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11688-0

9 February 1983

The Chairman and Members
Rideau Valley Conservation Authority
Box 55, Mill Street
Manotick, Ontario
K0A 2N0

Rideau River Floodline Mapping Tributaries -
Smiths Falls to Kars

Dear Sirs:

By means of an engineering agreement dated October 1979, our firm was authorized to undertake a floodplain mapping study of the Tributaries to the Rideau River from Smiths Falls to Kars.

The floodline mapping and determination of fill lines have now been completed. The results of our studies are presented in the enclosed report along with the detailed technical considerations.

We would like to express our sincere appreciation to all appropriate members of the Authority staff and the Ministry of Natural Resources for their cooperation and assistance throughout the course of this study. All of which is respectfully submitted.

Yours very truly,

JAMES F. MacLAREN LIMITED

H.W. Cornfield, P.Eng.
Project Director

HWC/wb

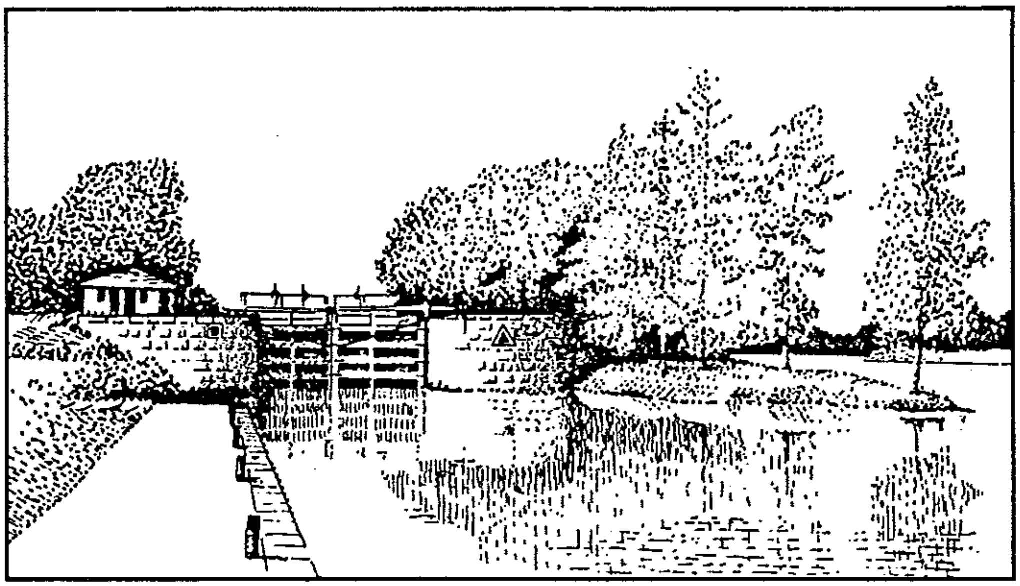
encl.

Rideau River Floodline Mapping

Tributaries – Smiths Falls to Kars

Report to

Rideau Valley Conservation Authority



February 1983

MacLaren

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1.0 INTRODUCTION

In June of 1976, floodline mapping of the Rideau River between Smiths Falls and Kars was completed by James F. MacLaren Ltd. Flood lines associated with the 1:100 year flood were defined by performing a flood frequency analyses of historic stream flow data and 1:100 year water surface elevations were computed by combining rating curves of numerous canal and water level control structures with backwater modelling on river reaches between lock stations.

In January of 1979, the Rideau River floodline mapping was extended through the Town of Smiths Falls from Old Slys Locks to the Poonamalie Dam.

The "original" 1976 mapping from Kars to Smiths Falls covered a limited study area, insofar as the width of the mapped portion of the floodplain was restricted. As a result, in several locations where the floodplain is very wide due to poor topographical definition of the river valley, or at confluences with Rideau River tributaries, the flood lines were terminated and the "Limit of Study" identified.

The objective of this supplementary study, authorized by the Rideau Valley Conservation Authority in August of 1979 was to extend the limit of the mapped portion of the flood plain by providing floodlines on the first mile (1.6 kilometres) of minor tributaries from their confluence with the Rideau, by laterally extending the mapped area in zones of excessively wide floodplain, and in the case of Kemptville Creek, by extending flood lines from the Rideau River upstream to the probable limit of future development in the Town of Kemptville (selected as the Canadian Pacific Railway crossing north of the hamlet of Oxford Mills).

The supplementary study has involved four main work activities:

a) Field Work - collection of data related to physical dimensions of watercourse crossings, roughness coefficients, below water watercourse cross-sections.

b) Hydrology - developing estimates of the 1:100, 1:50, 1:25, 1:10 and 1:5 year design flows on tributaries of the Rideau River.

- assessing the relative magnitudes of summer rainstorm events and spring snow-melt events on a frequency basis, as they pertain to runoff on the small tributary watersheds.

c) Hydraulics - computation of water surface elevations.

d) Mapping - plotting of the Regional Storm flood line on orthophoto mosaic mapping.

This report documents the findings and conclusions identified through the study and the assumptions upon which they are based.

2.0 DESIGN FLOWS

In order to compute water surface elevations on tributaries of the Rideau River, it was first required to arrive at estimates of flow on each tributary for the 1:5, 1:10, 1:25, 1:50 and 1:100 year design events.

A substantial amount of effort was required to resolve the question of whether the Regional (1:100 year) flood is likely to be associated with a summer rainstorm or spring snowmelt event on the smaller tributaries such as Rideau Creek or Brassils Creek. It has been widely recognized that the Regional Storm on the Rideau River itself would be a snowmelt event, but the relative significance of the responses of small watersheds to these two quite distinct types of flooding events is subject to considerable question.

In Appendix 1 dealing with the Hydrologic Methodology employed in the study, the assumptions and results of our investigations have been documented in some detail.

The basic findings of the hydrologic analyses are as follows:

a) It has been shown that for all of the tributaries studied, the 1:100 year flood (the Regional Flood) can be expected to be generated by a spring time event involving snowmelt with or without coincident rainfall. This has been demonstrated by comparing the results of regional frequency analysis and the results of hydrologic modelling on representative small tributary drainage basins.

b) The results of the frequency analysis may be used in backwater modelling to compute flood elevations and they have been found to be compatible with the findings of previous hydrologic studies of the Rideau River.

c) The design flows used in backwater modelling are as listed in Table A1-2, in Appendix 1.

3.0 FLOOD AND FILL LINE MAPPING

3.1 General

The flood plain is that portion of a river valley beside a river channel which is inundated during periods of high water. The history of a flood prone river shows a recurring pattern of overflow in the flood plain area.

Mapping a flood plain involves delineating land areas which would be covered by flood waters. The extent of flooding depends on the magnitude of the flood and, in this portion of Ontario, the Ministry of Natural Resources has recommended that the area flooded on an average of once in 100 years (or an area which has a one percent probability of being flooded in any given year) be determined for regulatory purposes (the Regional Flood).

It is important to note that the above definition is based upon open water flooding conditions. Frequency projections for ice jam flooding are complicated by the fact that they may occur at various locations at any time given the right meteorological and hydraulic conditions.

3.2 Determination of Flood Water Levels

Water surface elevations were calculated using conventional open channel hydraulic methods. Elevations were assigned to a total of 169 cross-sections appropriately along each of the Rideau River tributaries. In addition, all 28 bridges in these tributaries were taken into consideration. To facilitate the computation, a computer backwater model, HEC-2 (U.S. Army Corps of Engineers, 1973), was used. The model, which employs the Standard Method of computation, calculates water surface elevations on the basis of representative stream valley cross-sections, vegetal growth within the valley, the slope of the stream bed and man-made constrictions. The valley cross-sections were primarily obtained from mapping by aerial surveys flown for flood plain mapping purposes. Detailed field work was carried out in the summer of 1980 to supplement this data and provide details of the bridges, weirs, and hydraulic roughness of the stream.

In the procedure of the Standard Step Method, backwater analyses are started from an initial location where the water surface can be readily determined. In this case, the navigational water levels as published by the Canadian Hydrographic Service were used as a starting point for each of the tributaries. Table 1.0 indicates the starting water surface elevation used for each of the tributaries.

In addition, Column 3 of Table 1.0 indicates the cross-section up to which the Rideau River 1/100 year floodline is greater than the floodline computed on the individual tributaries. Up to these noted cross-sections the plotted floodline was according to the 1/100 year Rideau River water level noted in Column 2. Note that Cranberry Creek is entirely controlled by the Rideau River flood elevations.

Water surface profiles were calculated for the Regional Storm Event (1/100 years), compared with the 1/100 year Rideau River level, and the greater of the two water levels were plotted on the orthophoto base mapping provided by Kenting Earth Sciences Ltd. Further details of the model and the computations may be found in Appendix 2.

<u>Otter Creek</u>

This creek has an extremely wide, flat floodplain, and as a result the HEC-2 model has a number of extended cross-sections. At these sections, the end stations are assumed to act as a "wall" not permitting water to flow any further out into the floodplain. This approximation is quite adequate since inspite of the restriction of the water, levels only rise 0.1 of a foot or less.

Brassil's Creek

Brassil's Creek, an extremely steep water course in certain reaches is subject to critical and super-critical flow. Figure 1.0 shows a profile analysis of the creek, indicating the various hydraulic control points and the likely flow regime in the different reaches. As a result, the HEC-2 modelling was done on an individual reach basis. Output from both the sub-critical and super-critical HEC-2 runs were used in the appropriate reaches to estimate the design water levels.

3.3 Fill Lines

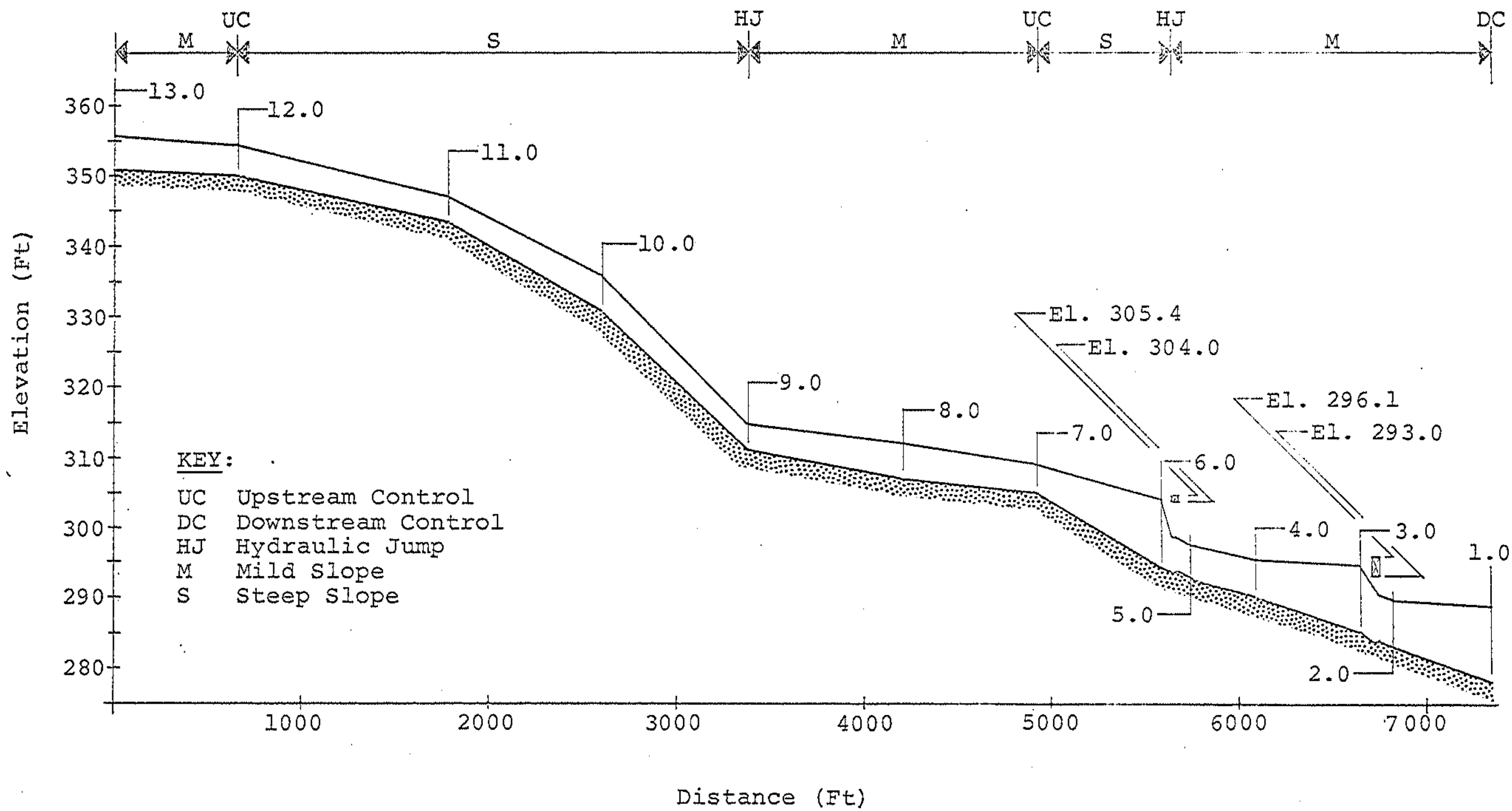
Fill and construction limits, as presented on the accompanying photomosaic drawings identify areas in which special consideration is required before proceeding with development of the dumping of fill. Slope erosion and flood susceptibility were adopted as the prime environmental hazards which preclude unregulated development; subsequently, the fill lines were positioned by stereoscopic interpretation of aerial photographs and verified during the field suvey. The criteria, upon which the fill and construction lines were established, are given under the following hazard classifications:

a) Slope Erosion

In those instances in which the river valley walls exhibit a suceptibility to erosion and potential instability, the fill lines are located at the limit of potential erosion.

b) Flood Susceptibility

For those reaches in which erosion potential is not a prime factor, the reservation of sufficient open space to permit the passage of the 100 year flood is considered a reasonable objective. In order to prevent the inundation of developed areas by flood waters, the fill lines in these areas are positioned at a distance of fifty feet from the 100 year floodline.



HYDRAULIC ANALYSIS OF BRASSIL'S CREEK

FIGURE 1.0

TABLE 1.0

BOUNDARY CONDITIONS FOR COMPUTATION AND

PLOTTING OF FLOODLINES ON THE RIDEAU RIVER TRIBUTARIES

	Starting Water Surface Elevations (Ft., GSC)	100 Yr. Elev. At The Rideau (Ft., GSC)	Rideau R. Controls Up to X-Sect
Cranberry Creek	280.5	286.9	All
MacDermott Drain	280.5	287.0	12.0
Kemptville Creek	280.5	287.5	19.0
Murphy Drain	280.5	287.6	12.0
Brassil's Creek	280.5	288.6	2.0
Rideau Creek	291.5	293.1	4.0
Barber's Creek	337.5	339.5	btw 1.0 & 2.0
Rosedale Creek	339.5	342.1	2.0
Otter Creek	348.8	350.3	btw 1.0 & 2.0

TABLE 2.0

REGIONAL DESIGN FLOOD ELEVATIONS ALONG BRASSIL'S CREEK

		Water Level Plotted Based On:			
Hydraulic Slope	X-Section	Rideau R. 1/100 Yr.	Sub-Critical Flow	Super Critical Flow	Critical Flow
	1.0	288.6			
	2.0		289.7		
	2.1		290.0		
	2.2		292.2		
Mild	3.0		294.5		
	4.0		295.5		
	5.0		297.1		
	5.1		298.6		
	5.2		298.6		
Steep	6.0		[304.0]	← Hydraulic Jump	297.2
	7.0				309.2
Mild	8.0		312.2		
	9.0		[314.8]	← Hydraulic Jump	314.5
	10.0			336.2	
Steep	11.0			347.3	
	12.0				354.5
Mild	13.0		356.2		

[314.8] Plotted Water Surface Elevation

4.0 FLOOD HAZARD AREAS

Table 3.0 describes the flood conditions under the occurrence of the regional event for each of the mapped tributaries. They are listed in order of highest to lowest flood hazard to existing structures in the floodplain.

Out of the nine (9) creeks mapped, it would appear that the most significant flood hazard area is along Kemptville Creek. Numerous homes and cottages are infringing upong the floodplain and as a result, damages will be significant should a major flood occur. Murphy Drain and the Reeve Craig area also have several homes and farms infringing upon the floodplain.

The remaining tributaries listed have areas which would only restrict travel during floods. Damages should be negligible.

Brassil's Creek, Rideau Creek and Hutton Creek are not listed on Table 3.0 because no houses or other structures are located in the floodplains of these tributaries.

TABLE 3.0

FLOOD HAZARD AREAS

Reference! Year!

Tributary	Description of Flood Conditions Under The Regional Event
Kemptville Creek	. 39 rural residential houses flooded outside of Kemptville . 10 residential houses flooded in Kemptville . 2 Major bridges overtopped . 450 m of major roadway flooded (RR #19).
Murphy Drain	. 12 rural residential houses flooded . 5 rural residential homes access restricted . 6 farming operations partially flooded.
Reeve Craig Area	. 6 farming operations partially flooded . 2400 m of major roadway flooded (RR #13) (in addition to previous study).
Rosedale Creek	. 150 m of major roadway flooded (Hwy #43).
Barber's Creek	. 2 farm out buildings flooded . 1 farming operation access restricted.
Cranberry Creek	. 600 m of major roadway flooded (RR #13).
MacDermott Creek	. 200 m of major roadway flooded (RR #19).

APPENDIX 1

HYDROLOGIC METHODOLOGY

A1.0 HYDROLOGIC METHODOLOGY

In order to compute water surface elevations on tributaries of the Rideau River, it was first necessary to arrive at estimates of flow on each tributary for the required events.

The tributaries which were included in this supplementary study are listed in Table A1-1.

The following paragraphs describe the rational which was applied in arriving at design flow estimates.

A1.1 General Description of Tributary Watersheds

Apart from Kemptville Creek, with a drainage area of 177 square miles (453 km^2), the tributaries of interest vary in drainage area from 4 to 51 square miles (10 to 130 km^2). Generally, the watersheds are relatively flat and significant portions of them are taken up in bogs and marshes due to poor grades. Table A1-1 summarizes some of the basic hydrologic characteristics of the watersheds.

In the case of Murphy Drain and McDermott Drain, the watercourse are little more than outlet ditches for agricultural drainage systems. The remaining watercourses rise in poorly drained marsh areas and meander slowly toward the Rideau.

Water problems identified on these creeks by the Rideau Valley Conservation Report (1968) are more related to low flows than to flooding. In the areas mapped, development adjacent to the watercourses is nearly non-existent, except for within the Town of Kemptville.

A1.2 Streamflow Records

Streamflow records have been collected by the Water Survey of Canada on Kemptville Creek, near Kemptville since 1970. No streamflow gauges are maintained on the other watercourses.

Outside the Rideau River watershed, but in the region, streamflow records are available on several watercourses within the South Nation River Baisin, and records are kept on the Jock River a downstream tributary of the Rideau.

A1.3 Hydrologic Methodology

There are two basic types of runoff events which may give rise to high water and flooding on any given watercourse. Flooding may follow periods of heavy snowmelt with or without coincident rainfall, the high volume of water on the drainage area being the predominant factor. Alternatively, flooding may follow heavy rainstorm events in which the intensity of rainfall and its volume combine to exceed the watercourse's capacity to contain the ensuing runoff.

In the Rideau Valley it is generally agreed that the Rideau River itself and its larger tributaries (the Jock River, Kemptville Creek) the Regional (1:100 year) flood would be generated by a snowmelt/rainfall event. Historically, all annual peak flows on the gauged watercourses have occurred in the months of February, March or April, supporting this conclusion.

On the smaller watersheds it is possible, however, that summer rainstorm events (higher intensity, shorter duration) of relatively infrequent occurence might generate higher runoff rates than equally infrequent snowmelt events.

TABLE A1-1

Watercourse	Area (mi^2)	Area (km^2)	Length (mi)	Length (km)	Slope (Ft. Per Mile)
Cranberry Creek	17.6	45	9.4	15	4.2
McDermott Drain	4.0	10	2.8	4.5	7.1
Kemptville Creek	177	453	40	64	2.5
Murphy Drain	17.2	44	6.3	10	10.3
Brassils Creek	33.7	86	8.1	13	98.0
Rideau Creek	19.9	51	5.6	9	9.8
Barbers Creek	16.1	41	7.5	12	6.0
Rosedale Creek	22.8	58	6.6	10.5	12.9
Otter Creek	50.9	131	10.6	17	5.5

Hence, it was required in this study to compare the responses of the smaller watersheds to these two types of streamflow generating events, in terms of runoff rates which might be expected at given frequencies.

A1.3.1 Frequency Analysis of Streamflow Data

Estimates of the 1:100 year flow associated with snowmelt events and flows for lower return periods have been developed using a regional flood frequency analysis incorporating the streamflow records of five regional gauges as follows:

Bear Brook near Bourget, drainage area = 170 mi^2
31 years of record

Castor River at Russell, drainage area = 167 mi^2
31 years of record

Jock River near Richmond, drainage area = 216 mi^2
9 years of record

Kemptville Creek near Kemptville, drainage area = 158 mi^2
9 years of record

South Nation River at Spencerville, drainage area = 95 mi^2
31 years of record

Initially, the records of the Kemptville Creek and Jock River stations were extended by correlation; the former with the record of the South Nation River and the latter with the record of the Castor River, based on the respective proximities of one watershed to the others.

Sangal and Kallio[1] have placed all of these watersheds in one Flood Frequency Region (St. Lawrence Lowlands) of similar hydrologic characteristics, based on physiographic characteristics, the natural of surficial soils, climatic conditions and land use. Hence it is considered appropriate to use these five stations nearest the Rideau watershed in the regional analysis.

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[1] Magnitude and Frequency of Floods in Southern Ontario
Sangal and Kallio
Technical Services Bulletin No. 99, Inland Waters Directorate
Fisheries and Environment Canada

The extended data series were generalized by dividing each annual maximum daily flow by the mean annual flood computed for the station over the 31 years of record.

A single data series consisting of the median value of ratios of maximum daily flow to mean annual flood from the five stations was then subjected to a frequency analysis using the Log-Pearson Type III Distribution, with the result that the ratios of daily discharge to mean annual flood for various return periods, are as follows for the given flood frequency region:

Return Period	Mean Daily Discharge Mean Annual Flood Ratio
5 years	1.24
10 years	1.48
25 years	1.79
50 years	2.04
100 years	2.31

To determin design flows for the various ungauged watersheds discharging to the Rideau River, the above ratios are multiplied by the estimated mean annual flood for each given watershed.

The mean annual flood for such watersheds has been computed using the equation:

$$MAF = 65.7\ (A)^{0.761}$$

where MAF is the mean annual flood
CFS in a watershed
A Drainage agrea (Square Miles)

This equation was developed by Sangal and Kallio for the flood frequency region by plotting the mean of flood series versus drainage area, weighting more heavily the data based on 10 or more years of records and fitting to the plot an expression of the appropriate form.

TABLE A1-2

DESIGN FLOW FROM FREQUENCY ANALYSIS

Watercourse	Area (mi^2)	Mean Annual Flood (CFS)	Design Flow (CFS) For Given Return Period 5 Yr.	10 Yr.	25 Yr.	50 Yr.	100 Yr.
Cranberry Creek	17.6	580	790	940	1140	1300	1470
McDermott Drain	4.0	190	260	310	370	430	480
Kemptville Creek	177	3370	4600	5490	6640	7560	8560
Murphy Drain	17.2	570	780	930	1120	1280	1450
Brassils Creek	33.7	960	1310	1560	1890	2150	2440
Rideau Creek	19.9	640	870	1040	1260	1440	1630
Barbers Creek	16.1	540	870	880	1060	1210	1370
Rosedale Creek	22.8	710	970	1160	1400	1590	1800
Otter Creek	50.9	1310	1790	2130	2580	2940	3330

Finally maximum instantaneous discharge is estimated by multiplying the daily discharge by a factor of 1.10 which is the overall average value of the ratio of peak instantaneous flow to maximum daily discharge as recorded at the four streamflow gauges recording instantaneous flow.

Table A1-2, summarizes the design flows computed using the analysis method described above. The flows are as estimated at the watercourses' confluences with the Rideau River.

A1.3.2 Hydrologic Modelling of Small Watersheds

In order to obtain estimates of the runoff rates which would be associated with summer rainstorm events it was intended first to prepare an hydrologic model of Kemptville Creek, calibrated by adjusting watershed parameters to achieve agreement between computed and recorded flows for known rainstorm events. Having done this, hydrologic models would be prepared for two small drainage basins using consistent watershed parameters. Computed flows for the 1:100 year rainstorm would them be compared with the 1:100 year flow from frequency analysis methods to determine which event (snowmelt or rainstorm) is critical at the 100 year return period.

Attempts to calibrate the HYMO model for three selected events on Kemptville Creek, for which water level records and precipitation records were available yielded poor results. For relatively minor events the uncalibrated HYMO model appeared to underestimate total runoff volume and overestimate peak flow.

Continued attempts to bring HYMO results more in line with recorded flows pointed out the following deficiencies in the modelling, which could add to error in the estimates:

a) Hydrograph attenuation at the Oxford Mills Dam, upstream of the Kemptville Creek gauge.

b) Variability of rainfall throughout the watershed. The precipitation records of two or three rain gauges should be used in the analysis, instead of only the Kemptville station.

c) Antecedant moisture conditions for real events may not be accurately represented by conditions I, II, or III (standard classes used).

d) In-channel flow routing was modelled without the benefit of accurate channel cross-sections.

Since it had been agreed that the regional flood on Kemptville Creek would be associated with a snowmelt event (estimated by frequency analysis), the purpose of the exercise was essentially to compare rainstorm and snowmelt events with respect to the associated flows in the small drainage basins. It was considered beyond the scope of the project to continue the calibration effort for the Kemptville Creek by accounting for all of the above mentioned factors in the HYMO model. Calibration results would have been used toward producing more representative hydrologic models of the small watersheds. However, the hydrologic parameters to be better defined through calibration characteristically vary with the size of the drainage basin; and the magnitude of the 'storm' hence, because Kemptville Creek is much larger than the others, the accuracy of parameters determined through calibration might not be any greater than that of parameters determined by "text-book" methods as specified for the uncalibrated model.

Therefore, attempts to calibrate the Kemptville Creek model were abandoned and a comparison was made between flows from the frequency analysis and flows as determined using uncalibrated HYMO models of two representative small drainage basins - Rideau Creek and Barbers Creek.

Soil complex numbers were determined by referring to the Ontario Soil Surveys and Generalized Land Use (agricultural, forest, wetlands, etc.) as presented in the National Topographic Series. Complex numbers were assigned to hydrologic units considering Hydrologic Soil Group and Land Use.

The application of the empirical "Williams Equations" for time to peak (tp) and recession constant values (k) led to low values of the resulting watershed constant (B), as compared with typical values applied in previous hydrologic studies undertaken in the Eastern Region.

To resolve the matter of the B value, several runs were performed to estimate peak flows for varying B values and for antecedant moisture conditions (AMC) II and III (average and saturated conditions, respectively). The results of the HYMO runs are summarized in Table A1-3, while Table A1-4 contains the basic input data to the HYMO models for Barbers Creek and Rideau Creek.

A1.3.3 Conclusions

On comparing the results of frequency analysis with those of hydrologic modelling for Rideau and Barbers Creeks, it was found that for each case where antecedant moisture condition II was assumed, the hydrologic modelling of the 1:100 year storm produced a peak flow considerably less than the 1:100 year instantaneous peak flow as computed by frequency analysis.

In previous studies it has been deemed appropriate when modelling the response of a basin the 1:100 year storm to assume AMC II as opposed to AMC III. This assumption is believed to be valid since AMC III implies saturated soil moisture conditions at the onset of the rainfall which would be very unlikely to occur in conjunction with as rare an event as the 1:100 year rainstorm.

TABLE A1-3

RIDEAU RIVER TRIBUTARIES HYDROLOGY

COMPARISON OF FLOWS (IN CFS) FOR VARIOUS COMBINATIONS OF ANTECEDANT MOSITURE CONDITION AND VALUE OF "B-PARAMETER"

RIDEAU CREEK: Area = 19.93 mi^2
Modelled as 3 sub-basins
Weighted CN = 59 (AMC II)
77 (AMC III)

Peak Flow During 1:100 Year Storm

B-Value (Mean)	AMC II CN' = 59	AMC III CN' = 77	
211		1620	
306	704	2072	1:100 Year Flow From Freq. Anal. = 1630 CFS
421	915	2670	

BARBER CREEK: Area = 15.44 mi^2
Modelled as 1 basin
CN = 64 (AMC II)
84 (AMC III)

Peak Flow During 1:00 Year Storm

B-Value (Mean)	AMC II CN' = 64	AMC III CN" = 84	
215		1010	
304	500	1406	1:100 Year Flow From Freq. Anal. = 1370 CFS
412	685	1921	

TABLE A1-4

PART A

PERCENTAGE OF DRAINAGE AREA IN GIVEN SOIL/LAND USE COMPLEX

RIDEAU CREEK:

Soil Group	A			AB			B			BC			C			CD			D		
Land Use	F	A	W	F	A	W	F	A	W	F	A	W	F	A	W	F	A	W	F	A	W
Sub Area 1	21	28					4	6		15	8	17	1								
Sub Area 2	35	36			4					7		3	3	12							
Sub Area 3	31	55											3	11							

BARBERS CREEK:

Soil Group	A			AB			B			BC			C			CD			D		
Land Use	F	A	W	F	A	W	F	A	W	F	A	W	F	A	W	F	A	W	F	A	W
Watershed	1						16	27	13	5	26	4	3	5							

Land Use Designations

F - Forest
A - Agricultural
W - Wetlands

PART B

ASSUMED CURVE NUMBERS FOR GIVEN COMBINATIONS

HYDROLOGIC SOIL GROUP AND LAND USE

	HYDROLOGIC SOIL GROUP (AMC II)						
Land Use	A	AB	B	BC	C	CD	D
Forested	50	54	58	65	71		77
Agriculture	59.1	62.4	65.2	71.1	76.3	-	-
Wetlands	-	-	54	58	-	-	-
Urban	54	62	70	75	-	-	-

TABLE A1-4 (Continued)

PART C

CURVE NUMBERS FOR RIDEAU AND BARBERS CREEKS

RIDEAU CREEK:

Sub-Area	Area (mi^2)	CN (AMC II)	CN (AMC III)
1	12.7	59.2	77.2
2	5.0	58.8	76.8
3	2.23	58.5	76.5

BARBERS CREEK:

1	15.44	64	84

The results of the calculations lead to the conclusion that peak flows generated by the 1:100 year storm on the small watersheds with average antecedent moisture, are significantly less than peak flows on the same watersheds generated by snowmelt or snowmelt with rainfall, with a return frequency of 100 years.

Hence, it has been shown that the estimates of peak flows on the tributaries as determined by frequency analysis are appropriate for use in the backwater computation phase of the project.

As part of floodline mapping of Kemptville Creek completed in 1972 (James F. MacLaren Limited) a regional frequency analysis was performed using records from the Bear Brook, Castor River and South Nation River gauges. The 1:100 year daily flow was estimated to be 7,800 CFS at the Town of Kemptville, and the corresponding instantaneous peak flow was 8,600 CFS (compared to 8,560 CFS at the mouth, reported in Table A1-2).

The 1976 report on the Rideau River concluded that the peak flows on the Rideau River and Kemptville Creek coincide with respect to time and a peak flow at the confluence of the two watercourses of 8,600 CFS was adopted.

The results of the current frequency analysis which incorporates streamflow records up to and including 1978 are quite compatible with earlier conclusions drawn with respect to Kemptville Creek, and suggest that the flow estimates prepared for the Rideau River tributaries may be used for the purposes of computing water surface elevations and completing the flood and fill line mapping.

A1.3.4 Distribution of Design Flows

On all creeks except Kemptville Creek only the first mile (approximately) upstream from the Rideau has been mapped. Unless the tributaries "branch" within this first mile one constant design flow throughout the reach has been used for computing 1:100 year water levels.

In the case of branching tributaries and for Kemptville Creek design flows have been distributed along the watercourse based on drainage area and by applying the equations adopted for the mean annual flood with appropriate peaking factors.

APPENDIX 2

HYDRAULIC METHODOLOGY

A2.0 HYDRAULIC METHODOLOGY

The HEC-2 computer program developed by the U.S. Army Corps of Engineers[1] was used to compute backwater profiles. Channel cross-sections and associated hydraulic parameters were defined at frequent locations along the floodplain. The locations were chosen where changes occur in slope, cross-sectional area, or channel roughness and at bridges and culverts. The program applies Bernoulli's Theorem for the total flow energy at each cross-section and Mannings formula for the friction head loss between cross-sections to calculate water levels. Other losses such as channel contraction and expansion, culvert losses and bridge losses can also be accounted for by the program.

The Bernoulli equation takes the following form:

$$\frac{P}{} + z + \frac{V^2}{2g} = H$$

Where $\frac{P}{}$ = pressure head

$\frac{V^2}{2g}$ = velocity head

z = height above datum

H = total energy head above the datum

For natural channels there is invariably energy dissipation due to flow resistance. The total energy head, H, therefore, decreases in the direction of flow.

The friction loss between section is computed from the Manning equation:

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[1] Corps of Engineers, Computer Program, 723-X6-L202A, "HEC-2 Water Surface Profiles", Users Manual 1976.

$$V = \frac{1.49\ R^{2/3}\ S^{1/2}}{n}$$

Where S_f = friction slope

R = hydraulic radius (feet)

n = Manning's resistance coefficient

V = velocity (feet per second)

Such factors as vegetation and channel roughness were evaluated from field inspection and from analysis of the recent aerial photography and have been represented in the flood stage calculations by Manning's n.

Expansion or contraction of flow due to changes in the channel cross-section is a common cause of energy losses within a reach. Therefore, loss coefficients were specified and used to compute the transition losses as a function of the absolute difference in velocity heads between the sections. Bridge and culvert losses also took into account expansion and contraction losses as well as the head losses through the structure itself.

The HEC-2 backwater calculation through culverts utilizes a "special bridge" routine which makes use of the orifice flow equation:

$$Q = A \left(\frac{2gh}{K}\right)^{1/2}$$

Where H = difference between the energy gradient elevation upstream and tailwater elevation downstream (ft.)

A = area of the orifice (ft.)

g = gravitational acceleration (ft/sec^2)

Q = total orifice flow (cfs)

K = total loss coefficient for the structure including entrance, exit, bend and friction losses

$$K = K_e + K_{ex} + K_b + K_f$$

Where K_e = typical entrance loss coefficient is 0.1

$K_{ex} = 1.0$

$$K_f = \frac{29.1\ n^2 L}{R^{4/3}}$$

n = Manning's friction factor
= 0.014 concrete pipe
= 0.021 corrugated metal pipe

R = hydraulic radius (ft.)

L = culvert length (ft.)

K_b = the bend loss coefficient was estimated as a function of the angle of deflection in the culvert.

Weir flow, representing a discharge over a waterway crossing, is computed by the weir equation:

$$Q = C\ L\ H^{3/2}$$

Where C = coefficient of discharge

L = effective length of the weir (ft.)

H = difference between the energy grade line elevation and the weir crest elevation (ft)

Q = total flow over the weir (cfs)

Orifice flow or a combination of weir and orifice flow can be handled by the model.