zeros
(h) Zeros.

The frequency analysis can be performed by one or more of the Generalized Extreme Value (GEV), the Three-Parameter Lognormal (3PLN), the Log Pearson Type III (LP3) and the Wakeby distributions.

3.7.1 Spring Events

The streamflow station, Bear Brook near Bourget, Station No. 02LB008, had 31 years of recorded data, and is located at the outlet of Reach 3 in the Bear Brook watershed delineation (See Figure 3.1). There were 31 recorded maximum daily discharges and 4 recorded maximum instantaneous discharges. Of those data, 12 years of maximum daily discharges were recorded under ice conditions. Table 3.34 provides a summary of the recorded discharges at Station No. 02LB008.

Initially, the recorded data was to be used for the calibration of the HYMO model for the Bear Brook watershed. A review of the daily discharges indicated that:

(1) The majority of the recorded data were under ice conditions during the winter and spring periods.

(2) In those years where the recorded maximum daily discharges were under ice-free conditions, ice conditions did occur up to a few days prior to the peak discharge.

(3) During the 1979 and 1981 spring runoff, where ice conditions prevailed, extreme peak flows were recorded.

As a result of the ice conditions, no hourly discharges were made available to be utilized in the calibration of the HYMO models.

The recorded data was used as input sample data into the CFA88 program. Three cases were considered in the single station frequency analysis:

(1) All the data was included in the analysis.

(2) Periods of records under ice conditions were excluded in the sample data.

(3) The 1979 and 1981 records, and the records under ice conditions were excluded.

All the above-mentioned distributions were used to fit the sample data sets. The results of the distributions indicated that the Three-Parameter Lognormal distribution was the most appropriate one for the sample data sets. The summary of the frequency analysis for the three conditions are given in Table 3.35. Again, by applying a peaking factor of 1.122, the maximum instantaneous peak flood flows were determined. The peaking factor was obtained from a previous study entitled Delisle River Flood Plain Mapping and Water Management Study (Ref. 18).

The results were also reviewed by the Project Team during the preliminary hydrologic analyses meeting. The Project Team decided that all the recorded data be included in the frequency analysis (Condition I).

3.7.2 Summer Events

The maximum daily discharge of each year from May to November, for the period of records, are provided in Table 3.34. The maximum daily discharges were used to carry out the single station frequency analysis, in order to calibrate the summer runoff events for the entire watershed.

Since only twelve years of data are available, the 25 year peak flows obtained by frequency analysis (four distributions) were used for comparison with the rainfall simulations (see Section 3.2.8.2). The results of the single station frequency analysis are given in Table 3.36.

3.8 COMPARISON OF RESULTS

The following five methodologies were used to estimate the peak flood flows for the 100, 50, 25, 10, 5 and 2 year return frequency events for the Bear Brook watershed:

(1) Synthetic Unit Hydrograph Method, and Single Station Frequency Analysis.

(2) Delisle River Regional Flood Frequency Method.

(3) Regional Regression Method.

(4) Watershed Classification Method.

(5) Index Flood Method.

A summary of the maximum instantaneous peak flows from the above methodologies, at the outlet of Bear Brook, are tabulated in Table 3.37. As can be seen from the results, the 100 year peak flows vary from 173.0 cms (Method 2) to 357.1 cms (Method 1), the mean in the order of 259 cms. The 2 year peak flows vary from 87.0 cms (Method 4) to 174.3 cms (Method 1), the mean about 121 cms.

It should be emphasized that the Method 1 results were calibrated, to obtain the same 100 year peak flow determined by frequency analysis at the streamflow station. Therefore, for comparison purposes, the peak flood flows for all the various return frequency events determined by Method 1 and single station frequency analysis are given in Table 3.38. As expected, as a result of model calibration at the 100 year level, the peak flows at the more frequent levels (2, 5 and 10 year) varied quite substantially. The HYMO models at these levels could be re-calibrated as more data (streamflow, snowpack, etc.) becomes available. To be conservative, the peak flows determined from the 7-day snowmelt events (Method 1) were selected to be used in the hydraulic analysis of the main channel of the Bear Brook regime.

As outlined in Section 3.2.8.2, a calibrated CN value of 79 was used to generate the summer peak flows of the 100, 50, 25, 10, 5 and 2 year storm events for the Bear Brook watershed. For comparison, the summer peak flows are given in Table 3.36, in conjunction with the peak flows obtained from frequency analysis. At the higher return events, the simulated peak flows fell within the range of flows from the various distributions, with the exception of the Three Parameter Log Normal. This could be due to the small data set (12 years) used in the frequency analysis. The rainfall events could be further verified as more data becomes available, but based on the results obtained, these events are considered to be sufficiently accurate for use in this study.

For Basins 5 to 9, peak flows were determined at various points of interest along the sub-basins, to be utilized in the hydraulic assessments. The 100 year routed peak flows obtained from the 7-day snowmelt (Table 3.21) and the rainfall (Table 3.27) analyses are summarized in Table 3.39. Based on the results, it would appear that, for the 100 year event, Basins 5 and 6 are rainfall related, and Basins 7, 8 and 9 are snowmelt related.

For the other return frequency events, the peak flows (from Tables 3.20 and 3.26) are summarized in Table 3.40. Based on the results of the simulations, the higher peak flows were selected to be used in the hydraulic assessments along the reaches of Basins 5 to 9. The selected peak flows are provided in Table 3.40. It should be noted that the selected peak flows are provided at the outlet of each basin only for the various storm events.

TABLE 3.1

24-HOUR DISTRIBUTION

Time (hr)	Incr. Amt. (%)	Time (hr)	Incr. Amt. (%)	Time (hr)	Incr. Amt. (%)
0.	0.	10.25	0.9	13.5	1.1
1.0	1.0	10.5	1.4	13.75	1.1
2.0	1.2	10.75	1.5	14.0	1.0
3.0	1.2	11.0	1.6	15.0	3.1
4.0	1.4	11.25	1.7	16.0	2.9
5.0	1.6	11.5	3.1	17.0	2.0
6.0	1.6	11.75	10.4	18.0	2.0
7.0	2.0	12.0	27.6	19.0	1.6
8.0	2.0	12.25	4.4	20.0	1.6
8.5	1.1	12.5	2.8	21.0	1.4
9.0	1.6	12.75	1.9	22.0	1.2
9.5	1.6	13.0	1.8	23.0	1.2
10.0	1.8	13.25	1.6	24.0	1.0

TABLE 3.2

100 YEAR 7-DAY SNOWMELT EVENT

USING KEIFER & CHU DISTRIBUTION

Duration (days)	Total Depth (mm)	Incremental Depth (mm)	Distributed Incremental Depth (mm)	Percentage
1	59.52	59.52	25.20	10.80
2	93.75	34.23	33.62	14.50
3	125.70	31.95	34.23	14.70
4	159.32	33.62	59.52	25.60
5	187.72	28.40	31.95	13.70
6	212.92	25.20	28.40	12.20
7	232.58	19.66	19.66	8.50
TOTALS		232.58	232.58	100.00

TABLE 3.3

AES 30% CURVE - 12 HOUR DISTRIBUTION

Hour	AES 30% Curve - 12 Hour Rainfall Distribution for Southern Ontario Incr. %	Accum. %
0	0	0
1	15	15
2	25	40
3	22	62
4	14	76
5	12	88
6	8	96
7	3	99
8	1	100
9	0	100
10	0	100
11	0	100
12	0	100

TABLE 3.4

ESTIMATES OF "a"

Vegetation (Land Use)	Basal Area Rating "a"	
	Poor Condition	Good Condition
Fallow	0.10	0.30
Row crops	0.10	0.20
Small grains	0.20	0.30
Hay:		
Legumes	0.20	0.40
Sod	0.40	0.60
Pasture:		
Bunchgrass	0.20	0.40
Temporary (sod)	0.40	0.60
Permanent (sod)	0.80	1.00
Woods and forests	0.80	1.00

Notes:

1) Extracted from Holtan, et al (Ref. 24).
2) Adjustments to basal area rating "a" needed for weeds and grazing.
3) Value of poor condition fallow is after row crop.
4) Value of good condition fallow is after sod.

TABLE 3.5

LAND USE

PHU	Land Use (% of Area) Residential	Crop	Pasture	Wood
R1	2.0	61.0	34.0	3.0
R2	3.0	49.0	28.0	20.0
B2	0.2	1.8	1.0	97.0
R3	4.0	35.0	19.6	41.8
R3.1	4.0	47.4	26.6	22.0
R3.2 & B3.3	0.9	44.8	25.0	29.2
B3.4	4.0	28.9	16.3	50.8
B3.5	0.0	3.2	1.8	95.0
R4	32.0	50.0	28.0	20.0
R4.1	0.5	17.0	9.5	73.0
B4.2	1.8	19.3	10.9	68.0
R5	0.4	48.5	27.3	23.8
B5	1.4	51.5	29.0	18.0
R6	0.0	57.4	32.3	10.3
B6	2.9	50.8	28.6	17.8
R7	0.0	53.6	30.2	16.3
B7	1.8	53.4	30.1	14.6
R8	0.0	52.0	29.7	18.1
B8	1.2	45.0	25.3	28.7
R9	0.0	45.2	25.3	24.6
B9	4.3	51.4	28.8	15.4
B10	0.1	2.8	1.5	95.6
R10 & R11	5.6	40.7	22.8	30.9
R12	4.0	33.0	18.0	45.0
B13	2.0	37.0	21.0	40.0
B14	3.4	28.2	16.1	52.3

TABLE 3.6

DEPTH OF SOIL SURFACE LAYER

Soil Type	HSG	Depth Of Surface Layer (mm)
Allendale Series (Afsl)	B/C	203
Beinsville Series (Bsil)	B/C	152
Bearbrook Series (Bfsl)	B/C	127
Bearbrook Series (Bc) (Bcss)	C	127
Cerp Series (Ccl)	C	203
Grenville (Gl) (Glst) (Glsh)	B	178
Caster (Cfsl)	C	203
Farmingata Series (Fl)	B	76
North Gower (NGcl)	C	203
Rubica Series (Rfs)	C	102
St Samuel (Sfs)	B/C	178
St Rosalie (Roc)	C	127
Uplands (Ufs)	B	127
Matilda (Mtl)	C	152
Vars Series (Vgl)	B	279
Mountain (Mn fsl)	D	152
Wendover (Wcss) (Wc)	D	127
Peat (P) organics	D	0

Abbreviations:

c - clay
f - fine
s - sandy
ss - sandy spots
g - gravel
l - loam
si - silt
st - stone
sh - shallow

TABLE 3.7

HYDROLOGIC CAPACITIES OF SOIL TEXTURE CLASSES

HSG	Texture Class	S (% vol)	G (% vol)	AWC (% vol)	AWC/G
B	sandy loam	30.9	18.6	12.3	0.66
B	fine sandy loam	36.6	23.5	13.1	0.56
B	very fine sandy loam	32.7	21.0	11.7	0.56
B	loam	30.0	14.4	15.6	1.08
B/C	silt loam	31.3	11.4	19.9	1.74
B/C	sandy clay loam	25.3	13.4	11.9	0.89
C	clay loam	25.7	13.0	12.7	0.98
C	silty clay loam	23.3	8.4	14.9	1.77
D	sandy clay	19.4	11.6	7.8	0.67
D	silty clay	21.4	9.1	12.3	1.34
D	clay	18.8	7.3	11.5	1.58

Notes:

1) Information extracted from Holtan, et al (Ref. 24).
2) Hydrologic soil group (HSG) classification is based on information from MTO Bridge Hydraulics Manual (Ref. 9) and SCS National Engineering Handbook (Ref. 1).
3) S is equal to total porosity minus 15 bar moisture content.
4) G is equal to total porosity minus 0.3 bar moisture content.
5) AWC is the water available to plants (AWC = S - G).

TABLE 3.8

VALUES OF SMAX FOR EACH SOIL TYPE

HSG		G (%)	Depth Of Topsoil (mm)	SMAX (mm)
B	(l)	14.4	150	21.6
	(fs)	27.2	127	34.5
	(gl)	15.8	150	23.7
B/C	(sil)	11.4	150	17.1
	(fsl)	23.5	150	35.3
	(fsl)	23.5	127	29.8
	(fs)	27.2	150	40.8
C	(fsl)	23.5	150	35.2
	(fsl)	23.5	127	16.5
	(cl)	13	150	19.5
	(fs)	27.2	102	27.7
	(l)	14.4	150	21.9
	(c.ss)	11.6	127	14.7
	(c)	7.3	127	9.3
	(fsl)	23.5	150	35.2
D	(c.ss) (sc)	11.6	127	14.7
	(c)	7.3	127	9.3
	(fsl)	23.5	150	35.7
	organics	14.4	0	0

TABLE 3.9

SUMMARY OF PARAMETERS

HOLTAN INFILTRATION EQUATION

PHU	SMAX (mm)	SA (mm)	a (mm/hr/ mm**1.4)	GI (%)	fc (mm/hr)	DCON (mm/hr)
R1	25.46	25.46	0.2830	0.4	3.966	3.966
R2	24.16	24.16	0.3715	0.4	2.960	2.960
B2	35.83	35.83	0.7837	0.4	3.724	3.724
R3	28.28	28.28	0.4848	0.4	3.523	3.523
R3.1	13.98	13.98	0.3792	0.4	2.685	2.685
R3.2 & B3.3	17.41	17.41	0.4682	0.4	3.681	3.681
B3.4	23.97	23.97	0.5314	0.4	3.420	3.420
B3.5	31.22	31.22	0.7736	0.4	3.365	3.365
R4	30.71	30.71	0.3280	0.4	3.287	3.287
R4.1	29.57	29.57	0.6563	0.4	3.784	3.784
B4.2	30.67	30.67	0.6271	0.4	3.155	3.155
R5	20.76	20.76	0.3968	0.4	2.740	2.740
B5	13.78	13.78	0.3543	0.4	2.834	2.834
R6	21.71	21.71	0.3264	0.4	2.717	2.717
B6	28.67	28.67	0.3621	0.4	4.431	4.431
R7	15.31	15.31	0.3599	0.4	3.637	3.637
B7	12.69	12.69	0.3444	0.4	3.295	3.295
R8	12.39	12.39	0.3686	0.4	2.011	2.011
B8	22.28	22.28	0.4206	0.4	2.893	2.893
R9	12.66	12.66	0.4284	0.4	2.552	2.552
B9	15.40	15.40	0.3440	0.4	2.447	2.447
B10	8.50	8.50	0.0595	0.4	1.510	1.510
R10 & R11	20.72	20.72	0.4226	0.4	2.790	2.790
R12	31.73	31.73	0.5000	0.4	1.685	1.685
B13	30.64	30.64	0.4790	0.4	2.073	2.073
B14	31.96	31.96	0.5410	0.4	1.780	1.780

TABLE 3.10

SUMMARY OF CALIBRATED PARAMETERS

HOLTAN INFILTRATION EQUATION

PHU	SMAX (mm)	SA (mm)	a (mm/hr/ mm**1.4)	GI (%)	fc (mm/hr)	DCON (mm/hr)
R1	8.40	8.40	0.2830	0.4	2.975	2.975
R2	7.97	7.97	0.3715	0.4	2.220	2.220
B2	11.83	11.83	0.7837	0.4	2.793	2.793
R3	9.33	9.33	0.4848	0.4	2.642	2.642
R3.1	4.61	4.61	0.3792	0.4	2.014	2.014
R3.2 & B3.3	5.75	5.75	0.4682	0.4	2.761	2.761
B3.4	7.91	7.91	0.5314	0.4	2.565	2.565
B3.5	10.30	10.30	0.7736	0.4	2.524	2.524
R4	10.14	10.14	0.3280	0.4	2.465	2.465
R4.1	9.76	9.76	0.6563	0.4	2.838	2.838
B4.2	10.12	10.12	0.6271	0.4	2.366	2.366
R5	6.85	6.85	0.3968	0.4	2.055	2.055
B5	4.55	4.55	0.3543	0.4	2.125	2.125
R6	7.17	7.17	0.3264	0.4	2.038	2.038
B6	9.46	9.46	0.3621	0.4	3.323	3.323
R7	5.06	5.06	0.3599	0.4	2.728	2.728
B7	4.19	4.19	0.3444	0.4	2.471	2.471
R8	4.09	4.09	0.3686	0.4	1.508	1.508
B8	7.35	7.35	0.4206	0.4	2.170	2.170
R9	4.18	4.18	0.4284	0.4	1.914	1.914
B9	5.08	5.08	0.3440	0.4	1.835	1.835
B10	2.81	2.81	0.0595	0.4	1.133	1.133
R10 & R11	6.84	6.84	0.4226	0.4	2.093	2.093
R12	10.47	10.47	0.5000	0.4	1.264	1.264
B13	10.11	10.11	0.4790	0.4	1.555	1.555
B14	10.55	10.55	0.5410	0.4	1.335	1.335

TABLE 3.11

SUMMARY OF WATERSHED PARAMETERS

HYMO MODEL

PHU	Area (sq km)	Net Elevation Difference (m)	Length of Watercourse (m)	Equivalent Slope (%)
R1	1.88	3.67	5325	0.06891
R2	13.75	32.70	15150	0.21583
B2	27.38	5.11	11325	0.04508
R3	10.25	18.91	5050	0.37452
R3.1	4.90	11.60	4675	0.24814
R3.2 & B3.3	14.20	19.86	11000	0.18057
B3.4	12.80	19.21	10000	0.19207
B3.5	8.85	19.68	12900	0.15258
R4	23.50	9.91	10910	0.09085
R4.1	30.60	22.27	12942	0.17209
B4.2	65.90	18.66	22274	0.08377
R5	21.25	3.52	7385	0.04765
B5	19.59	22.28	12500	0.17825
R6	2.63	16.12	3170	0.50846
B6	5.18	7.00	3385	0.20668
R7	21.75	5.06	10155	0.04984
B7	12.78	25.46	10480	0.24293
R8	1.38	2.62	2000	0.13087
B8	31.97	12.25	15460	0.07922
R9	5.38	7.86	4590	0.17129
B9	32.12	24.22	13865	0.17471
B10	14.50	5.88	6195	0.09490
R10 & R11	7.35	6.27	3675	0.17049
R12	15.45	12.12	9000	0.13464
B13	15.05	11.89	8080	0.14714
B14	37.43	22.09	13055	0.16923

TABLE 3.12

SUMMARY OF TIME TO PEAK ANALYSIS

PHU	MTC Tc (hr)	MTC Tp (hr)	SCS Tc (hr)	SCS Tp (hr)	HYMO Tp (hr)
R1	5.12	3.41	3.97	2.64	3.20
R2	9.71	6.47	8.35	5.56	8.46
B2	8.86	5.91	5.72	3.81	5.43
R3	2.92	1.95	1.98	1.32	2.37
R3.1	3.16	2.11	2.19	1.46	2.26
R3.2 & B3.3	7.12	4.75	4.78	3.19	4.12
B3.4	6.46	4.31	4.34	2.89	4.12
B3.5	9.06	6.04	5.77	3.85	4.40
R4	7.71	5.14	6.19	4.13	7.08
B4.1	7.83	5.22	5.52	3.68	5.96
B4.2	14.42	9.60	11.07	7.38	11.98
R5	5.99	4.00	5.88	3.92	8.35
B5	7.86	5.24	5.31	3.54	5.11
R6	1.98	1.32	1.23	0.82	1.23
B6	2.36	1.57	1.83	1.22	2.30

TABLE 3.12 (Cont'd)

SUMMARY OF TIME TO PEAK ANALYSIS

PHU	MTC		SCS		HYMO
	Tc (hr)	Tp (hr)	Tc (hr)	Tp (hr)	Tp (hr)
R7	8.15	5.43	7.38	4.92	8.96
B7	6.46	4.31	4.11	2.74	3.74
R8	1.74	1.16	1.46	0.97	1.68
B8	10.88	7.25	8.54	5.69	9.05
R9	3.31	2.21	2.49	1.66	2.75
B9	8.33	5.55	5.79	3.86	6.12
R10 & R11	2.57	1.71	2.10	1.40	2.83
B10	4.55	3.03	3.94	2.63	5.19
R12	6.95	4.63	5.85	3.90	6.65
B13	5.42	3.61	4.08	2.72	4.61
R14	7.77	5.18	5.60	3.73	6.38

TABLE 3.13

SUMMARY OF K AND Tp

FOR VARYING B VALUES

PHU	Tp (hr)	K (hr) B=250	B=300	B=350	B=400
R1	3.20	4.48	3.52	2.88	2.50
R2	8.46	11.84	9.31	7.61	6.60
B2	5.43	7.60	5.97	4.89	4.24
R3	2.37	3.32	2.61	2.13	1.85
R3.1	2.26	3.16	2.49	2.03	1.76
B3.3 & R3.2	4.47	6.26	4.92	4.02	3.49
B3.4	4.12	5.77	4.53	3.71	3.21
B3.5	4.40	6.16	4.84	3.96	3.43
R4	7.08	9.91	7.79	6.37	5.52
R4.1	5.96	8.34	6.56	5.36	4.65
B4.2	11.98	16.77	13.18	10.78	9.34
R5	8.35	11.69	9.19	7.52	6.51
B5	5.11	7.15	5.62	4.60	3.99
R6	1.23	1.72	1.35	1.11	0.96
B6	2.30	3.22	2.53	2.07	1.79
R7	8.96	12.54	9.86	8.06	6.99
B7	3.74	5.24	4.11	3.37	2.92
R8	1.68	2.35	1.85	1.51	1.31
B8	9.05	12.67	9.96	8.15	7.06
R9	2.75	3.85	3.03	2.48	2.15
B9	6.12	8.57	6.73	5.51	4.77
B10	5.19	7.27	5.71	4.67	4.05
R10 & R11	2.83	3.96	3.11	2.55	2.21
R12	6.65	9.31	7.32	5.99	5.19
B13	4.61	6.45	5.07	4.15	3.60
B14	6.38	8.93	7.02	5.74	4.98

TABLE 3.14

SUMMARY OF RESULTS

7-DAY 100 YEAR SNOWMELT

Points Of Interest	Peak Flood Flows B=250 (cms)	B=300 (cms)	B=350 (cms)	B=400 (cms)
Outflow from R1	202.2	219.7	234.2	243.5
Outflow from R3*	234.8	262.0	281.7	297.0
Outflow from R5	156.8	173.0	187.0	198.8
Inflow into R5	150.9	170.4	187.0	198.1
Outflow from B5	31.7	37.2	41.9	45.3
Outflow from R6	137.6	158.5	176.6	189.1
Inflow into R6	136.8	157.2	174.1	185.5
Outflow from B6	13.7	15.5	16.9	17.8
Outflow from R7	135.7	156.2	173.3	184.8
Inflow into R7	180.8	205.7	227.6	243.4
Outflow from B7	30.4	37.0	43.2	48.5
Outflow from R8	170.7	197.0	220.5	237.5
Inflow into R8	174.5	199.4	221.4	237.3
Outflow from B8	30.3	36.4	42.4	47.1
Outflow from R9	144.2	163.0	179.0	190.1
Outflow from R10	144.6	167.7	188.4	203.2
Outflow from B9	47.7	57.2	66.2	73.5
Outflow from B10	27.0	31.6	35.5	38.3
Outflow from R12	89.7	102.6	114.2	123.8

* Approximate location of Streamflow Station No. 02LB008

<u>Note</u>: Using the SCS 24-hour distribution to spatially distribute the 7-day 100 year snowmelt value.

TABLE 3.15

SUMMARY OF SNOWMELT ANALYSIS

100 YEAR EVENT

Points Of Interest	Peak Flood Flows For B=300 2-Day (cms)	5-Day (cms)	7-Day (cms)	10-Day (cms)
Outflow from R1	107.4	261.6	300.5	315.0
Outflow from R3*	127.3	311.5	357.1	419.2
Outflow from R5	81.6	203.8	233.4	233.2
Inflow into R5	79.3	195.9	232.6	221.6
Outflow from B5	20.7	44.1	46.0	50.3
Outflow from R6	74.3	184.6	217.0	207.6
Inflow into R6	74.6	182.5	212.5	218.1
Outflow from B6	9.8	21.7	20.2	12.1
Outflow from R7	74.1	181.4	211.2	216.8
Inflow into R7	102.5	238.4	272.1	269.4
Outflow from B7	17.5	35.1	40.6	38.7
Outflow from R8	97.2	226.1	259.5	253.3
Inflow into R8	99.5	226.2	254.9	263.0
Outflow from B8	19.0	42.0	46.9	49.1
Outflow from R9	80.5	184.2	208.0	213.8
Outflow from R10	85.6	187.3	209.3	207.4
Outflow from B9	29.7	62.5	70.5	68.2
Outflow from B10	17.0	35.7	37.6	39.8
Outflow from R12	54.1	121.8	131.5	143.4

* Approximate location of Streamflow Station No. 02LB008

Notes:

1) Using the calibrated Holtan infiltration parameters given in Table 3.10.

2) Using the SCS 24-hour Distribution to spatially distribute the 100 year snowmelt amount.

TABLE 3.16

SUMMARY OF RESULTS

7-DAY SNOWMELT

USING CALIBRATED PARAMETERS FOR HOLTAN EQUATION

Points Of Interest	Peak Flood Flows For B=300 100 Yr (cms)	50 Yr (cms)	25 Yr (cms)	10 Yr (cms)	5 Yr (cms)	2 Yr (cms)
Outflow from R1	357.1	327.3	297.0	254.9	223.1	174.3
Outflow from R3*	368.9	338.4	308.3	268.5	236.7	188.9
Outflow from R5	215.4	197.0	178.8	155.1	136.4	109.2
Inflow into R5	199.7	182.8	165.7	143.8	126.4	101.7
Outflow from B5	30.5	28.2	25.9	22.9	20.4	16.8
Outflow from R6	179.5	164.6	149.9	129.9	114.4	92.1
Inflow into R6	180.0	164.5	149.4	129.6	114.2	92.0
Outflow from B6	10.8	10.0	9.2	8.2	7.3	6.0
Outflow from R7	177.3	162.3	147.9	128.4	113.2	91.2
Inflow into R7	199.6	183.7	167.6	146.9	130.9	106.8
Outflow from B7	22.6	21.0	19.3	17.0	15.2	12.5
Outflow from R8	185.1	170.2	155.2	136.0	121.1	98.9
Inflow into R8	185.9	171.4	156.8	137.5	122.6	100.0
Outflow from B8	37.0	34.2	31.4	27.6	24.6	20.1
Outflow from R9	148.9	137.2	125.4	109.9	98.0	79.9
Outflow from R10	147.6	136.1	124.7	109.4	97.5	80.0
Outflow from B9	45.6	42.2	38.8	34.2	30.5	25.0
Outflow from B10	22.6	20.9	19.2	17.0	15.2	12.5
Outflow from R12	88.5	81.8	75.1	65.9	58.7	47.8

* Approximate location of Streamflow Station No. 02LB008

Notes:

1) Using the calibrated Holtan infiltration parameters of SMAX, SA and a given in Table 3.10, and GI = fc = DCON = 0.20.

2) Total snowmelt depths increased by 50%.

TABLE 3.17

SUMMARY OF SUB-BASIN PARAMETERS

HYMO MODEL

PHU	Area (sq km)	Net Elevation Difference (m)	Length of Watercourse (m)	Equivalent Slope (%)
Basin 5				
R5.1	1.16	7.76	2712	0.2860
R5.2 & R5.3	2.02	5.83	2450	0.2378
R5.4	3.50	21.31	3696	0.5767
B5.5	6.00	5.07	5240	0.0967
R5.6	0.89	13.93	1967	0.7082
B5.7	1.40	15.69	2600	0.6036
R5.8	0.36	4.90	1300	0.3772
R5.9	0.53	9.46	1424	0.6643
R5.10	1.39	2.38	1450	0.1642
B5.11	0.99	4.17	3000	0.1389
B5.12	0.64	2.80	1302	0.2153
B5.13	0.71	23.95	1950	1.2281
Basin 6				
R6.1	0.29	1.67	1176	0.1416
R6.2	2.99	8.70	2748	0.3167
B6.3	1.47	7.15	800	0.8938
B6.4	0.43	8.14	1350	0.6026
Basin 7				
R7.1	1.22	7.76	2544	0.3052
R7.2	0.90	3.70	1365	0.2709
R7.3	2.59	24.30	3654	0.6651
R7.4	0.24	7.70	1068	0.7208
B7.5	0.62	3.04	1350	0.2253
R7.6	0.88	7.33	2288	0.3203
B7.7	1.62	5.44	2550	0.2133
B7.8	1.45	5.44	2400	0.2267
B7.9	2.86	5.35	3495	0.1531
B7.10	0.41	11.90	1100	1.0818

TABLE 3.17 (Cont'd)

SUMMARY OF SUB-BASIN PARAMETERS

HYMO MODEL

PHU	Area (sq km)	Net Elevation Difference (m)	Length of Watercourse (m)	Equivalent Slope (%)
Basin 8				
R8.1	3.13	8.93	4720	0.1891
R8.2	5.66	7.26	4340	0.1672
R8.3	11.83	1.79	7224	0.0248
B8.4	11.35	5.21	5990	0.0870
Basin 9				
R9.1 & R9.10	0.81	0.58	1740	0.0336
R9.2	0.41	5.12	3990	0.1282
R9.3	2.31	7.99	1944	0.4111
R9.4	4.54	3.58	3200	0.1119
R9.5	0.68	3.77	1880	0.2004
B9.6	0.86	1.85	1752	0.1054
B9.7	3.84	5.30	3800	0.1395
B9.8	5.06	3.57	3702	0.0963
R9.9	1.63	5.22	2920	0.1787
R9.11	2.67	3.45	2904	0.1187
B9.12	5.75	9.03	5400	0.1672
R9.13	1.00	6.18	2584	0.2392
B9.14	0.87	1.23	3648	0.0338
B9.15	1.75	9.98	2100	0.4753

TABLE 3.18

SUMMARY OF TIME TO PEAK ANALYSIS

PHU	MTO Tc (hr)	MTO Tp (hr)	SCS Tc (hr)	SCS Tp (hr)	HYMO Tp (hr)
Basin 5					
R5.1	2.057	1.371	1.364	0.909	1.210
R5.2 & R5.3	1.824	1.216	1.354	0.902	1.505
R5.4	2.182	1.454	1.321	0.881	1.309
B5.5	4.189	2.792	3.438	2.291	3.817
R5.6	1.278	0.852	0.751	0.501	0.678
B5.7	1.667	1.111	0.990	0.660	0.896
R5.8	1.049	0.699	0.696	0.464	0.625
R5.9	0.987	0.658	0.600	0.400	0.552
R5.10	1.207	0.805	1.043	0.695	1.393
B5.11	2.671	1.780	1.946	1.297	1.655
B5.12	1.109	0.739	0.864	0.576	0.955
B5.13	1.161	0.774	0.604	0.402	0.492
Basin 6					
R6.1	1.179	0.786	0.939	0.626	0.897
R6.2	1.858	1.238	1.325	0.883	1.523
B6.3	0.472	0.315	0.343	0.229	0.554
B6.4	0.974	0.649	0.598	0.399	0.535
Basin 7					
R7.1	1.895	1.263	1.266	0.844	1.171
R7.2	1.074	0.716	0.821	0.547	0.960
R7.3	2.161	1.440	1.240	0.826	1.120
R7.4	0.788	0.525	0.466	0.311	0.391
B7.5	1.144	0.762	0.874	0.582	0.936
R7.6	1.744	1.163	1.145	0.763	1.013
B7.7	1.984	1.322	1.456	0.970	1.500
B7.8	1.865	1.243	1.357	0.905	1.390
B7.9	2.745	1.830	2.109	1.405	2.240
B7.10	0.710	0.473	0.408	0.272	0.382

TABLE 3.18 (Cont'd)

SUMMARY OF TIME TO PEAK ANALYSIS

PHU	MTO		SCS		HYMO
	Tc (hr)	Tp (hr)	Tc (hr)	Tp (hr)	Tp (hr)
Basin 8					
R8.1	3.522	2.347	2.450	1.633	2.259
R8.2	3.128	2.085	2.408	1.605	2.774
R8.3	7.087	4.724	7.438	4.958	9.468
B8.4	4.589	3.059	3.969	2.645	4.992
Basin 9					
R9.1 & R9.10	2.100	1.400	2.210	1.473	2.596
R9.2	3.943	2.628	2.500	1.666	1.436
R9.3	1.280	0.853	0.918	0.612	1.144
R9.4	2.555	1.703	2.223	1.482	2.888
R9.5	1.615	1.077	1.179	0.786	1.108
B9.6	1.672	1.114	1.431	0.954	1.564
B9.7	2.952	1.968	2.331	1.554	2.602
B9.8	3.013	2.008	2.635	1.756	3.319
R9.9	2.345	1.563	1.730	1.153	1.705
R9.11	2.416	1.610	2.016	1.344	2.349
B9.12	3.886	2.590	2.850	1.899	2.954
R9.13	2.062	1.374	1.407	0.938	1.242
B9.14	4.367	2.910	3.900	2.600	3.219
B9.15	1.381	0.921	0.921	0.614	1.008

TABLE 3.19

RECESSION CONSTANTS FOR SELECTED WATERSHED PARAMETERS

PHU	Tp (hr)	Recession Constants (K) B=300	B=350	B=400
Basin 5				
R5.1	1.210	1.33	1.09	0.94
R5.2 & R5.3	1.505	1.66	1.35	1.17
R5.4	1.309	1.44	1.18	1.02
B5.5	3.817	4.20	3.44	2.98
R5.6	0.678	0.75	0.61	0.53
B5.7	0.896	0.99	0.81	0.70
R5.8	0.625	0.69	0.56	0.49
R5.9	0.552	0.61	0.50	0.43
R5.10	1.393	1.53	1.25	1.09
B5.11	1.655	1.82	1.49	1.29
B5.12	0.955	1.05	0.86	0.74
B5.13	0.492	0.54	0.44	0.38
Basin 6				
R6.1	0.897	0.99	0.81	0.70
R6.2	1.523	1.68	1.37	1.19
B6.3	0.554	0.61	0.50	0.43
B6.4	0.535	0.59	0.48	0.42
Basin 7				
R7.1	1.171	1.29	1.05	0.91
R7.2	0.960	1.06	0.86	0.75
R7.3	1.120	1.23	1.01	0.87
R7.4	0.391	0.43	0.35	0.30
B7.5	0.936	1.03	0.84	0.73
R7.6	1.013	1.11	0.91	0.79
B7.7	1.500	1.65	1.35	1.17
B7.8	1.390	1.53	1.25	1.08
B7.9	2.240	2.46	2.02	1.75
B7.10	0.382	0.42	0.34	0.30

TABLE 3.19 (Cont'd)

RECESSION CONSTANTS FOR SELECTED WATERSHED PARAMETERS

PHU	Tp (hr)	Recession Constants (K) B=300	B=350	B=400
Basin 8				
R8.1	2.259	2.48	2.03	1.76
R8.2	2.774	3.05	2.50	2.16
R8.3	9.468	10.41	8.52	7.39
B8.4	4.992	5.49	4.49	3.89
Basin 9				
R9.1 & R9.10	2.596	2.86	2.34	2.02
R9.2	1.436	1.58	1.29	1.12
R9.3	1.144	1.26	1.03	0.89
R9.4	2.888	3.18	2.60	2.25
R9.5	1.108	1.22	1.00	0.86
B9.6	1.564	1.72	1.41	1.22
B9.7	2.602	2.86	2.34	2.03
B9.8	3.319	3.65	2.99	2.59
R9.9	1.705	1.88	1.53	1.33
R9.11	2.349	2.58	2.11	1.83
B9.12	2.954	3.25	2.66	2.30
R9.13	1.242	1.37	1.12	0.97
B9.14	3.219	3.54	2.90	2.51
B9.15	1.008	1.11	0.91	0.79

TABLE 3.20

FINAL PEAK FLOWS

7-DAY SNOWMELT EVENTS

Event (yr)	Outflow From PHU	Final Peak Flows (cms)
100	Basin 5	24.55
	Basin 6	7.53
	Basin 7	27.24
	Basin 8	36.90
	Basin 9	50.89
50	Basin 5	22.68
	Basin 6	6.97
	Basin 7	25.26
	Basin 8	33.95
	Basin 9	47.20
25	Basin 5	20.84
	Basin 6	6.43
	Basin 7	23.25
	Basin 8	30.98
	Basin 9	43.44
10	Basin 5	18.35
	Basin 6	5.66
	Basin 7	20.53
	Basin 8	27.25
	Basin 9	38.31
5	Basin 5	16.34
	Basin 6	5.07
	Basin 7	18.46
	Basin 8	24.27
	Basin 9	34.18
2	Basin 5	13.39
	Basin 6	0.85
	Basin 7	15.23
	Basin 8	19.85
	Basin 9	27.27

TABLE 3.21

ROUTED PEAK FLOWS

7-DAY 100 YEAR SNOWMELT

Outflow From PHU	Routed Peak Flows (cms)
Basin 5	
B5.12	1.53
B5.11	2.26
B5.11 & B5.12	3.79
R5.10	3.28
R5.9	3.14
B5.13	1.30
R5.8	3.94
B5.7	3.38
R5.6	2.10
B5.5	10.43
R5.4	16.85
R5.3 & R5.2	21.07
R5.1	24.55
Basin 6	
B6.4	0.84
B6.3	2.96
R6.2	6.94
R6.1	7.53
Basin 7	
B7.9	6.02
B7.8	3.42
B7.7	3.78
R7.6	15.09
B7.5	1.49
R7.4	16.91
B7.10	0.58
R7.3	23.45
R7.2	25.15
R7.1	27.24

TABLE 3.21 (Cont'd)

ROUTED PEAK FLOWS

7-DAY 100 YEAR SNOWMELT

Outflow From PHU	Routed Peak Flows (cms)
Basin 8	
B8.4	17.68
R8.3	28.74
R8.2	34.49
R8.1	36.90
Basin 9	
B9.7	7.75
B9.6	1.99
R9.5	10.96
B9.8	9.31
R9.4	27.72
R9.3	31.71
R9.2	29.22
B9.12	11.03
R9.11	15.43
B9.14	1.59
R9.13	3.43
R9.10 & R9.1	19.06
B9.15	4.18
R9.9	3.82
R9.1, R9.2, R9.9 & R10	50.89

TABLE 3.22

HYDROLOGIC SOIL GROUPS

RAINFALL ANALYSIS

PHU	Hydrologic Soil Groups (% of Area)			
	B	B/C	C	D
Basin 5				
R5.1	26.40	0.00	37.50	36.10
R5.2 & R5.3	20.00	0.00	38.40	41.60
R5.4	38.50	0.00	59.00	2.50
B5.5	26.00	0.00	74.00	0.00
R5.6	38.00	0.00	53.00	9.00
B5.7	34.50	0.00	59.80	5.70
R5.8	23.00	0.00	63.50	13.50
R5.9	21.20	0.00	73.00	5.80
R5.10	17.20	0.00	81.70	1.10
B5.11	9.70	0.00	90.30	0.00
B5.12	22.50	0.00	77.50	0.00
B5.13	22.70	0.00	77.30	0.00
Basin 6				
R6.1	33.30	0.00	0.00	66.70
R6.2	59.00	0.00	36.00	5.00
B6.3	85.00	0.00	15.00	0.00
B6.4	40.70	0.00	59.30	0.00
Basin 7				
R7.1	16.00	0.00	3.90	80.10
R7.2	17.90	0.00	16.10	66.00
R7.3	39.10	0.00	47.20	13.70
R7.4	66.70	0.00	6.70	26.70
B7.5	82.00	0.00	5.10	12.90
R7.6	21.80	0.00	67.30	10.90
B7.7	9.90	0.00	83.00	7.10
B7.8	4.40	0.00	95.60	0.00
B7.9	35.40	0.00	63.00	1.60
B7.10	3.80	0.00	88.50	7.70

TABLE 3.22 (Cont'd)

HYDROLOGIC SOIL GROUPS

RAINFALL ANALYSIS

PHU	Hydrologic Soil Groups (% of Area)			
	B	B/C	C	D
Basin 8				
R8.1	16.90	0.00	72.30	10.80
R8.2	14.50	0.00	80.40	5.10
R8.3	13.30	0.00	84.90	1.80
B8.4	11.30	0.00	86.40	2.30
Basin 9				
R9.1 & R9.10	0.00	0.00	73.00	27.00
R9.2	18.20	0.00	63.60	18.20
R9.3	23.30	0.00	59.40	17.30
R9.4	4.20	19.20	64.60	11.90
R9.5	0.00	14.30	76.20	9.50
B9.6	0.00	0.00	99.99	0.01
B9.7	0.00	0.00	96.00	4.00
B9.8	1.30	0.00	95.50	3.20
R9.9	41.20	0.00	19.60	39.20
R9.11	4.80	0.00	20.50	74.70
B9.12	8.00	42.00	50.00	0.00
R9.13	11.30	0.00	88.70	0.00
B9.14	43.00	0.00	57.00	0.00
B9.15	9.20	0.00	80.80	10.00

TABLE 3.23

LAND USE

RAINFALL ANALYSIS

PHU	Land Use (% of Area)			
	Residential	Crop	Pasture	Wood
Basin 5				
R5.1	0.00	18.10	10.30	71.60
R5.2 & R5.3	3.00	56.50	32.00	8.50
R5.4	0.00	52.60	29.70	17.70
B5.5	0.00	52.50	29.50	18.00
R5.6	0.00	25.80	14.60	59.60
B5.7	0.00	59.30	32.90	7.90
R5.8	3.00	61.00	36.00	0.00
R5.9	1.70	62.30	36.00	0.00
R5.10	2.20	57.60	32.40	7.90
B5.11	0.00	64.00	36.00	0.00
B5.12	28.10	45.30	26.60	0.00
B5.13	0.00	57.70	32.40	9.90
Basin 6				
R6.1	13.80	55.20	31.00	0.00
R6.2	3.00	48.50	27.40	21.10
B6.3	0.00	51.70	28.60	19.70
B6.4	4.60	60.50	34.90	0.00
Basin 7				
R7.1	0.00	61.50	34.40	4.10
R7.2	0.00	60.00	34.40	5.60
R7.3	5.40	41.30	23.20	30.10
R7.4	21.00	50.00	29.00	0.00
B7.5	6.50	58.00	32.30	3.20
R7.6	0.00	58.00	31.80	10.20
B7.7	0.00	63.60	35.80	0.60
B7.8	0.00	63.40	35.20	1.40
B7.9	0.00	51.40	29.00	19.60
B7.10	0.00	22.00	12.20	65.80

TABLE 3.23 (Cont'd)

LAND USE

RAINFALL ANALYSIS

PHU	Land Use (% of Area)			
	Residential	Crop	Pasture	Wood
Basin 8				
R8.1	0.00	57.00	32.10	10.90
R8.2	3.40	44.30	24.90	27.40
R8.3	1.50	27.00	15.20	56.40
B8.4	0.00	60.60	34.10	5.30
Basin 9				
R9.1 & R9.10	0.00	64.00	36.00	0.00
R9.2	0.00	46.30	24.40	29.30
R9.3	7.80	44.60	25.10	22.50
R9.4	7.80	46.00	25.80	20.50
R9.5	7.40	59.30	33.30	0.00
B9.6	5.80	59.30	33.70	1.20
B9.7	3.10	62.00	34.90	0.00
B9.8	0.00	64.00	36.00	0.00
R9.9	0.00	62.60	35.00	2.50
R9.11	0.00	36.40	20.50	43.10
B9.12	8.50	40.20	22.60	28.70
R9.13	0.00	63.00	35.00	2.00
B9.14	5.80	43.70	24.10	26.40
B9.15	5.00	50.00	28.00	17.00

TABLE 3.24

SOIL COVER CURVE NUMBERS

RAINFALL ANALYSIS

PHU	Area (sq km)	Soil Cover Curve Numbers (CN) Cond. II
Basin 5		
R5.1	1.16	74
R5.2 & R5.3	2.02	81
R5.4	3.50	77
B5.5	6.00	78
R5.6	0.89	73
B5.7	1.40	78
R5.8	0.36	80
R5.9	0.53	80
R5.10	1.39	79
B5.11	0.99	81
B5.12	0.64	80
B5.13	0.71	79
	19.59	
Basin 6		
R6.1	0.29	81
R6.2	2.99	74
B6.3	1.47	72
B6.4	0.43	78
	5.18	
Basin 7		
R7.1	1.22	83
R7.2	0.90	82
R7.3	2.59	76
R7.4	0.24	77
B7.5	0.62	75
R7.6	0.88	79
B7.7	1.62	81
B7.8	1.45	81
B7.9	2.86	77
B7.10	0.41	76
	12.79	

TABLE 3.24 (Cont'd)

SOIL COVER CURVE NUMBERS

RAINFALL ANALYSIS

PHU	Area (sq km)	Soil Cover Curve Numbers (CN) Cond. II
Basin 8		
R8.1	3.13	80
R8.2	5.66	78
R8.3	11.83	75
B8.4	11.35	80
	31.97	
Basin 9		
R9.1 & R9.10	0.81	83
R9.2	0.41	78
R9.3	2.31	78
R9.4	4.54	79
R9.5	0.68	81
B9.6	0.86	81
B9.7	3.84	81
B9.8	5.06	81
R9.9	1.63	80
R9.11	2.67	81
B9.12	5.75	76
R9.13	1.00	80
B9.14	0.87	75
B9.15	1.75	80
	32.18	

TABLE 3.25

12 HOUR RAINFALL DEPTHS

Event (yr)	Rainfall Depths (mm)
2	43.60
5	59.58
10	70.41
25	83.95
50	93.97
100	103.99

TABLE 3.26

FINAL PEAK FLOWS

RAINFALL ANALYSIS

Event (yr)	Outflow From PHU	Final Peak Flows (cms)
100	Basin 5	32.20
	Basin 6	9.12
	Basin 7	26.19
	Basin 8	20.95
	Basin 9	43.49
50	Basin 5	27.10
	Basin 6	7.67
	Basin 7	22.46
	Basin 8	17.67
	Basin 9	36.59
25	Basin 5	22.00
	Basin 6	6.29
	Basin 7	18.86
	Basin 8	14.58
	Basin 9	30.47
10	Basin 5	15.77
	Basin 6	4.47
	Basin 7	13.88
	Basin 8	10.56
	Basin 9	22.91
5	Basin 5	11.27
	Basin 6	3.20
	Basin 7	9.97
	Basin 8	7.48
	Basin 9	17.81
2	Basin 5	5.41
	Basin 6	1.47
	Basin 7	5.04
	Basin 8	3.71
	Basin 9	9.17

TABLE 3.27

ROUTED PEAK FLOWS

100 YEAR RAINFALL EVENT

Outflow From PHU	Routed Peak Flows (cms)
Basin 5	
B5.12	1.65
B5.11	2.38
B5.11 & B5.12	3.96
R5.10	7.25
R5.9	8.30
B5.13	1.42
R5.8	10.51
B5.7	3.34
R5.6	5.27
B5.5	8.43
R5.4	14.05
R5.3 & R5.2	26.53
R5.1	32.20
Basin 6	
B6.4	0.88
B6.3	2.29
R6.2	8.78
R6.1	9.12
Basin 7	
B7.9	5.31
B7.8	3.65
B7.7	4.02
R7.6	14.41
B7.5	1.31
R7.4	16.17
B7.10	0.55
R7.3	21.83
R7.2	23.59
R7.1	26.19

TABLE 3.27 (Cont'd)

ROUTED PEAK FLOWS

100 YEAR RAINFALL EVENT

Outflow From PHU	Routed Peak Flows (cms)
Basin 8	
B8.4	14.07
R8.3	17.84
R8.2	20.02
R8.1	20.95
Basin 9	
B9.7	7.66
B9.6	2.11
R9.5	10.99
B9.8	8.76
R9.4	25.49
R9.3	28.52
R9.2	23.90
B9.12	8.79
R9.11	13.42
B9.14	1.23
R9.13	2.97
R9.10 & R9.1	16.08
B9.15	4.49
R9.9	8.24
R9.1, R9.2, R9.9 & R9.10	43.49

TABLE 3.28

SIMULATED RAINFALL PEAK FLOWS

AT BOURGET STREAMFLOW STATION

Storm Event (yr)	Rainfall Peak Flows (cms)
2	14.93
5	33.83
10	49.19
25	71.29
50	89.36
100	107.61

TABLE 3.29

RESULTS OF MEAN DAILY AND MAXIMUM INSTANTANEOUS PEAK FLOWS

DELISLE RIVER REGIONAL FLOOD FREQUENCY METHOD

Return Period (yr)	Mean Daily Peak Flow (cms)	Maximum Instantaneous Peak Flow (cms)
2	101.5	113.9
5	115.5	129.6
10	126.0	141.4
25	136.2	152.8
50	145.4	163.1
100	154.2	173.0

TABLE 3.30

RESULTS OF MEAN DAILY AND MAXIMUM INSTANTANEOUS PEAK FLOWS

REGIONAL REGRESSION METHOD

Return Period (yr)	Mean Daily Peak Flow (cms)	Maximum Instantaneous Peak Flow (cms)
2	82.9	93.0
5	106.9	119.9
10	122.1	137.0
25	139.5	156.5
50	151.5	170.0
100	162.6	182.4

Note: Drainage Area (DA) = 458 sq km
Area Controlled by Lakes and Swamps (ACLS) = 1.44
Equivalent Slope (EQSLP) = 0.0004316 m/m
Mean Annual Runoff (MAR) = 340 mm
Mean Annual Snowfall (MAS) = 180 cm

TABLE 3.31

WATERSHED PARAMETERS

WATERSHED CLASSIFICATION METHOD

Total Drainage Area (sq km)	Net Fall (m)	Main Channel Net Length (m)	Equivalent Slope (m/m)	Base Class With Soil/Land Use Curve Number
458.00	31.51	73000	0.0004316	9.55

Adjustments to Base Class				Net Adjustment
Slope	Shape	Retention	Precipitation	
-0.70	-0.55	-1.05	-0.10	-1.15

Net Class	Net Coefficient	Instantaneous Peak Flow, Q25 (cms)
8.4	2.12	210.00

Note: Procedures to modify base class value to net class

1) Determine the adjustments for slope, shape and retention. Add the largest positive adjustment and the largest negative adjustment to obtain a net adjustment for slope, shape and retention.

2) Determine the adjustment for precipitation.

3) Add 1) and 2) to obtain the net adjustment.

4) Add net adjustment to the base class to obtain the net class.

TABLE 3.32

INSTANTANEOUS PEAK FLOWS

WATERSHED CLASSIFICATION METHOD

Return Period (yr)	Ratio To 25 Year	Instantaneous Peak Flows Qp (cms)
2	0.41	87.00
2.33	0.46	96.60
5	0.65	136.50
10	0.80	168.00
25	1.00	210.00
50	1.15	241.50
100	1.29	270.90

Note: The 2 year ratio is obtained from probability plot.

TABLE 3.33

RESULTS OF MEAN DAILY AND MAXIMUM INSTANTANEOUS PEAK FLOWS

INDEX FLOOD METHOD

Return Period (yr)	Mean Daily Peak Flow (cms)	Maximum Instantaneous Peak Flow (cms)
2	123.5	138.6
5	154.4	173.2
10	185.3	207.9
25	222.3	249.4
50	253.2	284.1
100	277.9	311.8

TABLE 3.34

RECORDED DISCHARGES AT STATION NO. 02LB008

BEAR BROOK NEAR BOURGET

Year	Maximum Daily Discharge (cms) Spring		Summer		
1949	108.00		5.75		(May)
1950	136.00		Incomplete		
1951	125.00		Incomplete		
1952	116.00		*		
1953	78.70		*		
1954	*		*		
1955	136.00	ICE	*		
1956	102.00		Incomplete		
1957	41.30		*		
1958	72.50		*		
1959	119.00		*		
1960	106.00		*		
1961	64.60	ICE	*		
1962	75.00		Incomplete		
1963	122.00	ICE	Incomplete		
1964	36.80		*		
1965	35.10	ICE	*		
1966	65.10	ICE	*		
1967	126.00		*		
1968	90.60		*		
1969	70.50	ICE	*		
1970	*		*		
1971	*		*		
1972	*		*		
1973	*		*		
1974	*		*		
1975	*		*		
1976	219.00		16.10	E	(May)
1977	150.00	ICE	41.10	A	(Oct)
1978	214.00	ICE	7.14		(May)
1979	265.00		35.90		(Nov)
1980	116.00	ICE	23.80		(Oct)
1981	270.00	**	76.40		(Jun)
1982	153.00	ICE	17.70		(Nov)
1983	85.00	ICE	49.90		(May)
1984	172.00		19.30		(May)
1985	105.00	ICE	4.80		(May)
1986	53.00		53.00		(May)

Notes:
* - No records.
** - Extreme recorded for the period of record.
ICE - Ice conditions, 12 years of records.
E - Estimated
A - Manual Gauge

TABLE 3.35

RESULTS OF FREQUENCY ANALYSIS FOR THE BOURGET STREAMFLOW GAUGE

	Three-Parameter Lognormal Distribution Condition I	Condition II	Condition III
No. of Sample Years	31	19	17
Coefficients of Skewness* (C.S.)	-0.063	-0.086	-0.033
Coefficients of Kurtosis** (C.K.)	3.133	3.429	3.733
100 Year Flood Flow (cms)	329 (369)***	376 (422)***	243 (273)***

Notes:

Condition I - All Data Records Used.
Condition II - All Ice Conditions Excluded.
Condition III - All Ice Conditions Excluded With 1979 & 1981 Records Removed.

* - should be approximately 0.0.
** - should be approximately 3.0.
*** - instantaneous flood flow (Peaking Factor = 1.122).

TABLE 3.36

RESULTS OF SUMMER EVENTS FREQUENCY ANALYSIS FOR THE BOURGET STREAMFLOW GAUGE

Return Period (yr)	Flood Flow (cms) GEV	3PLN	LP3	Wakeby	HYMO
2	22.8	20.3	25.6	23.2	14.9
5	43.2	45.9	47.1	47.5	33.8
10	60.3	71.0	59.2	62.8	49.2
20	80.3	102.0	68.6	75.9	N/A
25*	88.0	112.0	72.0	80.0	71.3
50	112.0	154.0	78.0	90.4	89.4
100	142.0	203.0	83.2	99.5	107.6

Notes:

* * - Obtained from probability plot.
* GEV - Generalized Extreme Value distribution
* 3PLN - Three-Parameter Lognormal distribution
* LP3 - Log Pearson Type III distribution
* HYMO - HYMO Model simulations

TABLE 3.37

COMPARISON OF PEAK FLOWS

BEAR BROOK WATERSHED

Return Period (yr)	Maximum Instantaneous Peak Flows Method 1 (cms)	Method 2 (cms)	Method 3 (cms)	Method 4 (cms)	Method 5 (cms)
2	188.9 (174.3)	113.9	93.0	87.0	138.6
5	236.7 (223.1)	129.6	119.9	136.5	173.2
10	268.5 (254.9)	141.4	137.0	168.0	207.9
25	308.3 (297.0)	152.8	156.5	210.0	249.4
50	338.4 (327.3)	163.1	170.0	241.5	284.1
100	368.9 (357.1)	173.0	182.4	270.9	311.8

Notes:

Method 1 - HYMO Method (7-Day Snowmelt), at the outlet of Reach 3.
Flows in brackets are at the outlet of the Bear Brook watershed.

Method 2 - Delisle River Regional Flood Frequency Method.

Method 3 - Regional Regression Method.

Method 4 - Watershed Classification Method.

Method 5 - Index Flood Method.

TABLE 3.38

COMPARISON OF PEAK FLOWS

SNOWMELT VERSUS FREQUENCY ANALYSES

Event (yr)	Maximum Instantaneous Peak Flows (cms) 7-Day Snowmelt Analysis	Frequency Analysis
100	369	369
50	338	323
25	308	N/A
20	N/A	266
10	269	223
5	237	180
2	189	118

Note: The maximum instantaneous peak flows for the Frequency Analysis were obtained by applying a peaking factor of 1.122.

TABLE 3.39

COMPARISON OF ROUTED PEAK FLOWS

100 YEAR SNOWMELT VERSUS RAINFALL

BASINS 5 TO 9

Outflow From PHU	Routed Peak Flows (cms) 100 Year Snowmelt	100 Year Rainfall
Basin 5		
B5.12	1.53	1.65
B5.11	2.26	2.38
B5.11 & B5.12	3.79	3.96
R5.10	3.28	7.25
R5.9	3.14	8.30
B5.13	1.30	1.42
R5.8	3.94	10.51
B5.7	3.38	3.34
R5.6	2.10	5.27
B5.5	10.43	8.43
R5.4	16.85	14.05
R5.3 & R5.2	21.07	26.53
R5.1	24.55	32.20
Basin 6		
B6.4	0.84	0.88
B6.3	2.96	2.29
R6.2	6.94	8.78
R6.1	7.53	9.12
Basin 7		
B7.9	6.02	5.31
B7.8	3.42	3.65
B7.7	3.78	4.02
R7.6	15.09	14.41
B7.5	1.49	1.31
R7.4	16.91	16.17
B7.10	0.58	0.55
R7.3	23.45	21.83
R7.2	25.15	23.59
R7.1	27.24	26.19

TABLE 3.39 (Cont'd)

COMPARISON OF ROUTED PEAK FLOWS

100 YEAR SNOWMELT VERSUS RAINFALL

BASINS 5 TO 9

Outflow From PHU	Routed Peak Flows (cms) 100 Year Snowmelt	100 Year Rainfall
Basin 8		
B8.4	17.68	14.07
R8.3	28.74	17.84
R8.2	34.49	20.02
R8.1	36.90	20.95
Basin 9		
B9.7	7.75	7.66
B9.6	1.99	2.11
R9.5	10.96	10.99
B9.8	9.31	8.76
R9.4	27.72	25.49
R9.3	31.71	28.52
R9.2	29.22	23.90
B9.12	11.03	8.79
R9.11	15.43	13.42
B9.14	1.59	1.23
R9.13	3.43	2.97
R9.10 & R9.1	19.06	16.08
B9.15	4.18	4.49
R9.9	3.82	8.24
R9.1, R9.2, R9.9 & R1	50.89	43.49

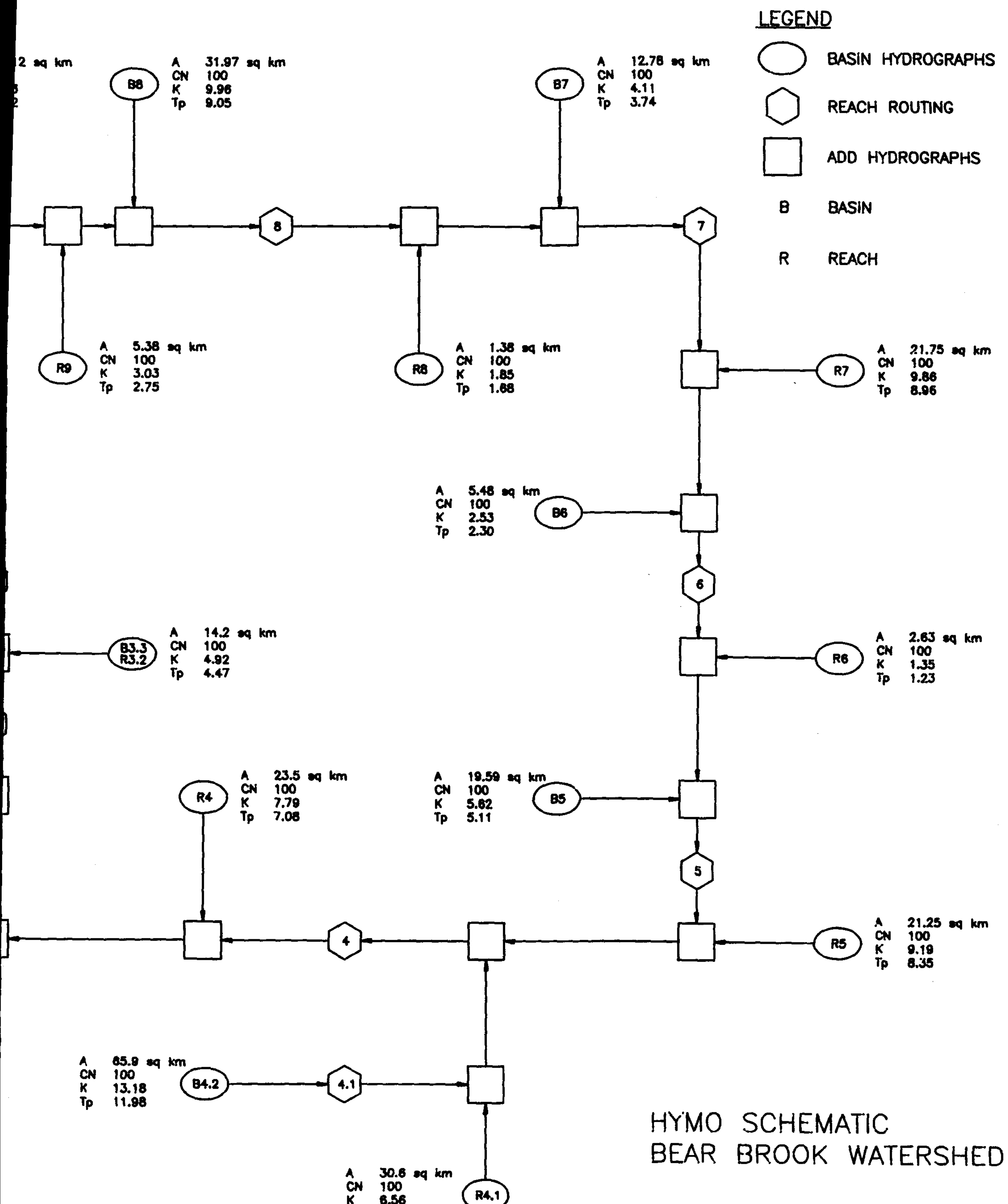
TABLE 3.40

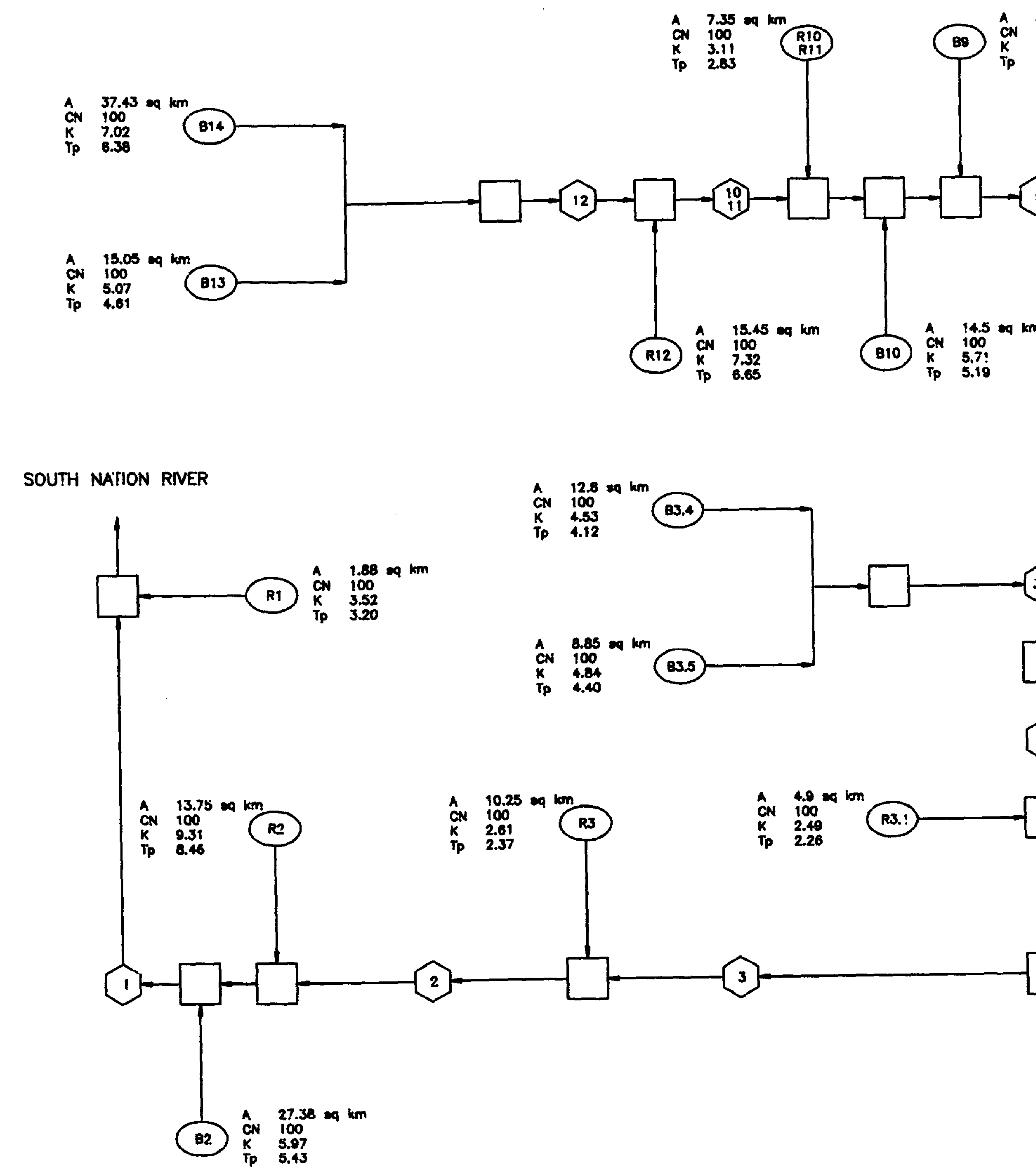
COMPARISON OF PEAK FLOWS

SNOWMELT VERSUS RAINFALL ANALYSES

BASINS 5 TO 9

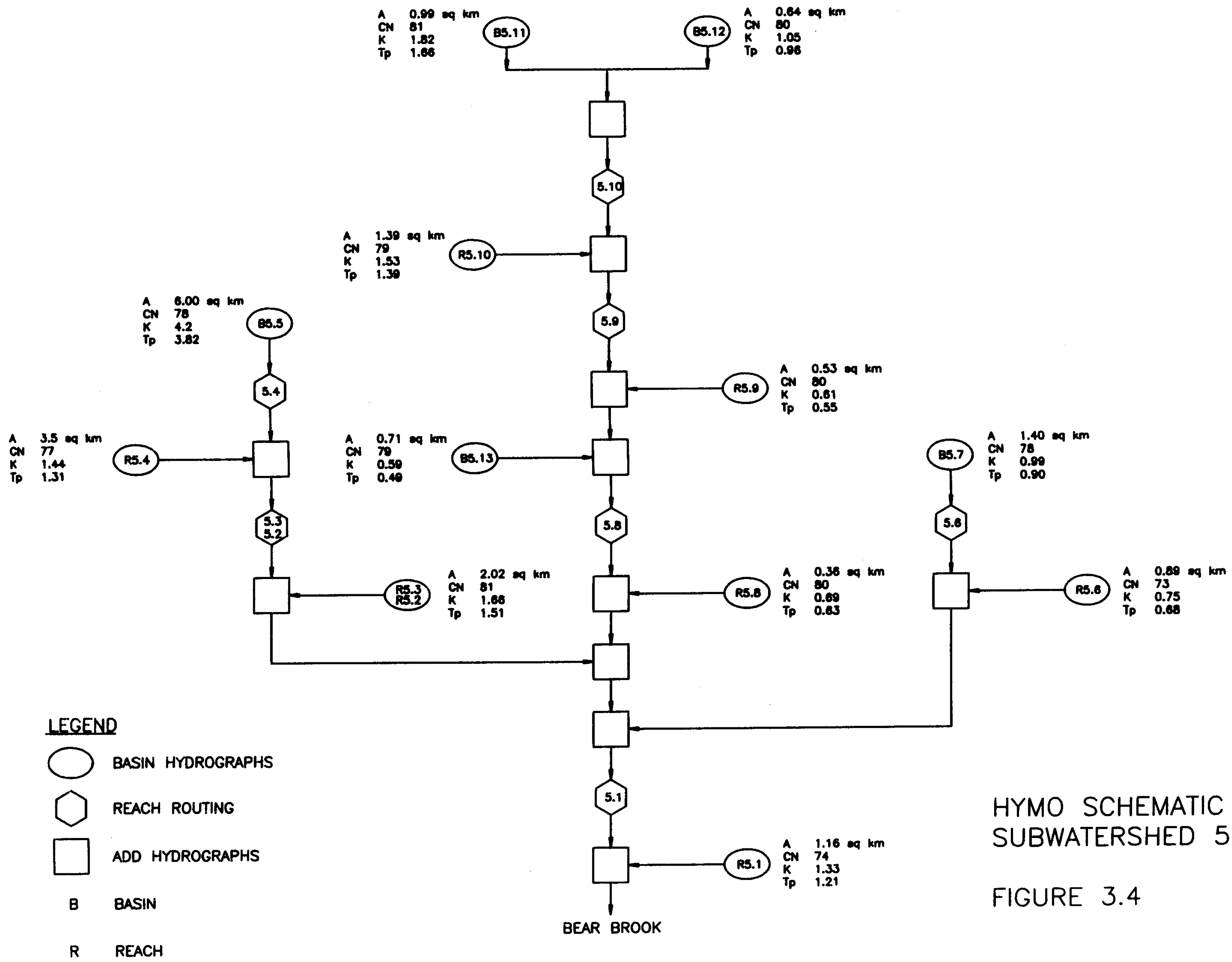
Event (yr)	PHU	Peak Flood Flows (cms) Snowmelt	Rainfall	Backwater
100	Basin 5	24.55	32.20	32.20
	Basin 6	7.53	9.12	9.12
	Basin 7	27.24	26.19	27.24
	Basin 8	36.90	20.95	36.90
	Basin 9	50.89	43.49	50.89
50	Basin 5	22.68	27.10	27.10
	Basin 6	6.97	7.67	7.67
	Basin 7	25.26	22.46	25.26
	Basin 8	33.95	17.67	33.95
	Basin 9	47.20	36.59	47.20
25	Basin 5	20.84	22.00	22.00
	Basin 6	6.43	6.29	6.43
	Basin 7	23.25	18.86	23.25
	Basin 8	30.98	14.58	30.98
	Basin 9	43.44	30.47	43.44
10	Basin 5	18.35	15.77	18.35
	Basin 6	5.66	4.47	5.66
	Basin 7	20.53	13.88	20.53
	Basin 8	27.25	10.56	27.25
	Basin 9	38.31	22.91	38.31
5	Basin 5	16.34	11.27	16.34
	Basin 6	5.07	3.20	5.07
	Basin 7	18.46	9.97	18.46
	Basin 8	24.27	7.48	24.27
	Basin 9	34.18	17.81	34.18
2	Basin 5	13.39	5.41	13.39
	Basin 6	0.85	1.47	1.47
	Basin 7	15.23	5.04	15.23
	Basin 8	19.85	3.71	19.85
	Basin 9	27.27	9.17	27.27





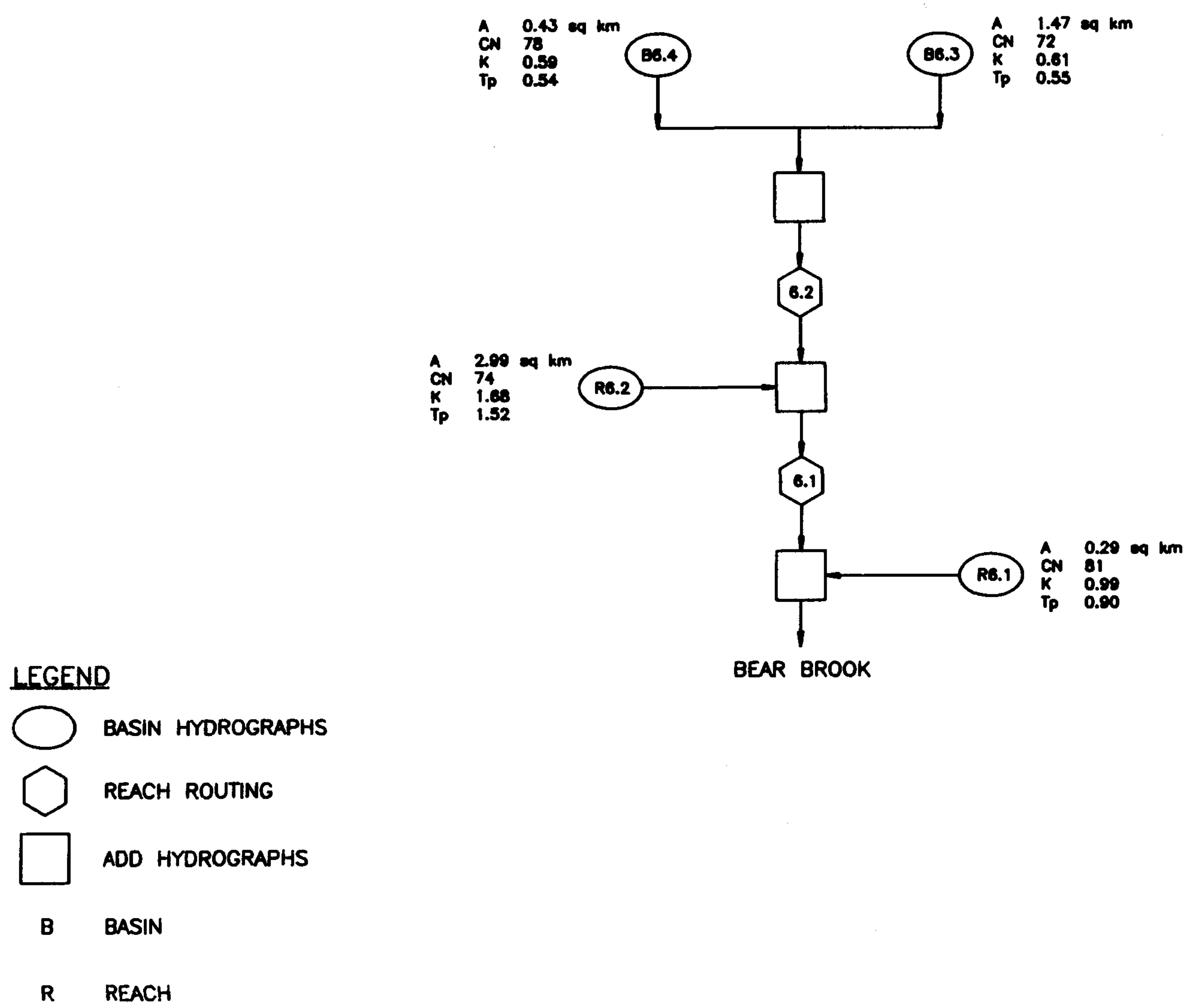
HYMO SCHEMATIC
BEAR BROOK WATERSHED

FIGURE 3.2



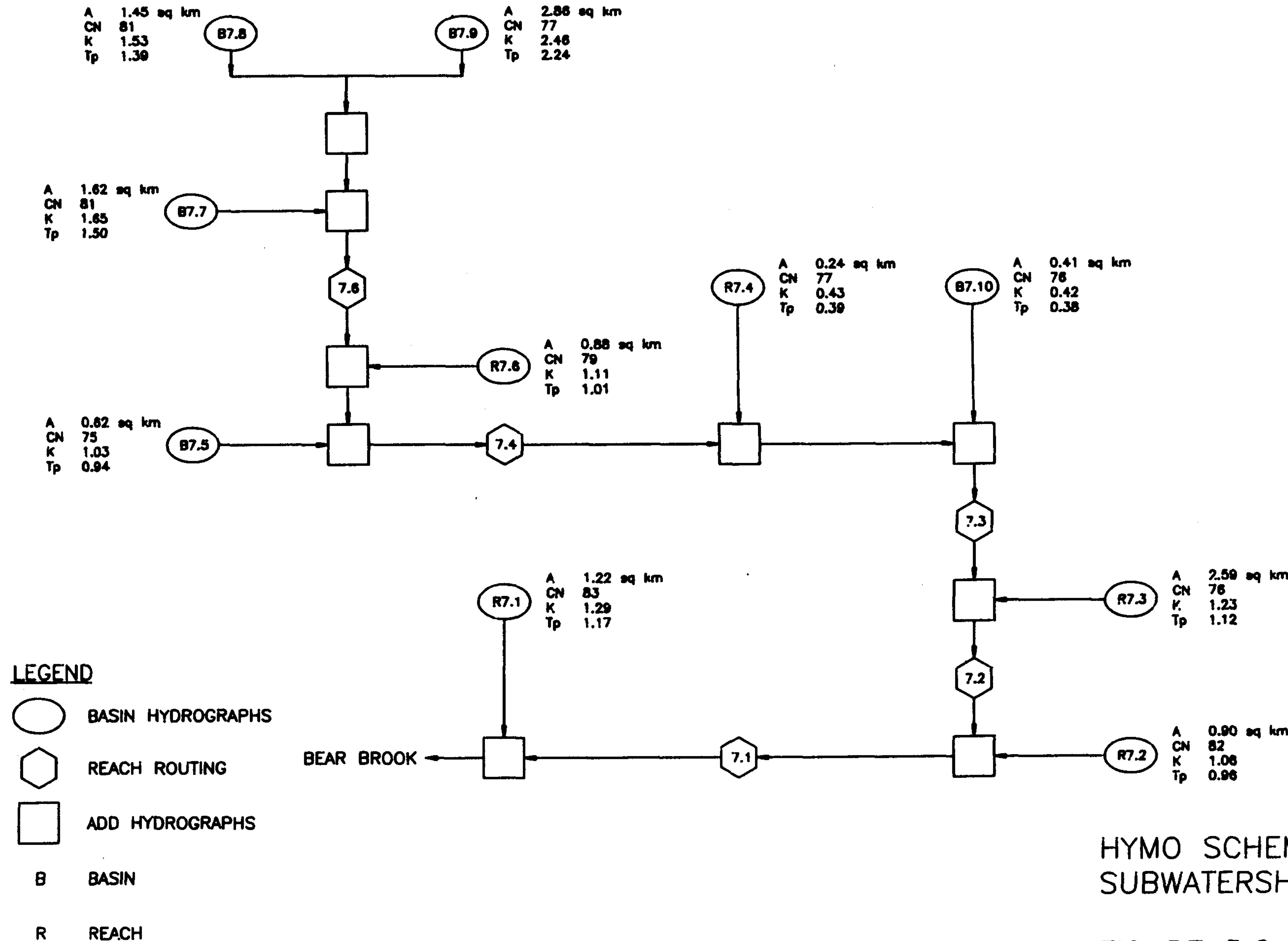
HYMO SCHEMATIC
SUBWATERSHED 5

FIGURE 3.4



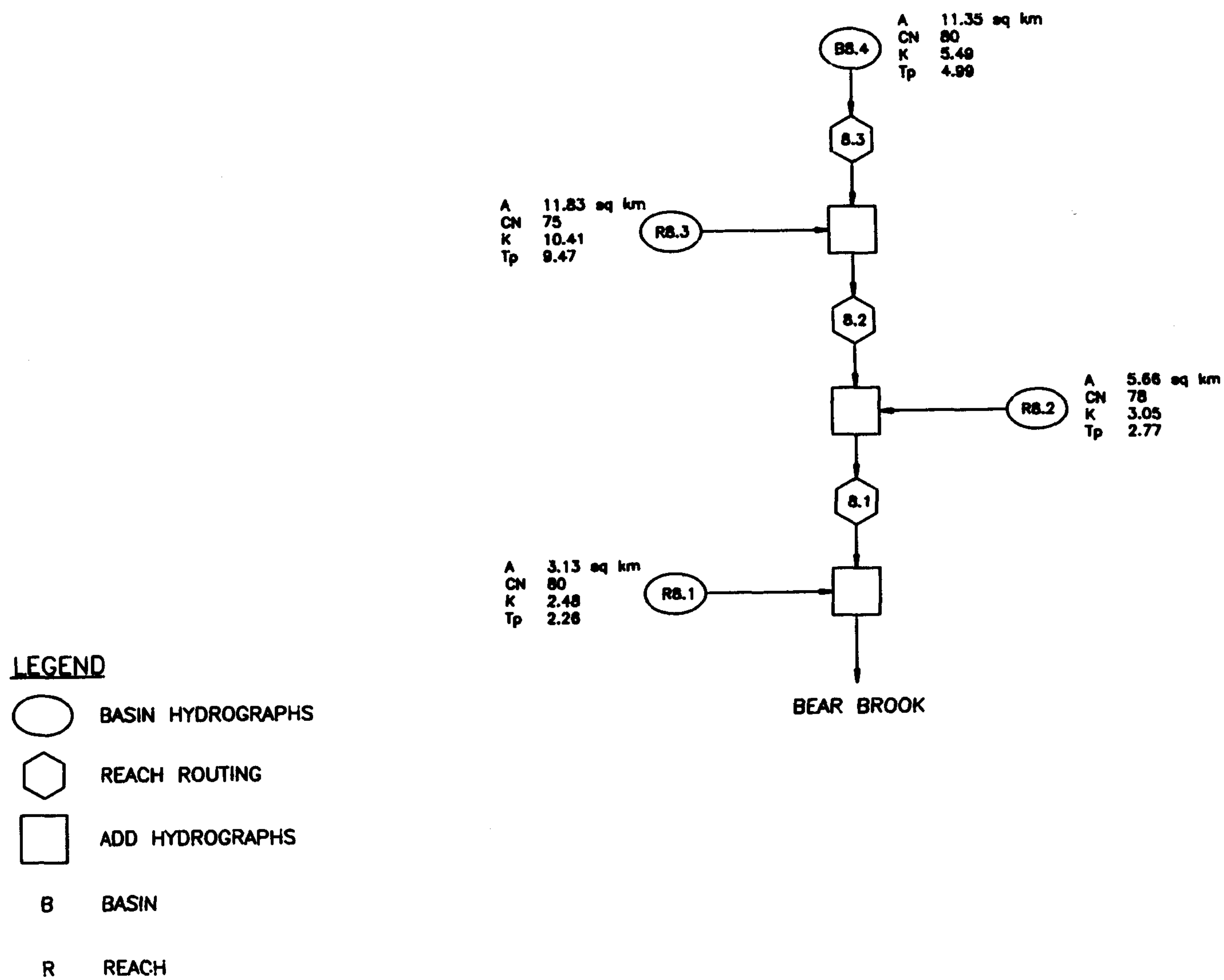
HYMO SCHEMATIC
SUBWATERSHED 6

FIGURE 3.5



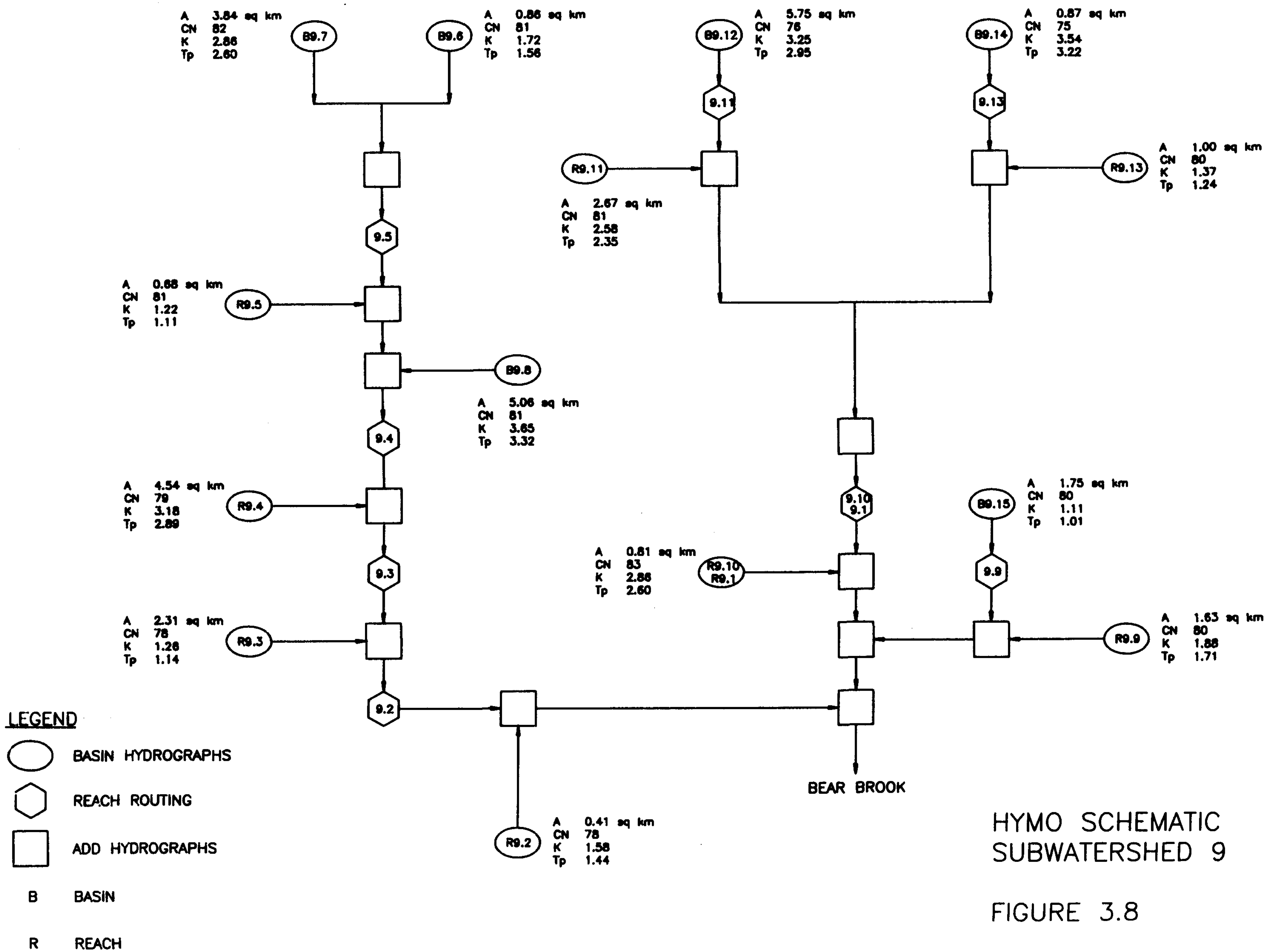
HYMO SCHEMATIC
SUBWATERSHED 7

FIGURE 3.6



HYMO SCHEMATIC
SUBWATERSHED 8

FIGURE 3.7



HYMO SCHEMATIC
SUBWATERSHED 9

FIGURE 3.8

4.0 HYDRAULICS

4.1 HYDRAULIC MODEL

The floodline, or water surface elevation, for the Regulatory and the lower return frequency flood events is a function of the design flows and the ability of the channel, flood plain and river crossings to carry or pass these flows. In order to establish the water surface elevations at various locations in the study watershed, a detailed hydraulic analysis must be carried out. The channel and flood plain properties, as well as the characteristics of the various structures along the channel, must be considered in this analysis.

The hydraulic program used by Ecos Garatech to compute the water surface profiles was developed at the Hydrologic Engineering Center (HEC) by the U.S. Army Corps of Engineers and is commonly known as HEC-2.

The program computes and plots (by printer) the water surface profiles of river channels of any cross-section for either subcritical or supercritical flow conditions. It is capable of analyzing the effects of various hydraulic structures such as bridges, culverts, weirs, embankments and dams. Roughness coefficients can be specified by a number of methods to account for the change in roughness with the depth of flow or the actual location of the flow within the flood plain. Input to the program may be in either Imperial or Metric units.

Hydraulic models of the study reaches were constructed by inputting specific cross-sections, taken along the length of the Channels being mapped, into the model. The digital cross-sections, orientated from left and right looking upstream, were obtained by photogrammetric photo interpretation, supplemented by field reconnaissance and construction drawings. The characteristics of the main channel and the flood plain, such as the hydraulic roughness, as obtained from field reconnaissance, were also included in the models. All river crossings and hydraulically significant structures and sections were also entered into the models to produce a physical representation of the study area.

The hydraulic model for the study (Bear Brook) area, so established, may also be used to determine the capacity of various structures and channel reaches and to determine the effects of channel improvements, dykes and flood fringe development on the water surface profiles.

4.2 STARTING WATER SURFACE ELEVATIONS

Along the main channel, the first cross-section (Chainage 0.0) was located in Lot 21, Concession 10 in Clarence Township, about 1500 metres downstream of the Cumberland township boundary.

The study limits are within the Cumberland Township boundary. Therefore, the corresponding flood lines were plotted within the Township boundary.

In order to be consistent with the 100 year water surface profile computed by Crysler & Lathem Ltd. in 1978, the slope-area method in the HEC-2 program was used to establish the starting water level elevation. The slope was found to be 0.000075. It was therefore assumed that the same slope was applicable to the other events.

Table 4.1 details the starting water surface elevations for the main channel based on the slope-area method, and associated tributaries.

4.3 BACKWATER FLOWS

In order to provide descending and appropriate values for the backwater simulations for the tributaries of Bear Brook, peak flows in Basins 5 and 6 were based on rainfall events, whilst peak flows in Basins 7, 8 and 9 were based on snowmelt events.

4.4 WATER SURFACE PROFILES

Detailed hydraulic models were constructed for the Bear Brook Study Area.

Upon completion of the hydrologic component of the study, water surface profiles associated with the Regulatory (100 year) flood and the 50, 25, 10, 5 and 2 year flood events were computed using the developed hydraulic models in conjunction with the HEC-2 computer program.

In June 1990, surveyed cross-sections in the vicinity of Elian Reginbald Drain (Chainage 6359 to Chainage 8229) were made available, as a result of channelization works proposed in the area. The field survey and the design of the channel improvements were undertaken by McNeely Engineering Ltd. The Water Planning and Management Branch of Environment Canada, therefore, re-generated the water surface elevations, which were incorporated into the Report and Flood Risk Map. The report, entitled "Bear Brook Hydraulics Study - Short Report" is appended in Appendix C.

The final design of the channel improvements was completed and construction works began in the summer months of 1991. The Conservation Authority retained EGA Consultants to undertake the hydraulic assessment, in order to update the hydraulic models and Flood Risk Map as a result of the improvements. The results of the hydraulic analysis is provided in Appendix D, in an addendum report entitled "Addendum Hydraulics Report, Bear Brook, Township of Cumberland", dated September 1991.

4.5 SENSITIVITY ANALYSIS

A sensitivity analysis of the roughness coefficient, Manning's 'n' value, was undertaken in order to observe the potential change in the water surface profile of the Regulatory flood.

The developed hydraulic models were modified to reflect a 10% increase and a 10% decrease in the 'n' values in the channel, the left and right overbanks, and at all the bridge crossings. The range of 'n' along the watercourses are provided in Table 4.2. Water surface profiles were then re-generated.

A review of the results indicated that with a 10% increase in the 'n' values, the increase in the water surface elevations varied from 0.01 to 0.16 m for the main channel, 0.01 to 0.07 m for Elian Reginbald Drain, 0.01 to 0.05 m for Bearbrook Drain, 0.01 to 0.10 m for McWilliams Drain, 0.01 to 0.08 m for Shaws Creek, and 0.01 to 0.06 m for McKinnons Creek and its tributaries (Basin 9). On the other hand, with a 10% decrease in the 'n' values, the decrease in the water surface elevations varied from 0.01 to 0.15 m for the main channel, 0.01 to 0.08 m for Elian Reginbald Drain, 0.01 to 0.08 m for Bearbrook Drain, 0.01 to 0.08 m for McWilliams Drain, 0.01 to 0.07 m for Shaws Creek, and 0.01 to 0.07 m for McKinnons Creek and its tributaries (Basin 9).

A further review of the resultant flood plain along Bear Brook (main channel) indicated that a change of more than 0.1 m occurred from Chainage 1520 to 6133, a meandering river section with little overbanks and hence, produced minimal horizontal displacement.

4.6 WATER LEVELS MONITORING PROGRAM

The Conservation Authority carried out a Water Levels Monitoring Program during the spring freshet. Water levels at several structures along Bear Brook and its tributaries were recorded each year. Seven structures are within the Study Limits.

In 1988, a maximum daily peak flow of 58.5 cms was recorded at the streamflow station near Bourget. In 1987, a maximum daily peak flow of 121 cms was recorded on March 27, with an instantaneous peak flow of 135 cms occurring on the same date.

The peak flow of 135 cms was less than the 2 year 7-day snowmelt event of 189 cms (see Table 3.38). The maximum recorded water levels at the structures in 1987 were less than the computed water surface elevations of the 2 year snowmelt event. Table 4.3 provides the monitored and computed water levels.

4.7 STRUCTURES

Floodwater unduly confined by structures can cause excessive water pondage. This may result in flooding of upstream properties, over-topping of roadways, excessive scour and erosion and, in severe cases, the loss of a structure. On the other hand, over-design of new structures for the sake of safety can add materially to the initial cost of the structure, and possibly increase downstream flood flows as a result of improved hydraulic characteristics. Increased flood flows would generally increase water levels and velocities, which may result in potential flood damages or increase damages.

Reconnaissance and field surveys within the study limits ascertained detailed information required to analyze the performance characteristics of the hydraulic structures. This information was used as computer input data, not only to determine the extent of flooding for the various flood events but also to analyze the performance of the individual bridges and structures.

The resultant stage-discharge rating curves for the individual structures, are provided in the support document entitled "Bridge Data". The structure performance data are provided in Table 4.4.

The term "structure velocity" given in Table 4.4 is defined as the average velocity of the flow discharging through the structure for an effective flow area.

4.8 CROSS-SECTIONS

As outlined in Section 4.1, digital cross-sections were obtained by photogrammetric photo interpretation. Upon completing the hydraulic analyses along Bear Brook and its tributaries, and plotting the 100 year flood plain, it was revealed that the 100 year flood plain went beyond (outside) the cross-sections along five (5) reaches. The five reaches and a description of the hydraulic characteristics are given below.

Reach 1

This reach is located along Bear Brook, downstream of Regional Road No. 33, and between Cross-sections 14707 and 15050 (Lots 15 and 16, Concession 6 - Sheet 70-24). The computed water surface elevations varied from 65.32 m (Cross-section 14707) to 65.33 m (Cross-section 15050), with corresponding energy grade lines of 65.32 m and 65.33 m, respectively.

The total head loss along the 343 m long reach is a minimal 0.01 m. As such, should the cross-sections be revised to extend completely across the 100 year flood plain, the results of the re-evaluation would not alter the computed 100 year flood plain, and would not significantly affect adjacent and upstream water levels.

Reach 2

This reach is located along Shaws Creek, between Cross-sections 325 and 2079 (Lots 17, 18, 19 and 20, Concession 8 - Sheet 66-24), and is a portion of the Bear Brook flood plain. The computed water surface elevations varied from 65.68 m (Cross-section 325) to 65.87 m (Cross-section 2079), with corresponding energy grade lines of 65.68 m and 65.99 m, respectively.

The difference in the computed water surface elevations is 0.19 m, and the total head loss along the 1754 m long reach is 0.31 m. The use of cross-sections extending completely across the flood plain would have produced approximately the same differences in the water surface elevations and total head loss.

Therefore, the use of cross-sections not extending completely across the 100 year flood plain would not alter the computed 100 year flood plain, nor significantly affect adjacent and upstream water levels.

Reach 3

This reach is located along Bear Brook, from the 8th Concession Road to Regional Road No. 31, and between Cross-sections 17864 and 21048 (Lots 16 and 17, Concession 8, and Lots 17 and 18, Concession 9 - Sheet 66-24). The computed water surface elevations varied from 65.64 m (Cross-section 17864) to 65.70 m (Cross-section 21048), with corresponding energy grade lines of 65.64 m and 65.70 m, respectively.

The difference in the computed water surface elevations is a minimal 0.06 m, and the total head loss along the 3184 m long reach is a minimal 0.06 m. Therefore, the use of cross-sections not extending completely across the 100 year flood plain would not alter the computed 100 year flood plain, nor significantly affect adjacent and upstream water levels.

Reach 4

This reach is located along Bear Brook, downstream (east) of the 8th Concession Road, and between Cross-sections 15542 and 16483 (Lots 15 and 16, Concession 7 - Sheet 66-24). The computed water surface elevations varied from 65.43 m (Cross-section 15542) to 65.45 m (Cross-section 16483), with corresponding energy grade lines of 65.43 m and 65.45 m, respectively.

The difference in the computed water surface elevations is a minimal 0.02 m, and the total head loss along the 941 m long reach is a minimal 0.02 m. Therefore, the use of cross-sections not extending completely across the 100 year flood plain would not alter the computed 100 year flood plain, nor significantly affect adjacent and upstream water levels.

Reach 5

This reach is located along the East Branch of the Savage Drain, upstream (west) of the Regional Road No. 31, and between Cross-sections 263 and 3139 (Lots 13, 14, 15, 16 and 17, Concession 10 - Sheets 66-24 and 62-24). The computed water surface elevations varied from 65.77 m (Cross-section 263) to 65.79 m (Cross-section 3139), with corresponding energy grade lines of 65.77 m and 65.79 m, respectively.

The difference in the computed water surface elevations is a minimal 0.02 m, and the total head loss along the 2876 m long reach is a minimal 0.02 m. As such, the use of cross-sections not extending completely across the 100 year flood plain would not alter the computed 100 year flood plain, nor significantly affect adjacent and upstream water levels.

4.9 RESULTS

The extent of flooding within the study area as a result of the Regulatory (100 year) flood was plotted on the South Nation River Conservation Authority's Flood Risk Mapping.

The results of the hydraulic investigations are:

(1) In order to provide descending and appropriate values for the backwater simulations for the tributaries of Bear Brook, peak flows in Basins 5 and 6 were based on rainfall events, whilst peak flows in Basins 7, 8 and 9 were based on snowmelt events. The selection of snowmelt or rainfall was based on which event had the highest flow, over most of the sub-basin, during the higher return events.

(2) The Manning's 'n' sensitivity analysis demonstrated that a 10% deviation in the values would not significantly alter the simulated Regulatory flood plain.

A change in the 100 year flood level of more than 0.1 m occurred along Bear Brook, from Chainage 1520 to 6133. This meandering river section has little overbank flow, and hence, produced minimal horizontal displacement.

(3) About 52% of the bridge structures crossing Bear Brook and its associated tributaries can discharge the various flood events, without weir flow occurring over the roadway embankment. The breakdown for the Regulatory flood event is 27 under low flow conditions, 8 under pressure flow conditions, 32 under pressure and weir flow conditions, and 1 under low and weir flow conditions.

(4) In reviewing the 100 year flood plain:

(a) It was estimated that 29 buildings are within the flood plain. The location of the buildings are provided in Table 4.5.

(b) There exists a constriction from Chainage 6912 to 7270. This river reach, about 358 metres long, has an oxbow and produced a significant difference of 2.18 m in the computed water levels, from 60.72 m to 62.90 m.

(c) From Chainage 18396 to 21048, there exists a large flood plain. The width of the flood plain is more than 1500 metres. The simulated water levels varied from 65.66 m to 65.70 m, a difference of 0.04 m in a 2652 m long reach.

TABLE 4.1

STARTING WATER SURFACE ELEVATIONS BEAR BROOK AND TRIBUTARIES

Location and Cross-section Number	Starting Water Surface Elevations (m)					
	100 Year Flood	50 Year Flood	25 Year Flood	10 Year Flood	5 Year Flood	2 Year Flood
0.00 Main Channel	57.76	57.51	57.25	56.88	56.58	56.11
1520.00 Main Channel	57.96	57.72	57.46	57.11	56.81	56.33
27.00 Elian Reginbald Drain	62.12	61.97	61.82	61.60	61.41	61.10
13.00 Bearbrook Drain	64.22	64.11	63.99	63.80	63.63	63.35
227.00 McWilliams Drain	65.64	65.55	65.44	65.28	65.10	64.78
325.00 Shaws Creek	65.68	65.59	65.48	65.32	65.15	64.83
190.00 East Branch Savage Drain	65.70	65.61	65.50	65.34	65.17	64.85
32.00 McFadden Drain	65.78	65.76	65.70	65.65	65.60	65.32
160.00 McKinnons Creek	65.70	65.61	65.50	65.34	65.17	64.85
101.00 Bickerton Drain	65.70	65.61	65.50	65.34	65.17	64.85

Note: Cross-section No. 1520 along the Main Channel is located at the Cumberland Clarence boundary, the start of the study.

TABLE 4.2

RANGE OF 'n' VALUES

Watercourse	Range of 'n' Values Left Overbank	Channel	Right Overbank	Structure
Bear Brook	0.04 to 0.075	0.04 to 0.045	0.04 to 0.07	0.023 to 0.027
Elian Reginbald Drain	0.05 to 0.08	0.03 to 0.055	0.05 to 0.08	0.015 to 0.024
Bearbrook Drain	0.035 to 0.06	0.04	0.035 to 0.06	0.015 to 0.024
McWilliams Drain	0.035 to 0.06	0.03 to 0.045	0.035 to 0.06	0.015 to 0.028
Shaws Creek	0.05 to 0.07	0.035 to 0.045	0.05 to 0.07	0.021 to 0.024
E. Br. Savage Drain	0.04	0.035	0.04	0.024
McFadden Drain	0.04	0.045	0.04	---
McKinnons Creek	0.035 to 0.06	0.035 to 0.050	0.03 to 0.06	0.015 to 0.024
Bickerton Drain	0.04	0.04 to 0.045	0.04	0.015

TABLE 4.3

MONITORED AND COMPUTED WATER LEVELS

SNRCA Site No.	Site Location	Water Levels (m) Monitored (1987)	Computed (2 Year SM)
2	Bear Brook Regional Road 31 (Sect. 21073)	64.52	64.83
3	Shaws Creek Regional Road 26 (Sect. 3342)	66.86	66.95
4	McKinnons Creek Lot 10, Conc. 9 (Sect. 4640)	74.91	75.43
5	McWilliams Drain Regional Road 28 (Sect. 4890)	78.59	78.98
6	Bear Brook Regional Road 33 (Sect. 15085)	64.02	64.40
7	Bear Brook Regional Road 35 (Sect. 9301)	62.45	63.26
8	Bear Brook Conc. 3 Road (Sect. 6171)	58.03	58.18

Notes: 1) SM denotes snowmelt.
2) Computed water levels taken at downstream face of structure.

TABLE 4.4

STRUCTURE PERFORMANCE DATA

Location	Flood Event (yr)	U/S W. L. (m)	Discharge (cms)	Structure Velocity (m/s)	Class of Flow	Total Head Loss (m)
BEAR BROOK (Main Channel)						
578	100	57.87	215	1.13	PF-WF	0.04
	50	57.62	197	1.15	PF-WF	0.05
	25	57.36	179	1.16	PF-WF	0.05
	10	57.00	155	1.15	PF-WF	0.05
	5	56.70	136	1.11	PF-WF	0.06
	2	56.21	109	1.11	PF	0.05
3151	100	58.41	215	1.90	PF-WF	0.21
	50	58.14	197	1.78	PF	0.18
	25	57.85	179	1.62	PF	0.15
	10	57.44	155	1.37	LF	0.09
	5	57.13	136	1.29	LF	0.07
	2	56.66	109	1.15	LF	0.07
6171	100	59.44	215	3.38	LF	0.52
	50	59.22	197	3.41	LF	0.55
	25	59.05	179	3.25	LF	0.52
	10	58.82	155	2.99	LF	0.44
	5	58.65	136	2.74	LF	0.37
	2	58.41	109	2.32	LF	0.25
8250	100	64.16	179	2.41	PF-WF	0.30
	50	64.05	165	2.26	PF-WF	0.26
	25	63.92	150	2.06	PF	0.21
	10	63.71	130	1.78	PF	0.15
	5	63.51	114	1.56	PF	0.10
	2	63.19	92.1	1.45	LF	0.04
9301	100	64.37	177	1.62	LF	0.11
	50	64.28	162	1.51	LF	0.10
	25	64.16	148	1.43	LF	0.09
	10	63.95	128	1.30	LF	0.08
	5	63.70	113	1.24	LF	0.07
	2	63.35	91.2	1.12	LF	0.06
13210	100	64.80	177	0.71	PF-WF	0.04
	50	64.70	162	0.71	PF-WF	0.05
	25	64.58	148	0.71	PF-WF	0.05
	10	64.40	128	0.71	PF-WF	0.05
	5	64.23	113	0.71	PF-WF	0.06
	2	63.98	91.2	0.71	PF-WF	0.08

TABLE 4.4 (Cont'd)

STRUCTURE PERFORMANCE DATA

Location	Flood Event (yr)	U/S W. L. (m)	Discharge (cms)	Structure Velocity (m/s)	Class of Flow	Total Head Loss (m)
BEAR BROOK						
13433	100	64.78	177	0.83	PF-WF	0.25
	50	64.68	162	0.83	PF-WF	0.22
	25	64.58	148	0.83	PF-WF	0.20
	10	64.42	128	0.77	PF-WF	0.18
	5	64.26	113	0.77	PF-WF	0.17
	2	64.04	91.2	0.83	PF-WF	0.15
13603	100	65.11	177	2.06	PF-WF	0.12
	50	65.00	162	2.09	PF-WF	0.15
	25	64.83	148	1.97	PF	0.12
	10	64.57	128	1.70	PF	0.08
	5	64.34	113	1.63	LF	0.04
	2	64.09	91.2	1.42	LF	0.03
13795	100	65.31	177	0.86	PF-WF	0.07
	50	65.21	162	0.86	PF-WF	0.07
	25	65.07	148	0.86	PF-WF	0.08
	10	64.86	128	0.88	PF-WF	0.10
	5	64.67	113	1.05	PF-WF	0.11
	2	64.43	91.2	1.44	PF-WF	0.13
15085	100	65.42	177	0.68	PF-WF	0.09
	50	65.34	162	0.80	LF-WF	0.11
	25	65.23	148	1.02	LF-WF	0.14
	10	65.04	128	1.54	LF-WF	0.16
	5	64.84	113	1.81	LF-WF	0.14
	2	64.57	91.2	1.61	LF	0.11
17418	100	65.64	200	1.57	PF-WF	0.17
	50	65.55	184	1.63	PF-WF	0.16
	25	65.44	168	1.72	PF-WF	0.16
	10	65.28	147	1.85	PF-WF	0.19
	5	65.10	131	1.96	PF	0.20
	2	64.78	107	1.60	PF	0.15
21073	100	65.78	105	1.07	LF-WF	0.08
	50	65.69	96.2	1.16	LF-WF	0.08
	25	65.57	87.8	1.21	LF-WF	0.07
	10	65.41	76.9	1.16	LF	0.07
	5	65.24	68.7	1.11	LF	0.07
	2	64.92	55.9	1.03	LF	0.07

TABLE 4.4 (Cont'd)

STRUCTURE PERFORMANCE DATA

Location	Flood Event (yr)	U/S W. L. (m)	Discharge (cms)	Structure Velocity (m/s)	Class of Flow	Total Head Loss (m)
BEAR BROOK						
22910	100	66.27	85.8	1.52	PF-WF	0.12
	50	66.19	78.7	1.43	PF-WF	0.11
	25	66.08	71.7	1.31	PF	0.08
	10	65.96	62.8	1.17	LF	0.05
	5	65.87	56.0	1.07	LF	0.04
	2	65.71	45.8	0.92	LF	0.03
24687	100	67.29	88.5	2.28	PF-WF	0.32
	50	67.18	81.8	2.20	PF	0.29
	25	67.16	75.1	2.04	PF	0.35
	10	66.96	65.9	1.79	PF	0.27
	5	66.80	58.7	1.59	PF	0.22
	2	66.56	47.8	1.88	LF	0.16
ELIAN REGINBALD DRAIN (Basin 5)						
817	100	65.00*	32.2	2.14	PF-WF	1.16
	50	64.90*	27.1	1.43	PF-WF	1.19
	25	64.78*	22.0	1.43	PF-WF	1.22
	10	64.64	15.8	1.43	PF-WF	1.25
	5	64.53	11.3	1.43	PF-WF	1.28
	2	64.29	5.41	1.43	PF-WF	1.27
1428	100	69.80	32.2	1.99	LF	0.29
	50	69.64	27.1	1.74	LF	0.25
	25	69.48	22.0	1.48	LF	0.22
	10	69.29	15.8	1.14	LF	0.19
	5	69.14	11.3	0.87	LF	0.20
	2	68.87	5.41	0.48	LF	0.22
3190	100	74.45	17.0	0.98	PF-WF	0.25
	50	74.39	14.1	0.98	PF-WF	0.29
	25	74.35	11.6	0.98	PF-WF	0.36
	10	74.31	8.18	0.98	PF-WF	0.49
	5	74.26	5.69	0.98	PF-WF	0.59
	2	74.20	2.58	0.98	PF-WF	0.82
3440	100	75.56	17.0	3.69	LF	1.03
	50	75.30	14.1	3.52	LF	0.84
	25	75.06	11.6	3.27	LF	0.65
	10	74.69	8.18	2.30	LF	0.34
	5	74.46	5.69	1.62	LF	0.19
	2	74.25	2.58	0.76	LF	0.05

TABLE 4.4 (Cont'd)

STRUCTURE PERFORMANCE DATA

Location	Flood Event (yr)	U/S W. L. (m)	Discharge (cms)	Structure Velocity (m/s)	Class of Flow	Total Head Loss (m)
ELIAN REGINBALD DRAIN						
4110	100	76.29	17.0	4.01	LF	0.57
	50	75.96	14.1	3.77	LF	0.47
	25	75.79	11.6	3.02	LF	0.48
	10	75.36	8.18	2.28	LF	0.32
	5	75.04	5.69	1.70	LF	0.20
	2	74.62	2.58	0.95	LF	0.08
4765	100	76.94	17.0	2.10	LF	0.23
	50	76.73	14.1	1.90	LF	0.19
	25	76.54	11.6	1.68	LF	0.17
	10	76.36	8.18	1.35	LF	0.19
	5	76.21	5.69	1.07	LF	0.20
	2	75.98	2.58	0.66	LF	0.21
5555	100	79.92	14.0	3.56	LF	1.04
	50	79.64	11.8	3.37	LF	0.87
	25	79.36	9.66	3.16	LF	0.70
	10	78.98	6.91	3.10	LF	0.50
	5	78.68	4.87	2.75	LF	0.37
	2	78.40*	2.32	1.55	LF	0.40
6948	100	84.57	14.0	2.71	PF-WF	1.06
	50	84.23	11.8	2.92	PF-WF	0.82
	25	83.92	9.66	2.92	PF-WF	0.63
	10	83.82	6.91	2.34	PF	0.69
	5	83.25	4.87	3.14	LF	0.30
	2	82.81	2.32	1.44	LF	0.12
7630	100	84.85	8.44	1.58	LF	0.16
	50	84.63	7.11	1.51	LF	0.14
	25	84.44	5.80	1.37	LF	0.12
	10	84.25	4.22	1.11	LF	0.08
	5	84.02	3.03	0.95	LF	0.08
	2	83.77	1.50	0.63	LF	0.12
7790	100	85.29	8.44	0.51	PF-WF	0.42
	50	85.26	7.11	0.51	PF-WF	0.61
	25	85.23	5.80	0.51	PF-WF	0.76
	10	85.15E	4.22	0.51	PF-WF	1.07
	5	85.04	3.03	0.51	PF-WF	0.97
	2	84.55	1.50	1.72	PF	0.73

TABLE 4.4 (Cont'd)

STRUCTURE PERFORMANCE DATA

Location	Flood Event (yr)	U/S W. L. (m)	Discharge (cms)	Structure Velocity (m/s)	Class of Flow	Total Head Loss (m)
Tributary						
485	100	76.23	5.27	3.96	PF	2.27
	50	75.72	4.39	3.30	PF	1.84
	25	75.56	3.51	2.64	PF	1.77
	10	75.14	2.52	2.59	LF	1.49
	5	74.88	1.81	2.41	LF	1.35
	2	74.43	0.88	2.11	LF	1.14
1360	100	80.66	3.34	3.16	PF-WF	1.14
	50	80.55	2.82	2.97	PF	1.09
	25	80.35	2.31	2.43	PF	0.94
	10	80.06	1.70	2.19	LF	0.74
	5	79.91	1.25	1.91	LF	0.65
	2	79.51	0.65	1.56	LF	0.48
Tributary (Sarsfield Drain)						
560	100	74.57	10.5	3.34	LF	1.60
	50	74.32	8.98	3.18	LF	1.42
	25	74.07	7.48	3.00	LF	1.25
	10	73.70	5.49	2.72	LF	1.03
	5	73.41	3.94	2.44	LF	0.88
	2	72.98	2.07	1.93	LF	0.69
870	100	76.41	10.5	3.04	LF	1.65
	50	76.17	8.98	2.87	LF	1.55
	25	75.92	7.48	2.69	LF	1.41
	10	75.57	5.49	2.40	LF	1.20
	5	75.26	3.94	2.14	LF	0.99
	2	74.82	2.07	1.67	LF	0.69
1355	100	77.56	8.30	2.84	LF	1.06
	50	77.36	7.11	2.72	LF	1.05
	25	77.16	5.95	2.49	LF	1.01
	10	76.90	4.45	2.15	LF	0.94
	5	76.70	3.23	1.79	LF	0.88
	2	76.47	1.67	1.27	LF	0.86
2020	100	79.93	8.30	3.10	LF	0.69
	50	79.78	7.11	2.95	LF	0.60
	25	79.62	5.95	2.56	LF	0.48
	10	79.40	4.45	2.06	LF	0.35
	5	79.21	3.23	1.64	LF	0.26
	2	78.97	1.67	1.04	LF	0.14

TABLE 4.4 (Cont'd)

STRUCTURE PERFORMANCE DATA

Location	Flood Event (yr)	U/S W. L. (m)	Discharge (cms)	Structure Velocity (m/s)	Class of Flow	Total Head Loss (m)
2230	100	80.99	7.25	2.86	LF	0.80
	50	80.79	6.20	2.74	LF	0.69
	25	80.57	5.15	2.54	LF	0.55
	10	80.28	3.77	2.07	LF	0.38
	5	80.07	2.75	1.66	LF	0.26
	2	79.79	1.44	1.01	LF	0.11
3430	100	84.16	7.25	2.65	PF-WF	0.41
	50	84.02	6.20	1.77	PF-WF	0.35
	25	83.94	5.15	1.77	PF-WF	0.36
	10	83.84	3.77	2.65	PF-WF	0.39
	5	83.74	2.75	2.43	PF	0.41
	2	83.26	1.44	1.27	PF	0.15
3630	100	84.72	7.25	2.47	PF-WF	0.52
	50	84.66	6.20	2.47	PF-WF	0.58
	25	84.61	5.15	2.47	PF-WF	0.61
	10	84.54	3.77	2.47	PF-WF	0.64
	5	84.48	2.75	2.47	PF-WF	0.69
	2	83.61	1.44	1.78	PF	0.25
4135	100	84.76	1.65	1.27	PF-WF	0.03
	50	84.72	1.40	1.27	PF-WF	0.06
	25	84.69	1.16	1.27	PF-WF	0.08
	10	84.62	0.86	1.27	PF-WF	0.07
	5	84.53	0.64	0.81	PF	0.05
	2	83.65	0.34	0.43	PF	0.02
BEARBROOK DRAIN (Basin 6)						
153	100	64.24	9.12	1.17	PF	0.06
	50	64.15	7.67	0.99	PF	0.05
	25	64.03	6.29	0.81	PF	0.03
	10	63.82	4.47	0.58	PF	0.02
	5	63.54	3.20	0.42	LF	0.01
	2	63.14	1.48	0.24	LF	0.00
1103	100	64.69	8.78	2.26	LF	0.34
	50	64.52	7.33	1.95	LF	0.26
	25	64.36	5.95	1.66	LF	0.19
	10	64.14	4.16	1.26	LF	0.12
	5	63.98	2.92	0.95	LF	0.08
	2	63.76	1.37	0.50	LF	0.02

TABLE 4.4 (Cont'd)

STRUCTURE PERFORMANCE DATA

Location	Flood Event (yr)	U/S W. L. (m)	Discharge (cms)	Structure Velocity (m/s)	Class of Flow	Total Head Loss (m)
BEARBROOK DRAIN						
2652	100	67.39	3.17	1.38	LF	0.96
	50	67.32	2.61	1.25	LF	0.95
	25	67.26*	2.12	1.12	LF	0.96
	10	67.18E	1.53	0.95	LF	1.02
	5	67.15*	1.08	0.82	LF	1.04
	2	67.06*	0.51	0.95	LF	1.04
3325	100	69.45	0.88	1.49	LF	0.21
	50	69.41	0.73	1.27	LF	0.21
	25	69.37*	0.59	1.05	LF	0.20
	10	69.34*	0.42	0.76	LF	0.20
	5	69.32*	0.31	0.58	LF	0.20
	2	69.25*	0.16	0.33	LF	0.19
McWILLIAMS DRAIN (Basin 7)						
235	100	65.65	27.2	0.76	PF-WF	0.01
	50	65.56	25.3	0.76	PF-WF	0.01
	25	65.45	23.2	0.76	PF-WF	0.01
	10	65.29	20.5	0.76	PF-WF	0.02
	5	65.11	18.5	0.76	PF-WF	0.02
	2	64.79	15.2	0.76	PF-WF	0.03
830	100	66.82	27.2	2.23	PF	0.58
	50	66.70	25.3	2.07	PF	0.50
	25	66.56	23.2	2.48	LF	0.40
	10	66.43	20.5	2.19	LF	0.30
	5	66.33	18.5	2.01	LF	0.25
	2	66.14	15.2	1.77	LF	0.20
2730	100	72.63	25.1	0.74	PF-WF	0.26
	50	72.61	23.3	0.74	PF-WF	0.26
	25	72.59	21.5	0.74	PF-WF	0.26
	10	72.56E	18.9	0.74	PF-WF	0.23E
	5	72.54E	17.0	0.74	PF-WF	0.23E
	2	72.51	14.0	0.74	PF-WF	0.22
3260	100	76.44	23.4	4.67	LF	2.74
	50	76.25	21.7	4.52	LF	2.58
	25	76.05	20.0	4.35	LF	2.45
	10	75.81	17.7	4.18	LF	2.30
	5	75.60	15.8	4.02	LF	2.15
	2	75.27	13.0	3.77	LF	1.93

TABLE 4.4 (Cont'd)

STRUCTURE PERFORMANCE DATA

Location	Flood Event (yr)	U/S W. L. (m)	Discharge (cms)	Structure Velocity (m/s)	Class of Flow	Total Head Loss (m)
McWILLIAMS DRAIN						
4890	100	81.81	23.4	4.14	PF-WF	2.71
	50	81.68	21.7	4.09	PF	2.62
	25	81.27	20.0	3.77	PF	2.26
	10	80.75	17.7	3.33	PF	1.83
	5	80.36	15.8	2.98	PF	1.50
	2	79.75E	13.0	2.45	PF	1.00E
5720	100	82.63	16.6	2.73	LF	0.57
	50	82.50	15.3	2.66	LF	0.56
	25	82.35	14.2	2.76	LF	0.63
	10	82.16	12.5	2.72	LF	0.64
	5	82.02	11.2	2.60	LF	0.61
	2	81.77	9.17	2.35	LF	0.54
8215	100	87.77	3.79	2.22	PF	0.59
	50	87.65	3.51	2.05	PF	0.51
	25	87.53	3.23	1.89	PF	0.44
	10	87.45E	2.86	1.67	PF	0.41E
	5	87.42E	2.55	1.49	PF	0.44E
	2	87.36	2.10	1.23	PF	0.46
Tributary (H. Shaw Drain)						
877	100	88.29	6.03	3.99	PF	1.07
	50	88.17	5.58	3.70	PF	1.01
	25	87.96	5.13	3.40	PF	0.85
	10	87.69	4.53	3.00	PF	0.66
	5	87.49	4.05	2.68	PF	0.54
	2	87.21	3.34	2.21	PF	0.37
1365	100	88.57	6.03	2.27	PF-WF	0.26
	50	88.56	5.58	2.27	PF-WF	0.36
	25	88.54E	5.13	2.27	PF-WF	0.57
	10	88.53E	4.53	2.27	PF-WF	0.74
	5	88.52E	4.05	2.27	PF-WF	0.88
	2	88.49	3.34	2.27	PF-WF	1.05
SHAWS CREEK (Basin 8)						
3342	100	67.40	34.5	1.71	LF	0.14
	50	67.34	31.8	1.61	LF	0.11
	25	67.27	29.2	1.51	LF	0.10
	10	67.19	25.6	1.37	LF	0.08
	5	67.12	22.7	1.25	LF	0.07
	2	67.01	18.5	1.06	LF	0.04

TABLE 4.4 (Cont'd)

STRUCTURE PERFORMANCE DATA

Location	Flood Event (yr)	U/S W. L. (m)	Discharge (cms)	Structure Velocity (m/s)	Class of Flow	Total Head Loss (m)
SHAWS CREEK						
3462	100	67.68	34.5	1.83	LF	0.24
	50	67.60	31.8	1.74	LF	0.22
	25	67.52	29.2	1.65	LF	0.21
	10	67.41	25.6	1.51	LF	0.19
	5	67.32	22.7	1.39	LF	0.17
	2	67.17	18.5	1.20	LF	0.14
6910	100	72.82	28.7	1.56	PF-WF	0.56
	50	72.79	26.5	1.56	PF-WF	0.58
	25	72.77	24.4	1.56	PF-WF	0.60
	10	72.72	21.4	1.56	PF-WF	0.63
	5	72.69	19.1	1.56	PF-WF	0.64
	2	72.63	15.6	1.56	PF-WF	0.68
7127	100	73.40	28.7	2.97	LF	0.52
	50	73.31	26.6	2.78	LF	0.46
	25	73.20	24.4	2.58	LF	0.38
	10	73.08	21.4	2.27	LF	0.32
	5	72.99	19.1	2.06	LF	0.26
	2	72.84	15.6	1.73	LF	0.19
7700	100	73.66	28.7	1.56	PF	0.14
	50	73.55	26.6	1.45	PF	0.12
	25	73.44	24.4	1.33	PF	0.09
	10	73.30	21.4	1.16	PF	0.07
	5	73.22	19.1	1.04	PF	0.06
	2	73.08	15.6	0.92	LF	0.04
9517	100	74.88	28.7	1.40	PF	0.10
	50	74.81	26.6	1.30	PF	0.08
	25	74.74	24.4	1.19	PF	0.06
	10	74.63	21.4	1.04	PF	0.04
	5	74.55	19.1	0.93	PF	0.03
	2	74.42	15.6	0.76	PF	0.02
10140	100	75.84	17.7	1.45	PF	0.64
	50	75.72	16.4	1.34	PF	0.56
	25	75.58	15.0	1.23	PF	0.48
	10	75.43	13.3	1.09	PF	0.38
	5	75.30	11.8	0.97	PF	0.31
	2	75.11	9.71	0.79	PF	0.24

TABLE 4.4 (Cont'd)

STRUCTURE PERFORMANCE DATA

Location	Flood Event (yr)	U/S W. L. (m)	Discharge (cms)	Structure Velocity (m/s)	Class of Flow	Total Head Loss (m)
E. BR. SAVAGE DRAIN (Basin 9)						
228	100	65.77	19.1	0.66	PF-WF	0.07
	50	65.75	17.7	1.11	PF-WF	0.14
	25	65.69	16.3	1.55	PF-WF	0.19
	10	65.64	14.4	1.77	PF-WF	0.30
	5	65.59	12.9	2.21	PF-WF	0.42
	2	65.31	9.83	2.17	PF	0.46
847	100	65.77	19.1	0.16	PF-WF	0.00
	50	65.76	17.7	0.16	PF-WF	0.01
	25	65.70	16.3	0.16	PF-WF	0.01
	10	65.65	14.4	0.16	PF-WF	0.01
	5	65.59	12.9	0.16	PF-WF	0.00
	2	65.31	9.83	0.16	PF-WF	0.00
1530	100	65.78	18.7	0.16	PF-WF	0.01
	50	65.76	17.5	0.16	PF-WF	0.00
	25	65.70	16.2	0.16	PF-WF	0.00
	10	65.65	14.5	0.16	PF-WF	0.00
	5	65.60	13.0	0.16	PF-WF	0.01
	2	65.32	9.94	0.16	PF-WF	0.00
McKINNONS CREEK (Basin 9)						
808	100	65.83	29.2	0.44	PF-WF	0.02
	50	65.75	27.2	0.52	LF-WF	0.02
	25	65.68	25.0	0.53	LF-WF	0.03
	10	65.55	22.1	1.69	LF	0.04
	5	65.47	19.7	1.57	LF	0.06
	2	65.38	16.2	1.35	LF	0.07
1217	100	66.14	29.2	0.63	PF-WF	0.02
	50	66.10	27.2	0.63	PF-WF	0.02
	25	66.06	25.0	0.53	PF-WF	0.02
	10	66.01	22.1	2.34	PF	0.11
	5	65.97E	19.7	2.08	PF	0.09
	2	65.92E	16.2	1.71	PF	0.02
1753	100	67.03E	29.2	0.47	PF-WF	0.09
	50	67.00	27.2	0.38	PF-WF	0.03
	25	66.97	25.0	0.38	PF-WF	0.03
	10	66.91	22.1	0.38	PF-WF	0.05
	5	66.84	19.7	0.38	PF-WF	0.05
	2	66.73	16.2	0.38	PF-WF	0.04

TABLE 4.4 (Cont'd)

STRUCTURE PERFORMANCE DATA

Location	Flood Event (yr)	U/S W. L. (m)	Discharge (cms)	Structure Velocity (m/s)	Class of Flow	Total Head Loss (m)
McKINNONS CREEK						
2140	100	67.55	31.7	0.48	PF-WF	0.05
	50	67.50	29.4	0.48	PF-WF	0.04
	25	67.46	27.1	0.48	PF-WF	0.05
	10	67.41	23.9	0.48	PF-WF	0.04
	5	67.36	21.5	0.48	PF-WF	0.05
	2	67.21	17.7	0.48	PF-WF	0.08
4010	100	71.63	31.7	2.41	LF	0.34
	50	71.55	29.4	2.31	LF	0.33
	25	71.48	27.1	2.21	LF	0.31
	10	71.38	23.9	2.06	LF	0.29
	5	71.30	21.5	1.94	LF	0.27
	2	71.16	17.7	1.76	LF	0.25
4640	100	76.26	31.7	2.08	LF	0.23
	50	76.15	29.4	2.01	LF	0.21
	25	76.05	27.1	1.93	LF	0.21
	10	75.90	23.9	1.83	LF	0.19
	5	75.79	21.5	1.73	LF	0.18
	2	75.60	17.7	1.59	LF	0.16
5640	100	79.43	27.7	2.15	LF	0.16
	50	79.36	25.7	2.02	LF	0.13
	25	79.29	23.6	1.89	LF	0.11
	10	79.19	20.9	1.72	LF	0.08
	5	79.12	18.7	1.56	LF	0.08
	2	79.02	15.5	1.34	LF	0.06
7900	100	82.65	27.7	1.24	PF-WF	0.33
	50	82.62	25.7	1.24	PF-WF	0.33
	25	82.57	23.6	1.24	PF-WF	0.33
	10	82.52	20.9	1.24	PF-WF	0.33
	5	82.47	18.7	1.24	PF-WF	0.33
	2	82.39	15.5	1.24	PF-WF	0.31
9935	100	84.65	9.57	2.32	LF	0.15
	50	84.63	8.86	2.18	LF	0.13
	25	84.61	8.16	2.06	LF	0.12
	10	84.57	7.19	1.87	LF	0.10
	5	84.54	6.43	1.73	LF	0.08
	2	84.47	5.30	1.53	LF	0.06

TABLE 4.4 (Cont'd)

STRUCTURE PERFORMANCE DATA

Location	Flood Event (yr)	U/S W. L. (m)	Discharge (cms)	Structure Velocity (m/s)	Class of Flow	Total Head Loss (m)
McKINNONS CREEK						
9975	100	85.08	9.57	1.65	LF	0.10
	50	85.02	8.86	1.59	LF	0.09
	25	84.96	8.16	1.53	LF	0.09
	10	84.88	7.19	1.43	LF	0.09
	5	84.81	6.43	1.35	LF	0.10
	2	84.69	5.30	1.25	LF	0.10
BICKERTON DRAIN (Basin 9)						
706	100	65.71	3.82	0.59	LF	0.01
	50	65.63	3.54	0.58	LF	0.02
	25	65.55	3.26	0.57	LF	0.01
	10	65.41	2.89	0.57	LF	0.01
	5	65.29	2.58	0.58	LF	0.02
	2	65.07	2.12	0.61	LF	0.03

Abbreviations:

LF-WF - Low flow and weir flow condition.
The water level is below the low chord of the structure and is flowing over the roadway embankment.

PF-WF - Pressure flow and weir flow condition.
The water level is above the low chord of the structure and is flowing over the roadway embankment.

PF - Pressure flow condition.
The water level is above the low chord of the structure but not over the roadway embankment.

LF - Low flow condition.
The water level is below the low chord of the structure.

U/S - Upstream

W.L. - Water Level

Notes:

1) * - critical depth.
2) E - estimated from plotted curve.

TABLE 4.5

LOCATION OF FLOODED BUILDINGS
100 YEAR FLOOD EVENT

No. of Buildings	Watercourse and Location	Sheet No.	Lot No.	Conc. No.	Estimated Average Depth of Flooding
2	First tributary of Elian Reginbald Dr.	74-28	16	3	0.2
2	Sarsfield Drain, tributary of Elian Reginbald Dr.	70-28	12	4	0.1
1	Bear Brook	70-28	16	5	0.6
7	Bear Brook, downstream of Regional Road 31	66-24	19	9	0.7
2	Bear Brook, upstream of 8th Conc. Road	66-24	19	8	0.7
4	Bear Brook, upstream of 8th Conc. Road	66-24	19	8	0.4
1	Bear Brook, upstream of 8th Conc. Road	66-24	19	8	0.1
1	Bear Brook, upstream of 8th Conc. Road	66-24	18	8	0.1
1	Bear Brook	66-24	17	9	0.4
2	McKinnons Creek	62-28	5	11	0.1
1	Bear Brook, upstream of Regional Road 35	70-24	18	5	0.7
1	Bear Brook, upstream of Regional Road 35	70-24	18	5	0.2

TABLE 4.5 (Cont'd)

LOCATION OF FLOODED BUILDINGS
100 YEAR FLOOD EVENT

No. of Buildings	Watercourse and Location	Sheet No.	Lot No.	Conc. No.	Estimated Average Depth of Flooding
1	Bear Brook, downstream of McNeely Road	70-24	16	6	0.5
1	Bear Brook, upstream of McNeely Road	70-24	16	6	0.2
1	Bear Brook, upstream of McNeely Road	70-24	16	6	0.1
1	McKinnons Creek	62-24	13	10	0.3

5.0 FILL LINE DELINEATION

Certain areas, outside the Regulatory flood plain itself, may not be suitable for development because of the potential risk of erosion and/or slope failure. In other areas, some regulation is required to ensure that excavated material is not deposited in the Regulatory flood plain.

In consultation with the Conservation Authority, guidelines were adopted in order that EGA could delineate the fill line for Bear Brook and its tributaries. Basically, the guidelines are as follows:

(1) Fill line will be plotted as a dashed line and will be located outside of the Regulatory flood plain where appropriate. Where the fill line intersects buildings, the building will be included within the fill line.

(2) The fill line will be plotted as a straight line and will follow the fill line criteria as closely as possible.

(3) The fill line, in areas where steep slopes are not prevalent, will have a minimum setback of 15 metres from the Regulatory floodline.

(4) The fill line, in areas of steep slopes will be setback a distance as indicated by slope setback studies or 15 metres from the top of the bank, whichever is greater.

(5) The fill line may, in certain areas, be a combination of any of the four above.

6.0 TWO-ZONE CONCEPT

As part of the Flood Plain Mapping program, the Conservation Authority in cooperation with the Township of Cumberland wish to consider the applicability of adopting the Two-Zone Floodway-Flood Fringe Concept along Bear Brook and its tributaries.

The Two-Zone Concept is applicable to flood plain lands where the depths of flooding in the overbank areas are generally not more than 1 metre and the velocities are 1 m/s or less.

Upon the delineation of the Regulatory flood plain along Bear Brook and its tributaries, the Project Team instructed EGA Consultants to plot, where possible, the 1-metre, 2-metre and 3-metre depths on the twelve map sheets. This was carried out in order that the Project Team may identify selective areas for the possible implementation of the Two-Zone Concept.

The following three selective areas were identified by the Project Team for the possible implementation of the Two-Zone Concept:

(1) Area 1 consists of flood plain lands along the main channel of Bear Brook, within Lots 17 and 18, Concession 10 and Lots 16, 17 and 18, Concession 11, at the westerly portion of the township boundary. This river reach is located from Cross-sections 22508 to 24585, on Map Sheet 62-24.

There are eleven (11) buildings located within these lots. No building is located within the 100 year flood plain.

(2) Area 2 lies within Lot 17, Concession 7 and Lots 18 and 19, Concession 8, on Map Sheet 66-24. It consists of flood plain lands of Bear Brook and Shaws Creek.

There are eleven (11) buildings located within these lots, of which eight (8) buildings are located inside the 100 year flood plain.

(3) Area 3 appears on Map Sheets 62-28 and 66-28. The lands within Lot 11, Concession 10 are in the McFadden Drain regime, a tributary of East Branch Savage Drain, the lands within Lot 12, Concession 9 are in the McKinnons Creek regime, and the lands within Lots 12 and 13, Concession 8 are in the McWilliams Drain regime.

There are about thirty-six (36) buildings located within these lots, of which none of the buildings is located inside the 100 year flood plain.

The identified areas were reviewed and where applicable, hydraulic analysis was undertaken to determine the potential changes, pertaining to the calculated water surface elevations and the velocities in the channel and overbanks.

6.1 AREA 1

This river reach is fairly straight, with even channel size and little overbanks. The hydraulic model was modified to simulate encroachments within the flood plains, i.e. provide infilling to a depth of 1 metre, and water surface elevations of the 100 year flood were re-generated.

The results (Table 6.1) showed that the increase in the water surface elevations are 0.10 m or less, and are not significant. As expected, the velocities in the left overbanks (VLOB), channel (VCH) and right overbanks (VROB) also increased. The maximum increase of 0.41 m/s was observed in the right overbank at Cross-section 24585. In addition, the velocities in the overbanks are less than 1 m/s. Table 6.2 provides the simulated velocities in the channel and overbanks.

It would appear that it is possible to implement the Two-Zone Concept in Area 1. However, it is recommended that a preliminary engineering study, including field surveys, be carried out to:

(1) Re-assess the increase in the water surface elevations as a result of the infilling.

(2) Determine the extent of the 0.10 m increase in the backwater.

6.2 AREA 2

The flood plain in Area 2 is very wide, measuring at least 1500 m across. The channel size is fairly consistent, averaging 2.3 m in depth. Shaws Creek discharges into Bear Brook in the same vicinity.

A review of the mapping and the results of the hydraulic analysis indicated that infilling to a depth of 1 metre would not:

(1) significantly increase water surface elevations and velocities in the channel and overbanks.

(2) provide a significant reduction in the flood plain lands (1500 m to 1300 m).

The overbanks along the main channel of Bear Brook in Area 2 is considered a backwater zone or ponding area. This ponding effect is also evident in the hydrologic evaluations, whereby peak discharges were significantly attenuated. The flood elevations created in the ponding areas are dependent upon the water surface elevations determined for the main channel. The infilling of the ponding areas would not affect the current hydraulic model developed to simulate the hydraulic characteristics of the main channel. As such, re-modeling would not provide any indication of increased water levels and velocities. However, should the overbank storage be infilled, then downstream discharges would increase and subsequently, so would water levels and velocities.

Due to the importance of maintaining overbank storage, it is recommended that the Conservation Authority not consider this area as a candidate for the potential implementation of the Two-Zone Concept.

6.3 AREA 3

The floodlines in these areas, McFadden Drain to McKinnons Creek to McWilliams Drain, are very complex and the flood elevations are correlated between adjoining watercourses.

A review of the mapping and the results of the hydraulic analysis revealed that the adjoining areas between the watercourses could possibly be infilled for the purposes of development without increasing the calculated water surface elevations and velocities in the channel and overbanks substantially.

Because of the complexity of the inter-relationships of the adjoining watercourses, the Two-Zone Concept cannot be evaluated.

Should development pressures increase in this area, it is recommended that the Authority initiate a preliminary engineering study to address and evaluate appropriate structural measures, such as channelization in conjunction with infilling and dyking, to control the extent of the flood plain of the Regulatory flood.

TABLE 6.1

COMPARISON OF CALCULATED WATER LEVELS

Cross-section Number	Calculated Water Levels (m) Existing	Proposed	Difference
22508	66.01	66.01	0.00
22683	66.08	66.09	0.01
22883	66.14	66.17	0.03
22933	66.27	66.31	0.04
23195	66.32	66.36	0.04
23358	66.38	66.44	0.06
23598	66.45	66.54	0.09
23826	66.53	66.61	0.08
24026	66.66	66.75	0.09
24189	66.71	66.80	0.09
24409	66.76	66.85	0.09
24495	66.75	66.84	0.09
24585	66.89	66.99	0.10

Note: Proposed condition is flood plain encroachment with infilling of 1-metre depth.

TABLE 6.2

COMPARISON OF CALCULATED VELOCITIES

Cross-section Number	Calculated Velocities (m/s)								
	VLOB			VCH			VROB		
	Exist.	Prop.	Diff.	Exist.	Prop.	Diff.	Exist.	Prop.	Diff.
22508	0.12	0.00	-0.12	0.83	0.89	0.06	0.37	0.50	0.13
22683	0.20	0.30	0.10	0.60	0.70	0.10	0.25	0.34	0.09
22883	0.27	0.47	0.20	1.03	1.21	0.18	0.41	0.56	0.15
22933	0.15	0.19	0.04	0.60	0.60	0.00	0.27	0.29	0.02
23195	0.19	0.26	0.07	0.83	0.98	0.15	0.28	0.41	0.13
23358	0.19	0.28	0.09	0.84	1.00	0.16	0.32	0.44	0.12
23598	0.24	0.29	0.05	0.77	0.80	0.03	0.30	0.36	0.06
23826	0.39	0.47	0.08	1.11	1.15	0.04	0.36	0.49	0.13
24026	0.29	0.36	0.07	1.04	1.05	0.01	0.35	0.42	0.07
24189	0.19	0.21	0.02	0.82	0.88	0.06	0.31	0.45	0.14
24409	0.27	0.37	0.10	0.91	0.95	0.04	0.29	0.31	0.02
24495	0.38	0.63	0.25	1.76	1.83	0.07	0.44	0.70	0.26
24585	0.29	0.36	0.07	1.25	1.38	0.13	0.29	0.70	0.41

Notes:
(1) Proposed condition is flood plain encroachment with infilling of 1-metre depth.
(2) Exist. is existing condition.
(3) Prop. is proposed condition.
(4) Diff. is difference.
(5) VLOB is velocity in the left overbank.
(6) VCH is velocity in the channel.
(7) VROB is velocity in the right overbank.

7.0 REMEDIAL MEASURES

In reviewing the Regulatory Flood Risk Maps the following were noted:

(1) A total of twenty-nine buildings were counted along 85 km of the Bear Brook and Tributaries as being within the Regulatory flood plain.

(2) Twenty-two of the buildings are located along the Bear Brook over a distance of 25 km. Sixteen of the buildings are located on Sheet 66-24 or Lot 18, Concessions 8, 9 and 10. The minimum and maximum depths of flooding are 0.1 m and 0.7 m respectively.

(3) Nine of the buildings are located along the Tributaries over a distance of 60 km. The minimum and maximum depth of flooding are 0.1 m and 0.3 m respectively.

Based on the above observations, it becomes very apparent that flooding of structures is not wide spread or significant in depth to cause large flood damages. A flood proofing program could easily be implemented to reduce the potential of flood damages to the existing buildings within the Regulatory flood plain.

For the present, the real issue of flooding along the Bear Brook is related to potential damages to agriculture crops. The Bear Brook is noted for its vast flood plain and the real questions centre around how much land will the spring event flood and how long will the lands remain wet thereby jeopardising spring planting. Furthermore, will summer rainfall events cause sufficient flooding to reduce crop yields along the flood fringe areas.

Although it is not part of the scope of work to address agricultural issues, we did review the following materials and reported our comments separately to the Authority. For the purposes of addressing potential remedial works, we include our review herein as it emphasize the type of remedial works that generally occur along the Bear Brook and the reasons why the measures are adopted.

(A) Report entitled "Bear River Municipal Drain Improvements-Township of Cumberland" by McNeely Engineering Limited dated August 1989.

(B) The information provided by the Authority concerning the proposed channelization measures within Concession 3 Lots 19 and 20 by McNeely Engineering Limited.

Prior to comment, the following should be noted:

(1) The report and information by McNeely Engineering Limited have been prepared to investigate and address agricultural concerns associated with sufficient outlet from fringe growing areas during summer events occurring from May to September.

This type of study differs significantly in comparison to establishing the flood plain of the Bear Brook and potential remedial measures to provide flood protection. The most significant differences are:

(a) Because the watershed is very large, the maximum peak flows and volume of runoff and subsequent water levels will occur during spring events. During this period, as pointed in the McNeely report, little to zero agricultural losses occur and in fact the silt deposited onto the growing areas is actually beneficial.

(b) Potential improvements are considered for summer events ranging from the 2 to 10 year frequency. This means that the actual change in channel configuration is not that great and in fact, most improvements are accomplished by realignments (straightening) and altering (lowering) the channel profile. These measures may have some impact on major flood plains, but for the most part, the impacts are minor and usually local.

(2) Little or no technical information such as cross-sections and backwater computer simulations or any other type of projections have been given in order to comment on possible beneficial or negative impacts. As such, the comments herein are for discussion purposes and not for the purposes of making decisions. Technical verification would be required to quantify the comments in order to provide the decision makers with appropriate information to clearly and confidently make appropriate decisions.

Report entitled "<u>Bear River Municipal Drain Improvements-Township of Cumberland</u>" by McNeely Engineering Limited dated August 1989.

There is no information contained in this report to suggest that the channel improvements would be effective in reducing the Regulatory flood plain.

Assuming the channel configuration is to be slightly modified and that most of the improvements occur in realignment and improved channel gradient, then improvements would be realized for the more frequent events (2 to 10 year) such as summer rainfalls and mild winters. In so far as the local farmers are concerned, improvements for free drainage during the growing season is far more important then whether their land is flooded during the spring runoff period. This is why improvements to channel gradients (especially lowering the creek profile) is far more beneficial then increasing the channel dimensions to carry spring floods.

All of this, of course, reverses itself when farming ceases and the land is sold for potential development. Over time as the watershed is consumed for development purposes, the emphasis will change from sufficient outlet (adequate discharge of agricultural lands) to reducing the vast flood plain.

The type of alternatives considered by the Heritage ward council as opposed to standard "hard core" engineering solutions appears to offer a sound environmental approach with sensible capital works that address a number of key issues. Considering the environmental awareness of everyone to-day, we would encourage this type of program. It should be noted however, that this type of program will have little or no impact on the extent of the Regulatory flood plain.

In order to determine the benefits, if any, of the proposed channel improvements relative to the spring runoff events and flood plains, we would suggest that a hydraulic assessment be undertaken to quantify changes in water levels, channel velocities and the extent of the upstream influence.

The information provided by the Authority concerning the proposed channelization measures within Concession 3 Lots 19 and 20 by McNeely Engineering Limited.

A cover letter, plan and river profile was provided for our review. As noted previously, the information is typical of a agricultural drainage proposal involving channel realignment and gradient improvements.

In reviewing the Regulatory flood plain mapping, it is clear that water levels sharply increase (2.5 metres) within this reach of the Bear Brook. This would suggest that channel improvement may decrease upstream water levels even for the spring events.

Because the proposal is lowering the channel invert by 0.6 metres at 0+600, there is no doubt that the proposed improvements will be effective in providing improved outlet conditions for agricultural purposes. However, there is no information available to quantify the maximum benefit gained. In other words, how much of a reduction will water levels be under the critical summer events and how far does this influence extend upstream.

Based on our experience in developing water management programs, we would expect to see an immediate drop in water levels by a maximum of 0.6 metres for all events including spring conditions. However, if we replot the Regulatory elevations in the immediate area of the proposed improvements, the change in horizontal location of the flood plain is not that significant. Furthermore, the drop in water levels will diminish as one proceeds upstream. Although difficult to assess the extent for the summer events, it would appear that the backwater influence for the Regulatory (100 year spring event) would diminish by County Road No. 35. Since there are no structures being affected within this reach of the Bear Brook, there appears to be no benefit in reducing water levels with respect to flood damages during spring runoff conditions.

The removal of the rock ledge may cause some concern with respect to lower summer water levels upstream of the proposed works. The lower water levels may expose, if any, water intakes for irrigation and drainage outlet works that were previously designed to function under different summer water level conditions.

Since the channel gradient is being increased, channel velocities will also increase within the immediate area of the improvements. Without knowing channel dimensions, we are not able to comment as to whether the velocity increase would potential be creating erosion concerns.

In considering downstream effects, we are of the opinion that the effects of the proposed works are local and would not alter the downstream water regime of the Bear Brook.

8.0 CONCLUSIONS AND RECOMMENDATIONS

8.1 CONCLUSIONS

As a result of the hydrologic and hydraulic analyses the following are concluded:

(1) The following methodologies were used to determine the peak flood flows for the various return frequency events:

(a) Synthetic Unit Hydrograph Method and Single Station Frequency Analysis.

(b) Delisle River Regional Frequency Method.

(c) Regional Regression Method.

(d) Watershed Classification Method.

(e) Index Flood Method

(2) The peak flood flows for the various return frequency events were based on:

(a) The Synthetic Unit Hydrograph Method (snowmelt events) for the main Bear Brook watershed.

(b) The Synthetic Unit Hydrograph Method (rainfall events) for the Elian Reginbald Drain and Bearbrook Drain watersheds (Basins 5 and 6, respectively).

(c) The Synthetic Unit Hydrograph Method (snowmelt events) for the McWilliams Drain, Shaws Creek and McKinnons Creek watersheds (Basins 7, 8 and 9, respectively).

(3) The Manning's 'n' sensitivity analysis demonstrated that a 10% deviation in the values would not significantly alter the simulated Regulatory flood plain.

A change of more than 0.1 m occurred along Bear Brook, from Chainage 1520 to 6133. This meandering river reach has little overbank flow, and hence, produced minimal horizontal displacement.

(4) About 52% of the bridge structures crossing Bear Brook and its tributaries can discharge the various flood events, without weir flow occurring over the roadway embankment. The breakdown for the Regulatory flood event is 27 under low flow conditions, 8 under pressure flow conditions, 32 under pressure and weir flow conditions, and 1 under low and weir flow conditions.

(5) In reviewing the 100 year flood plain:

(a) It was estimated that 29 buildings are within the flood plain. Because the maximum depth of flooding is less than 0.7 m, the flood damage to existing buildings would be minimal.

(b) A flood proofing program would eliminated flood damages to existing buildings within the Regulatory flood plain.

(c) There exist a constriction from Chainage 6912 to 7270. This river reach, about 358 metres long, has an oxbow and produced a significant difference of 2.18 m in the computed water levels, from 60.72 m to 62.90 m.

(d) From Chainage 18396 to 21048, there exist a very large area of the flood plain. The width of the flood plain is more than 1500 metres. The simulated water levels varied from 65.66 m to 65.70 m, a difference of 0.04 m in 2652 m length.

(6) The river reach, located at the westerly limit of the township boundary near Carlsbad Springs, from Chainage 22508 to 24585, appears to be suitable for the implementation of the Two-Zone Concept.

The resultant flood and fill lines were plotted on the South Nation River Conservation Authority's twelve Flood Risk Maps.

8.2 RECOMMENDATIONS

In order to establish limits of community development and institute land use practices consistent with environmental limitations, the following measures are recommended:

(1) The Conservation Authority accept the flood and fill lines as delineated on the twelve Flood Risk Maps, as the extent of hazard lands designation adequate for future zoning. That is, the lands lying within the flood and fill line delineations be considered as being susceptible to flooding and subject to erosion and potential slope failure.

(2) The Conservation Authority, in cooperation with the Township and the Ministry, prepare Official Plan Policies and Zoning By-Laws covering the regulation of Bear Brook and its tributaries in Cumberland Township, in accordance with the Provincial objectives of water management.

(3) The Conservation Authority should consider the merits of preparing and implementing a flood proofing program to alleviate flood damages to the buildings located within the Regulatory flood plain.

(4) The developed hydraulic computer models should be used to assess the effect of any proposed changes to the Bear Brook, Elian Reginbald Drain, Bearbrook Drain, McWilliams Drain, Shaws Creek, and McKinnons Creek hydraulic systems. Should any proposed changes be constructed, then the computer models must be updated to reflect current hydraulic conditions.

(5) The Conservation Authority not consider the implementation of the Two-Zone Concept along Bear Brook, pending further detailed hydrologic and hydraulic analyses.

APPENDIX A

TIME TO PEAK EQUATIONS

Time To Peak Equations

1) MTC Methodology

Bransby Williams equation:

Tc = (0.057 L)/((S**0.2)(A**0.1))

where	Tc	=	time of concentration (min)
	L	=	length of main channel to head of basin including undefined portion of channel (m)
	S	=	net slope (%)
	A	=	effective watershed area (ha)

Then, utilizing the SCS method:

Tp = Tl + DD/2

where	Tp	=	time to peak of unit hydrograph (hr)
	Tl	=	0.6 Tc
		=	lag time of watershed (hr)
	DD	=	0.133 Tc
		=	rainfall excess period (hr)

2) SCS Methodology

Kirpich equation:

Tc = ((11.9(L**3))/H)**0.385

where	Tc	=	time of concentration (hr)
	L	=	length of longest watercourse (mi)
	H	=	elevation difference (ft)

Then,

Tp = Tl + DD/2

3) HYMO Methodology

Williams equation:

Tp = 4.63(A**0.422)(S**-0.46)((L**2/A)**0.133)

where	A	=	watershed area (sq mi)
	S	=	flood plain slope (ft/mi)
	L	=	hydraulic length (mi)

APPENDIX B

REFERENCES AND ABBREVIATIONS

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ABBREVIATIONS

mi	miles
sq mi, mi2	square miles
km	kilometres
sq km, km2	square kilometres
m	metres
m2	square metres
ft	feet
ft2	square feet
hr	hours
ft/mi	feet per mile
in	inches
mm	millimetres
cm	centimetres
cfs, ft3/s	cubic feet per second
cms, m3/s	cubic metres per second
ft/s	feet per second
ft/ft	feet per foot
m/s	metres per second
m/m	metres per metre
yr	years
in/day	inches per day
mm/day	millimetres per day
mm/hr	millimetres per hour
min	minutes

ha	hectares
Low Chord (LC)	The highest elevation of a bridge opening or the elevation where pressure flow assumes control
TOR	Top of Road
CN	curve number (used in hydrology)
Ref.	reference
**	used in an equation represents "to the power of"
<=	less than or equal to
PHU	primary hydrologic unit (basins and reaches)
HSG	hydrologic soil group (a classification)
>	greater than
vol	volume
R	reach
B	basin
MTO, MOT	Ministry of Transportation of Ontario
SCS	Soil Conservation Services
AES	Atmospheric Environment Service
WCM	Watershed Classification Method
incr.	incremental
accum.	accumulative

APPENDIX C

BEAR BROOK HYDRAULICS STUDY
SHORT REPORT

BY WATER PLANNING AND MANAGEMENT BRANCH
ENVIRONMENT CANADA

Environment Canada

Bear Brook Hydraulics Study

Short Report

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Water Planning & Management Branch
March 20, 1991

Bear Brook Hydraulics Study

1.0 Limits of Study

This hydraulics analysis consists of cross sections from 6203 to 8683. Within the study limits, the reach of Bear Brook was subdivided into 4 parts consisting cross sections from: 6203 to 7005; 7005 to 7170; 7170 to 7819; and 7819 to 8683. However, the HEC-2 computations were carried out in three segments consisting of cross sections from: 6203 to 7005; 7005 to 7170 & 7170 to 8683.

2.0 Conditions of Analysis

This analysis was generally divided into 2 components: under **existing conditions** and with **channel improvements.** Although calculations for both components were completed, only the results under the existing conditions are presented in this report. The water surface profiles were calculated by the HEC-2 program for the t-events of: 100, 50, 20, 10, 5 & 2 year return periods.

3.0 Cross Sectional Data

Three different sets of data were used in the preparation of cross sectional data from:

3.1 Garatech, between cross sections 6203 to 7005 and 7819 to 8683.
3.2 McNeeley, between cross sections 7005 to 7620.
3.3 contour lines on map, sheet no. 74-24.

The priority in the use of cross sectional data is discussed below. First, all design data provided by McNeeley were used over other data in case of conflicts. Secondly, the data used by Garatech in their HEC-2 computations were incorporated into this present analysis. Thirdly, wherever the cross sections needed to be extended horizontally to a level higher than the calculated 100-year water levels, the elevations were taken from the contours of the map.

The survey and design data provided by McNeeley were identified as Cross Sections A, B, C, D, E, & F. The corresponding numbers for this study are listed in Table 1.0.

Table 1.0 Cross Section Numbers

WPM	McNeeley
7005	F
7090	E
7170	D
7270	C
7470	B
7620	A

4.0 Roughness, Contraction and Expansion Coefficients

Manning's 'n' values and the Contraction and Expansion coefficients were taken from the data provided by Garatech and summarized in Table 2.0.

Table 2.0 Manning's Roughness Coefficients

Cross Section	Left Overbank	Channel	Right Overbank	Contraction Coefficient	Expansion Coefficient
6203	.055	.040	.065	.3	.5
6359	.060	.040	.065	.1	.3
6558	.060	.040	.065	.1	.3
6912	.060	.040	.060	.1	.3
6947	.060	.040	.060	.1	.3
7005	.060	.040	.060	.1	.3
7090	.060	.040	.060	.1	.3
7170	.060	.040	.060	.1	.3
7270	.070	.040	.060	.1	.3
7470	.060	.040	.060	.1	.3
7620	.060	.040	.060	.1	.3
7700	.060	.040	.060	.1	.3
7740	.060	.040	.060	.1	.3
7790	.060	.040	.060	.1	.3
7819	.060	.040	.060	.1	.3
8109	.055	.040	.050	.1	.3
8229	.055	.040	.050	.3	.5
8250	.055	.024	.050	.3	.5
8256	.055	.024	.050	.3	.5
8275	.050	.040	.050	.1	.3
8403	.050	.040	.050	.1	.3
8683	.050	.040	.050	.1	.3

5.0 Water Surface Profile Computation

The calculation of the water surface profiles by the HEC-2 program started at Cross Section 6203 which is located 23 metres from the upstream side of the bridge at the Concession Road. The water surface elevations at Cross Section 6203 to begin the HEC-2 calculations for the t-event floods taken from the calculations provided by Garatech are summarized in Table 3.0.

Table 3.0 Starting Water Surface Elevations

Cross Section	100-Year	50-Year	20-Year	10-Year	5-Year	2-Year
6203	59.44	59.22	59.05	58.82	58.65	58.41

The HEC-2 program was used to calculate the water levels for the t-events. In the first part of the computation process, the calculation proceeded upstream from Cross Section 6203 to Cross Section 7005 where the split option began for the second part of the analysis.

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In the second part of the calculation, only 3 cross sections: 7005, 7090, & 7170 were included. The water surface elevations calculated for the Cross Section 7005 in the first part were used as starting water surface elevations for these calculations for the second part. The flow was split into 2 components: the channel and the oxbow. The reason to carry out this split flow calculations was that some discrepancies in the flows through the channel and around the oxbow were found in the previous study by Garatech. The water levels calculated by Garatech are summarized, with their corresponding flows, in Table 4.0.

Table 4.0 Calculated 100-Year Water Levels and Flow Rates by Garatech

Cross Section	Water Level	Flow Rate Left Overbank	Flow Rate Channel	Flow Rate Right Overbank
7005	60.85	35	160	5
7053	61.30	14	175	10
7155	62.51	35	148	16

At Cross Section 7170 in then present study, the flow was initially assumed to be contained within the channel and the oxbow, i.e. no increase of flow due to the drainage ditch along the north side of the farm. This extra flow was not considered to significantly alter the water levels calculated for the oxbow. An iterative procedure was adopted to calculate the t-event water levels and their corresponding flows through the channel and around the oxbow. The results of these calculated water levels and the corresponding flows are summarized in Table 5.0.

Table 5.0 Calculated Water Levels and Corresponding Flows Through Channel & Oxbow

Cross Section	100-Year Flow	100-Year Elev.	50-Year Flow	50-Year Elev.	20-Year Flow	20-Year Elev.	10-Year Flow	10-Year Elev.	5-Year Flow	5-Year Elev.	2-Year Flow	2-Year Elev.
Channel												
7005	141.4	61.06	133.5	60.99	125.4	60.88	114.0	60.76	104.3	60.62	89.8	60.43
7090	141.4	61.94	133.5	61.83	125.4	61.72	114.0	61.56	104.3	61.42	89.8	61.42
7170	141.4	62.65	133.5	62.53	125.4	62.39	114.0	62.20	104.3	62.02	89.8	61.74
Oxbow												
7005	58.6	61.06	49.5	60.99	40.6	60.88	30.0	60.76	21.7	60.62	12.2	60.43
7090	58.6	62.05	49.5	61.96	40.6	61.85	30.0	61.70	21.7	61.57	12.2	61.37
7170	58.6	62.65	49.5	62.53	40.6	62.39	30.0	62.20	21.7	62.02	12.2	61.74

The flow through the oxbow was considered to be equivalent to the left overbank flow in the Garatech analysis.

The third and fourth parts of the HEC-2 calculations were combined from Cross Section 7170 to 8683. The input data for Cross Sections 7170 to 7790 were taken from the survey and design notes provided by McNeeley for the channel and from contour lines for the overbank areas. For cross sections 7819 to 8683, the input data were largely taken from the previous study provided by Garatech and, with the aid of the map contours, horizontal extensions to elevations above the calculated water levels. Several cross sections were added. The locations of these additional cross sections in this present study correspond to the locations as indicated by McNeeley. The numbering of the cross sections in this present study followed the practice used by Garatech for ease of comparison. (See Figure 1.0 for locations of cross sections.) It was found that the channel inverts provided by McNeeley were generally higher than those used by Garatech for the corresponding cross sections. It was decided that wherever they were available the data from McNeeley would be used in priority over the others. The comparison of these channel inverts are summarized in Table 6.0.

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Cross Section	Table 6.0 Comparison of Channel Inverts McNeeley	Garatech
7005	57.90	57.12
7053	-	57.21
7090	58.00	-
7155	-	57.38
7170	58.40	-
7270	58.90	-
7273	-	57.57
7470	58.90	-
7481	-	57.92
7620	59.10	-

The calculated water surface elevations from Cross Section 6203 to Cross Section 8683 of this present study are summarized in Table 7.0.

Table 7.0 Summary of Calculated Water Surface Elevations

Cross Section	100-Year Chan.	100-Year Oxbow	50-Year Chan.	50-Year Oxbow	20-Year Chan.	20-Year Oxbow	10-Year Chan.	10-Year Oxbow	5-Year Chan.	5-Year Oxbow	2-Year Chan.	2-Year Oxbow
6203	59.44		59.22		59.05		58.82		58.65		58.41	
6359	59.92		59.73		59.56		59.32		59.13		58.85	
6558	60.34		60.16		60.00		59.77		59.58		59.30	
6912	60.72		60.57		60.43		60.24		60.06		59.81	
6947	60.77		60.62		60.48		60.26		60.08		59.83	
7005	61.06		60.99		60.88		60.76		60.62		60.43	
7090	61.94	62.05	61.83	61.96	61.72	61.85	61.56	61.70	61.42	61.57	61.20	61.37
7170	62.65		62.53		62.39		62.20		62.02		61.74	
7270	62.90		62.78		62.65		62.47		62.30		62.03	
7470	63.25		63.14		63.01		62.83		62.65		62.35	
7620	63.31		63.21		63.09		62.92		62.76		62.50	
7700	63.31		63.21		63.10		62.93		62.77		62.50	
7740	63.31		63.22		63.10		62.93		62.77		62.51	
7790	63.32		63.22		63.10		62.94		62.77		62.51	
7819	63.32		63.22		63.11		62.94		62.78		62.51	
8109	63.27		63.17		62.94		62.71		62.59		62.41	
8229	63.85		63.78		63.73		63.58		63.38		63.02	
8250	63.76		63.71		63.67		63.54		63.36		63.04	
8256	63.89		63.83		63.76		63.60		63.39		63.04	
8275	64.18		64.07		63.96		63.75		63.48		63.06	
8403	64.20		64.10		64.00		63.80		63.57		63.20	
8683	64.23		64.13		64.03		63.83		63.61		63.26	

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The comparison of the 100-year water surface elevations as calculated presently in this study and previously by Garatech are summarized in Table 8.0.

Table 8.0 Summary of Calculated 100-Year Water Surface Elevations

Water Levels	Cross Section		Water Levels
WPM	WPM	Garatech	Garatech
59.44		6203	59.44
59.92		6359	59.92
60.34		6558	60.34
60.72		6912	60.72
60.77	6947	-	-
61.06	-	7005	60.86
-	-	7053	61.30
61.94	7090	-	-
-	-	7155	62.51
62.65	7170	-	-
62.90	7270	-	-
-	-	7273	62.52
63.25	7470	-	-
-	-	7481	63.13
63.31	7620	-	-
63.31	7700	-	-
63.31	7740	-	-
63.32	7790	-	-
63.32		7819	63.24
63.27		8109	63.42
63.85		8229	63.82
63.76		8250	63.73
63.89		8256	63.87
64.18		8275	64.15
64.20		8403	64.18
64.23		8683	64.21

Bear Brook Hydraulics Study
Addendum

Additional Hydraulics Analysis

A hydraulic analysis of the 100-year flood event has been carried out for a short reach of Bear Brook between cross sections 6203 to 8683 (map sheet no. 74-24). A set of new ground data provided by McNeeley Engineering's field survey measurements was used for the cross section data. This field data showed that the stream bottom elevations were up to 0.6 metre higher than those used in the previous hydraulic model. The purpose of this analysis was to determine the impact on water levels and the mapping. Several supplementary cross sections have also been added with ground elevations taken from the contour map. Wherever the calculated water levels showed higher elevations, the cross sections were extended horizontally to higher elevations with the aid of the contours on the topographic map.

Split Flow Analysis

To determine the correct water levels and their corresponding flows through the main channel and the overflow channel for cross sections 7005, 7090, and 7170, two independent series of HEC-2 calculations have been carried out. The basic method was an iterative split flow analysis. One series of calculations was carried out along the main channel, and the other series along the overflow channel. An iterative process was used starting with common water level at the downstream cross section No. 6947 and balancing the calculated water level at the common upstream cross section No. 7270. By repeating the calculations, a reduced flow of 140 cms along the main channel produced a water level at Cross Section 7270 consistent with the water level for a flow of 60 cms calculated for the overflow channel.

When HEC-2 calculations were carried out along only the main channel with 140 cms applied to cross sections 7005, 7090, & 7170, the final calculated water levels were generally higher than previously calculated levels (column 5, Table A.1) by 0.18 metres at Cross Section 7819 but quickly converged at Cross Section 8229. The results are listed in column 2, Main Channel Flow Only of Table A.1.

When the calculations were carried out for the entire floodplain, i.e. the overflow channel was considered to be part of the section overbank, water levels are similar to the levels calculated for only the main channel. These results are listed in column 3, Floodplain Total Flow of Table A.1. Therefore, it is concluded that only the water surface profile calculated for the entire floodplain is required and no further analysis is necessary.

Analysis of Proposed Dredging Effects

To analyze the effects of the McNeeley's proposed design for dredging, a new set of design ground elevations were used to replace the existing elevations for the appropriate cross sections. The HEC-2 calculations were carried out. The results are listed in column 4, Floodplain With Dredging of Table A.1. These calculations showed levels significantly lower than the other profiles.

All calculations showed convergence near Cross Section 8229, with very little difference in profiles above the bridge just upstream of this section.

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Table A.1 - Comparison of New and Previously Calculated Water Levels

Cross Section	Main Channel Flow Only	Floodplain Total Flow	Floodplain With Dredging	Previous Water Levels
6203	59.44	59.44	59.44	59.44
6359	59.92	59.92	59.92	59.92
6558	60.34	60.34	60.34	60.34
6912	60.72	60.72	60.72	60.72
6947	60.77	60.77	60.77	
7005	60.74	61.06	60.98	60.86
7053				61.30
7090	62.01	62.01	61.39	
7155				62.51
7170	62.67	62.77	62.10	
7270	63.11	63.04	62.37	
7273				62.52
7470	63.36	63.32	62.89	
7481				63.13
7620	63.41	63.37	63.03	
7700	63.41	63.38		
7740	63.41	63.38		
7790	63.42	63.38		
7819	63.42	63.38	63.04	63.24
8109	63.55	63.52	63.28	63.42
8229	63.84	63.83	63.84	63.82
8250	63.75	63.74	63.76	63.73
8256	63.88	63.88	63.89	63.87
8275	64.17	64.16	64.17	64.15
8403	64.20	64.19	64.20	64.18
8683	64.23	64.22	64.23	64.21

APPENDIX D

ADDENDUM HYDRAULICS REPORT
BEAR BROOK
TOWNSHIP OF CUMBERLAND

BY EGA CONSULTANTS

South Nation River Conservation Authority

ADDENDUM HYDRAULICS REPORT

BEAR BROOK

TOWNSHIP OF CUMBERLAND

SEPTEMBER 1991

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1.0 INTRODUCTION

1.1 STUDY OBJECTIVE

The South Nation River Conservation Authority retained EGA Consultants (Ecos Garatech Associates Ltd.) to undertake a hydraulic assessment of the constructed channel improvements along Bear Brook in the vicinity of Elian Reginbald Drain.

The objectives of the assessment were:

(1) Update the hydraulic model of Bear Brook to incorporate the channel improvements.

(2) Re-generate water surface profiles for the 100, 50, 25, 10, 5 and 2 year flood events.

(3) Update the Regulatory Flood Risk Map, Sheet No. 74-24.

1.2 BACKGROUND INFORMATION

Pertinent background information on the channel improvements, pertaining to the channel alignment, typical cross-section and existing and constructed channel profiles were obtained from the Conservation Authority.

1.3 CONTRACT AND STUDY LIMITS

The extent of the contract limit of the channelization works covered a 840-metre river reach of Bear Brook in the vicinity of Elian Reginbald Drain, located within Lot 19, Concession 3.

Chainage 0+000 m of the channel alignment commenced at the bridge crossing located in Concession 4 (Cross-Section 8250), and increased downstream. Construction of drain improvements began at Section "A" (Chainage 0+475 m) and stopped at Section "B" (Chainage 1+315 m). Section "A" was equivalent to Cross-Section 7778 and Section "B" coincided with Cross-Section 6938 of the HEC-2 model.

A plan of the river reach illustrating the location of Sections "A" and "B" and the alignment of the drain improvements is provided in Appendix A.

2.0 HYDRAULICS

2.1 HYDRAULIC MODEL

The existing hydraulic model, obtained from the Flood Plain Mapping Study, was modified accordingly to reflect the channel improvements.

Where applicable, the typical cross-section and channel inverts were revised in accordance to the information provided by the Conservation Authority. The profile and typical cross-section of the improvements are provided in Appendix A.

2.2 WATER SURFACE PROFILES

The revised hydraulic model was used in conjunction with the HEC-2 program to re-generate the water surface profiles for the 100, 50, 25, 10, 5 and 2 year flood events.

2.3 RESULTS

The updated 100 year flood plain, as a result of the channelization measures, was plotted on the South Nation River Conservation Authority's Flood Risk Mapping, Sheet No. 74-24.

The results of the hydraulic assessment are:

(1) The improvements produced substantial reductions in the 100 year water surface elevations. Reductions varied from 0.10 m at Cross-Section 7005 to 0.67 m at Cross-Section 7270.

The calculated water surface elevations of the 100 year flood event for the existing and improved hydraulic conditions are given in Table 2.1.

(2) The bridge crossing located upstream of the improvements in Concession 4 experienced minimal changes in the hydraulic performance of the structure.

The water level of the 2 year flood was reduced by 0.22 m upstream of the structure (see Table 2.2).

The revised structure performance data of the bridge is provided in Table 2.3.

TABLE 2.1

CALCULATED WATER SURFACE ELEVATIONS

100 YEAR FLOOD EVENT

Cross-Section Number	Chainage (Per McNeely) (m)	CWSEL Existing (m)	CWSEL Proposed (m)	CWSEL Difference (m)
6912	1+341	60.72	60.72	0.00
6947*	1+306	60.77	60.78	+0.01
7005*	1+248	61.06	60.96	-0.10
7090*	1+163	62.01	61.41	-0.60
7170*	1+083	62.77	62.12	-0.65
7270*	0+983	63.04	62.37	-0.67
7470*	0+783	63.32	62.91	-0.41
7620*	0+633	63.37	63.04	-0.33
7700*	0+553	63.38	63.05	-0.33
7740*	0+513	63.38	63.05	-0.33
7790	0+463	63.38	63.06	-0.32
7819	0+434	63.38	63.06	-0.32
8109	0+144	63.52	63.28	-0.24
8229	0+024	63.83	63.84	+0.01
8250	0+003	63.74	63.76	+0.02
CON. 4 Bridge				
8256	N/A	63.88	63.89	+0.01
8275	N/A	64.16	64.17	+0.01
8403	N/A	64.19	64.20	+0.01
8683	N/A	64.22	64.23	+0.01

Notes:

* -- Cross-Sections within Contract Limits
CWSEL -- denotes Calculated Water Surface Elevations
N/A -- Not Applicable

TABLE 2.2

CALCULATED WATER SURFACE ELEVATIONS

2 YEAR FLOOD EVENT

Cross-Section Number	Chainage (Per McNeely) (m)	CWSEL Existing (m)	CWSEL Proposed (m)	CWSEL Difference (m)
6912	1+341	59.81	59.81	0.00
6947*	1+306	59.83	59.83	0.00
7005*	1+248	60.43	60.27	-0.16
7090*	1+163	61.09	60.73	-0.36
7170*	1+083	61.88	61.10	-0.78
7270*	0+983	62.11	61.39	-0.72
7470*	0+783	62.40	61.76	-0.64
7620*	0+633	62.53	62.01	-0.52
7700*	0+553	62.54	62.03	-0.51
7740*	0+513	62.54	62.04	-0.50
7790	0+463	62.55	62.05	-0.50
7819	0+434	62.55	62.06	-0.49
8109	0+144	62.70	62.13	-0.57
8229	0+024	63.15	62.92	-0.23
8250	0+003	63.15	62.95	-0.20
CON. 4 Bridge				
8256	N/A	63.16	62.96	-0.20
8275	N/A	63.19	62.97	-0.22
8403	N/A	63.30	63.14	-0.16
8683	N/A	63.35	63.21	-0.14

Notes:

* -- Cross-Sections within Contract Limits
CWSEL -- denotes Calculated Water Surface Elevations
N/A -- Not Applicable

TABLE 2.3

STRUCTURE PERFORMANCE DATA

Location	Flood Event (yr)	U/S W. L. (m)	Discharge (cms)	Structure Velocity (m/s)	Class of Flow	Total Head Loss (m)
BEAR BROOK (Main Channel)						
8250	100	64.17	179	2.40	PF-WF	0.29
	50	64.08	165	2.26	PF-WF	0.26
	25	63.96	150	2.04	PF	0.21
	10	63.81	130	1.78	PF	0.18
	5	63.43	114	1.56	PF	0.09
	2	62.97	92.1	1.55	LF	0.04

3.0 SUMMARY

In June of 1987, the South Nation River Conservation Authority retained EGA Consultants (Ecos Garatech Associates Ltd.) to undertake a flood and fill line delineation, as well as an assessment of the applicability of adopting the Two-Zone Concept along Bear Brook within the Township of Cumberland.

During the course of the Study and before the submission of the final Study Report and associated Flood Risk Maps, the Township of Cumberland awarded the Bear Brook Municipal Drain Improvements Project to McNeely Engineering Ltd. Final design was completed and construction works were to commence in the summer of 1991. Therefore, the Conservation Authority retained EGA Consultants to undertake the hydraulic assessment to update the hydraulic model and Flood Risk Map as a result of the channel improvements.

The extent of the channel improvements covered a 840-metre river reach along Bear Brook in the vicinity of Elian Reginbald Drain, located within Lot 19, Concession 3, Cumberland Township.

Final design information, pertaining to existing and constructed river profiles, typical cross-section and alignment of the channelization works were obtained from the Conservation Authority.

The existing hydraulic model was modified accordingly to reflect the channelization works. Water surface profiles were re-generated for the Regulatory (100 year) and the 50, 25, 10, 5 and 2 year flood events. The updated Regulatory flood plain was plotted on the Conservation Authority's Flood Risk Map, Sheet No. 74-24.

The channel improvements produced substantial reductions of 0.24 m to 0.67 m for the 100 year flood event between Cross-Sections 7090 and 8109. Insignificant changes in the structure performance were noted at the bridge crossing located in Concession 4 (Cross-Section 8250).

APPENDIX A

PLAN, PROFILE AND SECTION OF DRAIN IMPROVEMENTS

(Source: South Nation River Conservation Authority)

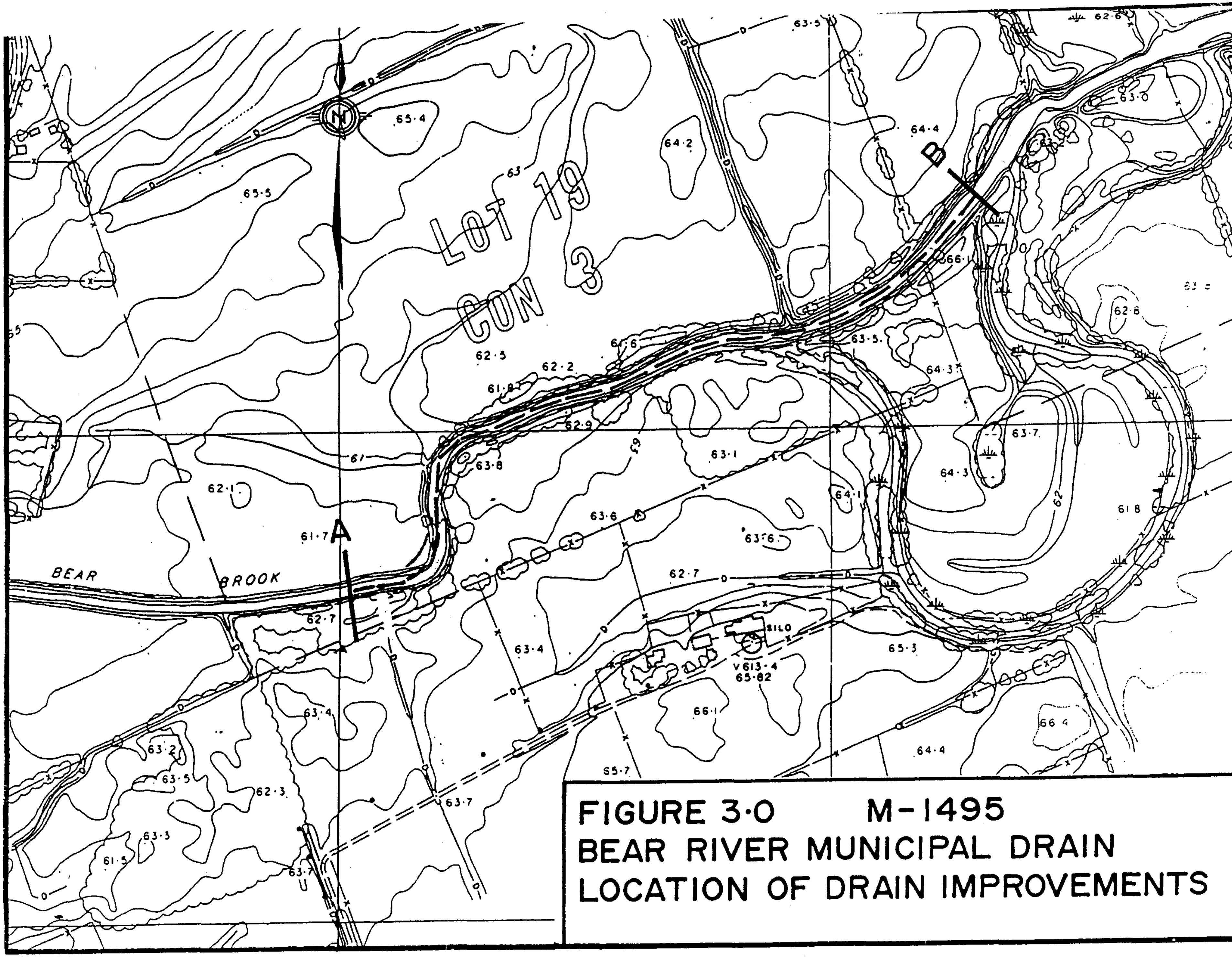
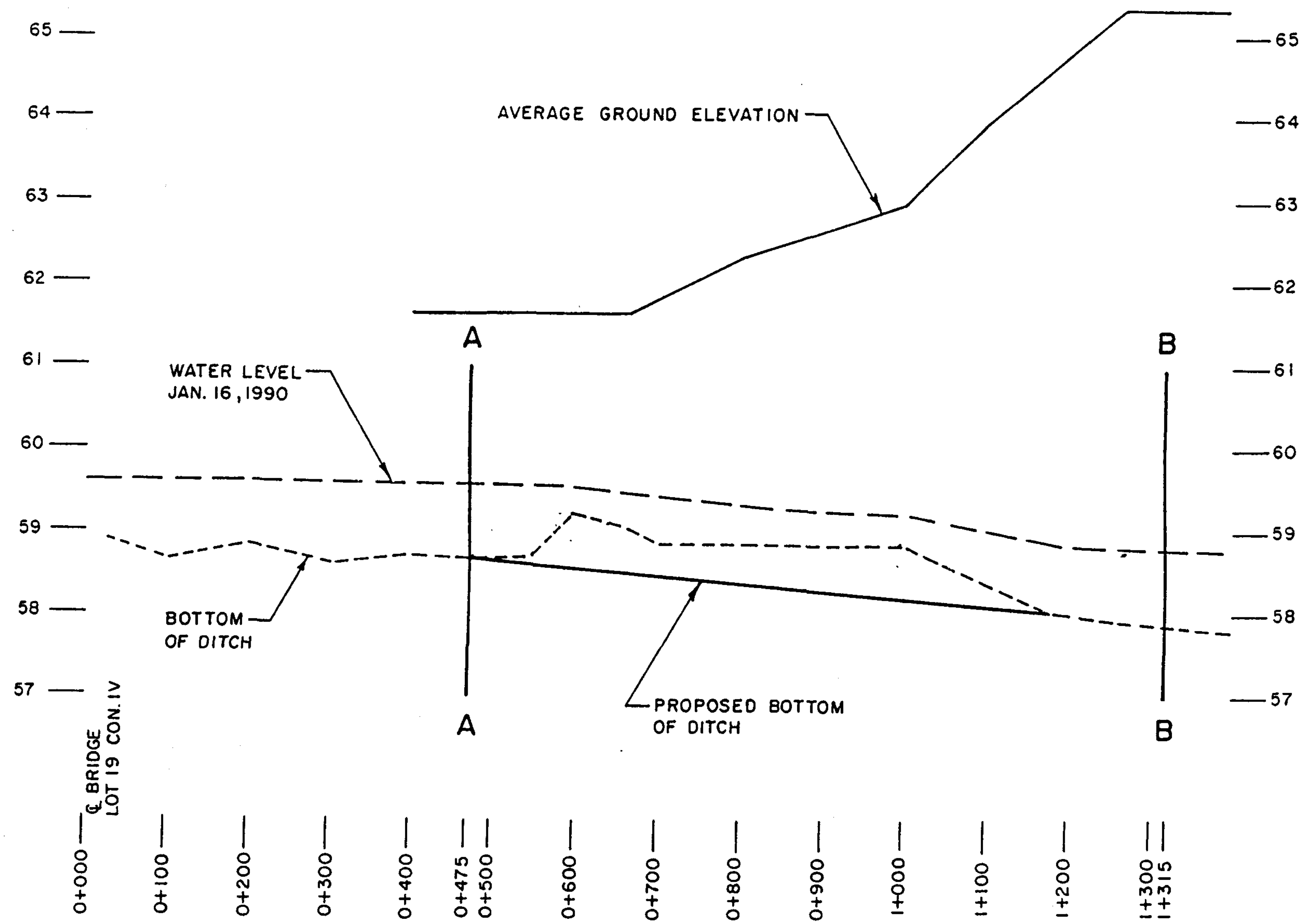
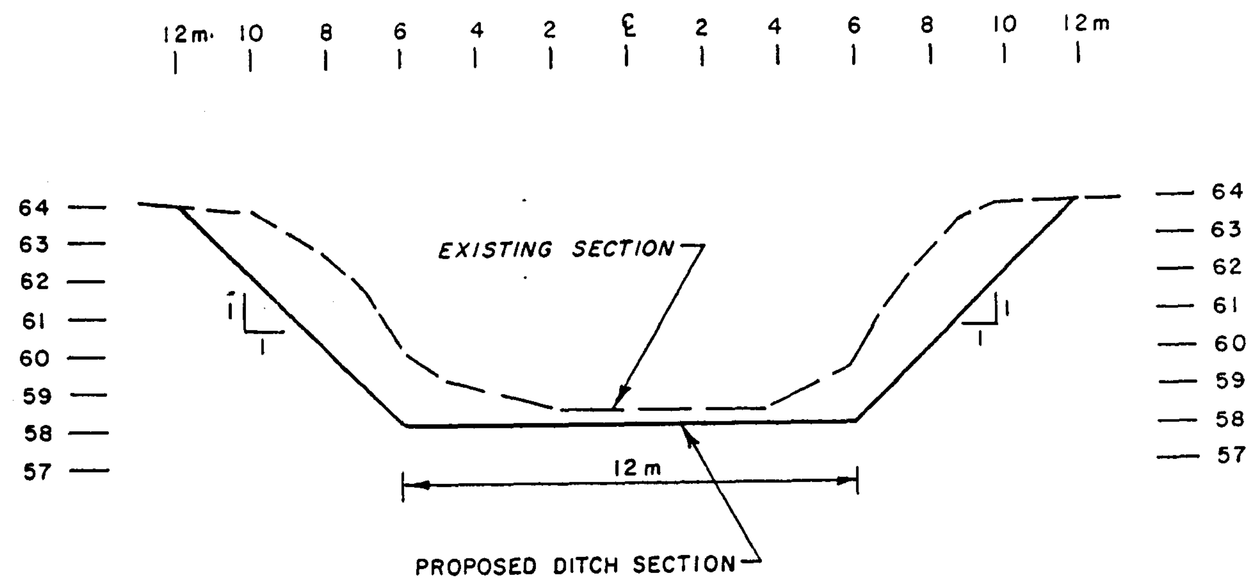


FIGURE 3·0 M-1495
BEAR RIVER MUNICIPAL DRAIN
LOCATION OF DRAIN IMPROVEMENTS



BEAR RIVER MUNICIPAL DRAIN
PROFILE OF DRAIN IMPROVEMENTS



BEAR RIVER MUNICIPAL DRAIN
SECTION OF DRAIN IMPROVEMENTS