Ottawa River Flood Plain Mapping

Report to

Mississippi Valley Conservation Authority
Rideau Valley Conservation Authority
Regional Municipality of Ottawa–Carleton
Ontario Ministry of Natural Resources
Environment Canada

December 1984

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December 11, 1984

Mississippi Valley Conservation Authority
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Lanark, Ontario
KOG 1KO

Attn: Mr. D. Hallet
General Manager

Re: Ottawa River Flood Plain Mapping - Final Report

Dear Sir:

We are pleased to submit herewith our Final Report on the Ottawa River Flood Plain Mapping through the Regional Municipality of Ottawa-Carleton.

The report summarizes the long statistical record of flow data on the Ottawa River and documents flows at select locations in the study reach for various return frequencies of floods up to the 100 year event. Wind and wave analysis for open reaches of the river are also documented.

Flood and Fill Line maps for the study reach from the West Carleton west boundary to the east boundary of Cumberland Township were prepared based on the 1 in 100 year return frequency flood. The 159 maps are in three (3) separate groups as follows:

- Mississippi Valley Conservation Authority — Sheets 1 to 87
- Rideau Valley Conservation Authority — Sheets 1 to 45
- Regional Municipality of Ottawa-Carleton — Sheets 1 to 27

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We wish to thank the Authority for their co-operation in the successful completion of this project.

Yours very truly,

MacLAREN PLANSEARCH INC.

H.W. Cornfield
Project Director

REGISTERED PROFESSIONAL ENGINEER
H. W. CORNFIELD
PROVINCE OF ONTARIO

HWC/cjb

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TABLE OF CONTENTS

TABLE OF CONTENTS (Cont'd.)

LIST OF FIGURES

LIST OF TABLES

LIST OF RELATED DOCUMENTS AND INFORMATION

1. Flood Risk Maps, Ottawa River, Mississippi Valley Conservation Authority -

   Sheet Index and 87 maps with a scale of 1:2000, contour interval 1 metre with 0.5 auxillary contours. The 1 in 100 year flood line and the fill line are shown.

2. Flood Rsk Maps, Ottawa River, Rideau Valley Conservation Authority -

   Sheet Index and 45 maps with a scale of 1:2000, contour interval 1 metre with 0.5 metre auxillary contours. The 1 in 100 year flood line and the fill line are shown.

3. Flood Risk Maps, Ottawa River, Regional Municipality of Ottawa-Carleton -

   Sheet index and 27 mnaps with a scale of 1:2000, contour interval 1 metre with 0.5 metre auxillary contours. The 1 in 100 year flood line and fill line are shown.

4. Cross Section Plots -

   Pen plotted cross sections of river sections used in the HEC-2 computations (1 copy).

5. Computer Output -

   Computer output from HEC-2. Input and output are also stored on magnetic tape by MacLaren Plansearch.

6. Ottawa River Photographs -

   Site photographs taken during the field survey.

SUMMARY

A flood plain mapping study of the Ottawa River within the boundaries of the Regional Municipality of Ottawa-Carleton has been completed. This is a first step in flood plain management and is designed to identify areas of high flood risk.

There is a history of flooding along the Ottawa River caused by high water levels and sometimes accentuated by waves. For example, this type of problem has lead the City of Ottawa to propose protection measures for the Britannia Bay area. Other areas where flooding has been a problem include Cumberland and Constance Bay. Proper delineation of the flood plain can lead to reduction in future flood losses through prevention of development in high risk areas. Further, by identifying the magnitude of existing flood risks it is possible to begin considering remedial measures such as diking or flood proofing to correct existing problems.

The responsibility for management of the flood plain lands within the study area is divided between (i) the Mississippi Valley Conservation Authority, (ii) the Rideau Valley Conservation Authority and (iii) the Regional Municipality of Ottawa-Carleton. The Ontario Ministry of Natural Resources and Environment Canada both support the three previously named government bodies in their flood plain management efforts. This project has been funded under the Canada-Ontario Flood Damage Reduction Program. The engineering services for this project were provided by MacLaren Plansearch Inc.

The basic steps taken in the study were as follows:

1) Background Data Collection - This included surveys, flow data, water level records, bridge drawings and maps.

2) Hydrology Investigations - This was primarily the assessment of flow records to obtain consistent data bases and the estimate of flow rates for various recurrence intervals.

3) Surveying and Map Preparation - Surveys were undertaken to provide ground control for air photos, to obtain additional information where mapping was not available and to check completed maps. Hydrographic surveys were carried out by Environment Canada for a portion of the river where no hydrographic charts were available. Maps with a scale of 1:2000 for the Ontario side of the river were prepared using digital mapping techniques by Northway-Gestalt Survey Corporation. Field checks by surveyors from MacLaren Plansearch indicated that the mapping met the specifications required for flood plain mapping.

4) Hydraulic Computations - The flood profiles along the river were computed using the HEC-2 computer program developed by the U.S. Army Corps of Engineers. This program is widely used and recognized for these types of computations. Profiles for the 2, 5, 10, 20, 50 and 100 year recurrence interval floods were estimated. Also, estimates of wave run-up have been prepared.

5) Documentation - The flood lines for the 1 in 100 year recurrence interval flood have been plotted on 1:2000 scale maps. Also shown is a fill line which is a regula-

tory setback line from the flood line. This report, together with the technical files and computer output contain the factual information upon which the conclusions have been founded.

Specific comments on the findings of our study are contained in the following paragraphs:

1) The flow records show that construction of dams on the Ottawa River has resulted in a reduction of peak flow rates of more frequent floods. However, the less frequent floods are large in relation to the storage in the reservoirs and thus the dams are essentially "run of the river" facilities during periods of high flow rate.

2) The flood flow rates at Chats Falls Dam were used for flood profile computations from Chaudiere Dam to the upstream limit of this study. Flood flow rates for Grenville/Carillon Dam were used from Chaudiere Dam to the downstream limit of the study.

3) Flood problems that are evident for the 1 in 100 year recurrence interval flood include the following:

    a) Cumberland Township, Lot 10, Concession 1 (see map sheets 18 of 27 and 19 of 27, Regional Municipality of Ottawa-Carleton). A large number of residences are subject to flooding in this area. Some of the homes at the edge of the flood plain could be protected by flood proofing. However, inundation in some areas could be over 2 meters which makes usual flood proofing measures impractical.

b) Petrie Islands - Structures on the Petrie Islands may be subject to flood depths of as much as 2 meters.

c) A few structures are located on the fringe of the flood plain at various other locations in the river reach which is under the jurisdiction of the Regional Municipality of Ottawa-Carleton.

d) Several structures on Upper Duck Island would be flooded.

e) Widespread flooding can be expected in the vicinity of Shirleys Bay and Britannia Bay. Flooding of approximately 1 metre depth would be experienced in some areas but waves could complicate the situation and accentuate damage. Also, flood waters could spill through the Britannia area and return to the Ottawa River in the vicinity of the filtration plant.

f) Another flood damage centre is the Constance Bay area. Numerous houses would be flooded under 1 in 100 year flood conditions. The depth of flooding would generally be less than 1.0 meter but waves could increase the damages.

g) Fitzroy Harbour and MacLarens Landing both contain residences which would be flooded.

h) Above Chats Falls Dam there are a number of residences or cottages that would be affected by flooding.

Flood proofing measures on a residence by residence basis could be implemented to minimize damages due to flooding along the reach of river from Britannia Bay to Chats Falls Dam. Diking does not appear to be an attractive alternative to control flood waters due to space limitations. Most of the dwellings affected by flooding are relatively close to the river.

Flooding along the Ottawa River from Britannia Bay to Chats Falls Dam could be reduced by modifying Deschenes Rapids so that high flow rates do not cause such high water levels. Lowering the flood levels at the top of Deschenes Rapids would lower flood levels all the way to Chats Falls Dam. Modifications to the rapids section would have to be done with care so that water levels during periods of low flow would not be reduced. This type of work would require detailed planning and careful construction.

Fill lines were based on the following general criteria:

1) 30 m back from 1 in 100 year floodline or,

2) 30 m back from top of bank where top of bank is clearly defined or from the point where the slope of the bank is less than 10 per cent.

3) Follows fences or roads or is a straight line easily identified by ground features.

In some cases, these rules were not strictly adhered to for the following reasons:

1) Existing buildings were excluded as much as possible while still providing a margin of safety for future development.

2) The fill line may encompass environmentally sensitive areas.

1.0 INTRODUCTION

The Ottawa River is one of the largest rivers in Canada. On its course to the St. Lawrence River, it flows through the Regional Municipality of Ottawa-Carleton, a major urban centre. Municipalities within the Region affected by flooding of the Ottawa River include: Cumberland, Gloucester, Ottawa and Constance Bay. Although the flood damages in this reach have not been as significant as in other locations along the river, future development along the river cannot be assured of any margin of safety from flooding. Thus, floodline and fill line mapping to define flood hazard levels of the Ottawa River reach within the Regional Municipality of Ottawa-Carleton has been undertaken by the Mississippi Valley Conservation Authority.

MacLaren Plansearch Inc. was retained by the Mississippi Valley Conservation Authority to provide the required engineering. Mapping services were provided by Northway-Gestalt Survey Corporation Limited.

The determination of flood water elevations for the Ottawa River was undertaken in a systematic manner and included: (1) data and background information collection; (2) hydrologic calculations to define flows into and out of the study area; (3) hydraulic calculations based on the flows determined and the physical characteristics of the river; (4) base map production, and finally; (5) floodline and fill line mapping. Based on this information, flood plain management plans can be evaluated and the best course of action taken to mitigate flood damages.

This report documents the engineering methodology and reasoning behind the floodline and fill mapping, and concludes by evaluating existing flood damages and recommending the course of future flood management techniques along the Ottawa River through the Regional Municipality of Ottawa-Carleton.

2.0 HYDROLOGY

2.1 APPROACH

Due to the long period of available data, the most direct approach to establishing design flows for the Ottawa River has been determined to be a flood frequency analysis. The major data requirements for such an analysis are a series of peak flows from a long period of record. The assumption that is usually made is that these series of flows are independent, homogeneous and free from any trend.

Independence means that the one given flow of the sequence should not be in any way related to previous peak flows. For instance, dependence of flows from one year to the next may be shown by a high peak runoff from one year which may fill reservoirs along a river to the point where the following year, because the reservoirs are close to full, may cause another high peak flow. Thus, these two flows are dependent upon one another.

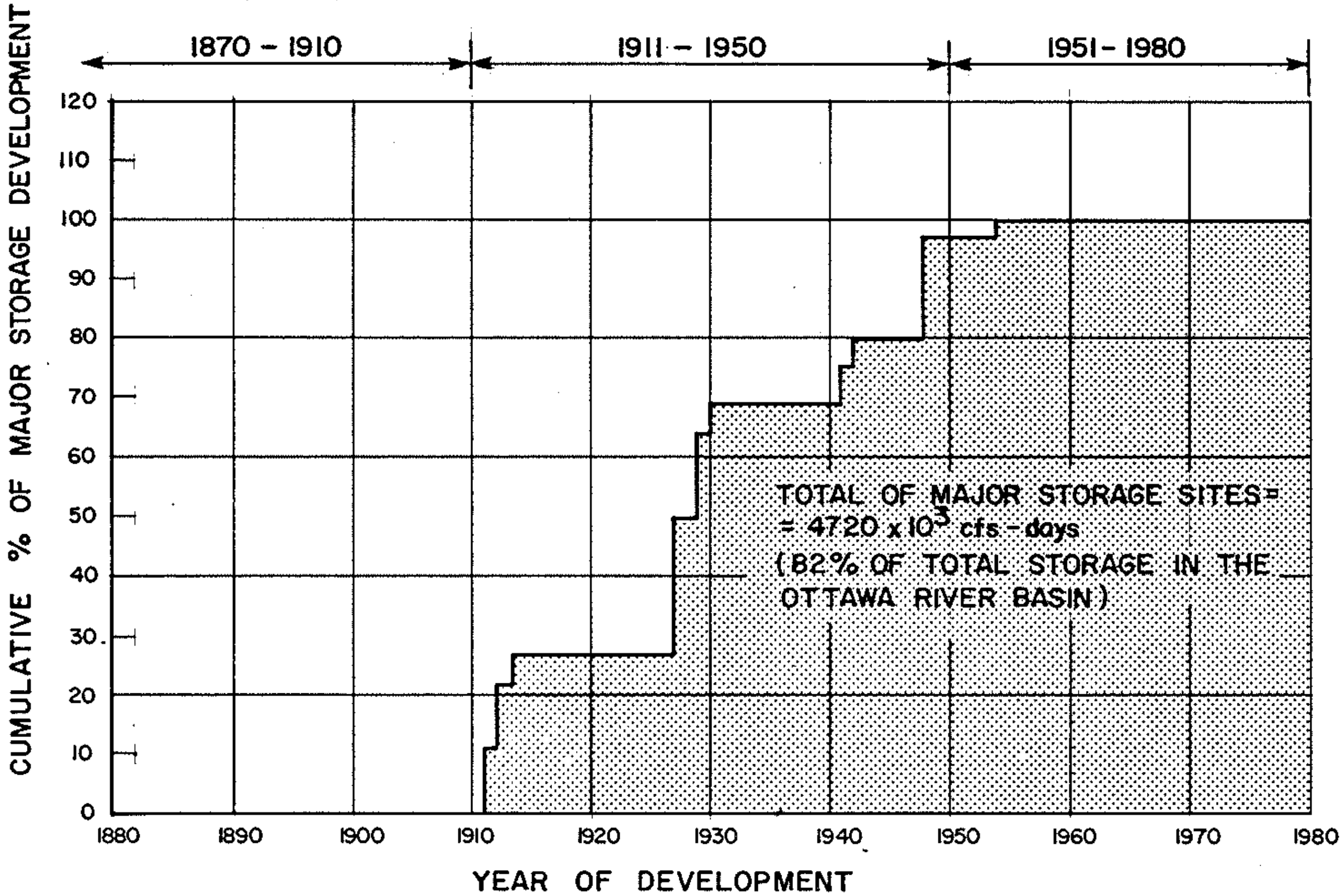
Although the Ottawa River is controlled by various dams, during the spring runoff months control by these dams is drastically reduced. In fact, the operation at even the major facilities becomes "run-of-the-river". As a result, dependence between peak flows from one year to the next is expected to be minimal. The assumption of independence and degree of dependence was tested by the Spearman Rank Serial Correlation Coefficient which was then statistically examined for significance. This test and its results are described further in Appendix A.

A homogenous sample implies that all data within the sample are collected under the same physical conditions in the watershed. If some more or less abrupt change occurred during the sampling period, then some differences could be expected between the statistics of the sub-samples before and after the change. For example, the construction of a reservoir, a forest fire or a landslide may cause a substantial change in the hydrologic response of the watershed. Homogenity of a sample was statistically tested by comparing means, standard deviations, skewness and kurtosis of sub-samples partitioned where it was suspected that a change may have occurred. A more detailed discussion of the statistical tests performed to determine homogenity is presented in Appendix A.

A trend is a linear or non-linear slow change in the statistical parameters of the series caused by a gradual change in the watershed. Urbanization or logging operations are examples of changes which may cause a trend in the peak flow series. The statistical significance of a trend was tested using the Spearman Rank Order Correlation coefficient. This test is discussed in detail in Appendix A.

For the Ottawa River, however, it is difficult to identify trends from shifts in peak flow rates. The types of changes in hydrologic regime are varied and the magnitude of an individual change is relatively small. As a result, their effects on the peak flows were considered on an integrated basis.

Figure 2.1 shows the historical development of major storage sites along the Ottawa River. The major storage sites represent 82% of the total basin storage to date. The remaining 18% of the storage sites are scattered throughout the water-



HISTORICAL DEVELOPMENT
OF MAJOR STORAGE SITES
ALONG THE OTTAWA RIVER

FIGURE 2.1

shed generally along the smaller, second and higher order tributaries. As a result, the control exerted by these structures during spring melt is inconsequential to flow in the main stream of the River.

Furthermore, the total major storage is approximately 25-35% of the average annual spring runoff and, as a result, the storage capacities are frequently exceeded.

As shown in Figure 2.1, three distinct periods exist in the implementation of the major Ottawa River storage facilities:

(1) 1870-1910 No major dams built.

(2) 1911-1950 97% of all major storage completed.

(3) 1951-1980 No significant further storage added.

Other changes in the watershed which may tend to confuse the effect of the construction of dams are urbanization and the logging of large tracts of land.

The extent of urbanization in the watershed has not been great relative to the size of the watershed. The trend towards urbanization began in the early 1900's and peaked in the middle 1960's. It has considerably tapered off since the early 1970's.

The logging industry was highly active in the area in the late 1800's and early 1900's. The industry has since come nearly to a standstill in the watershed. Regrowth has since established new forests in the watershed and the present effect is minimal.

The construction of dams is the predominant factor in reducing peak flows, while the effects of urbanization and logging operations would tend to increase peak flows, although it is expected that these increases would be very small.

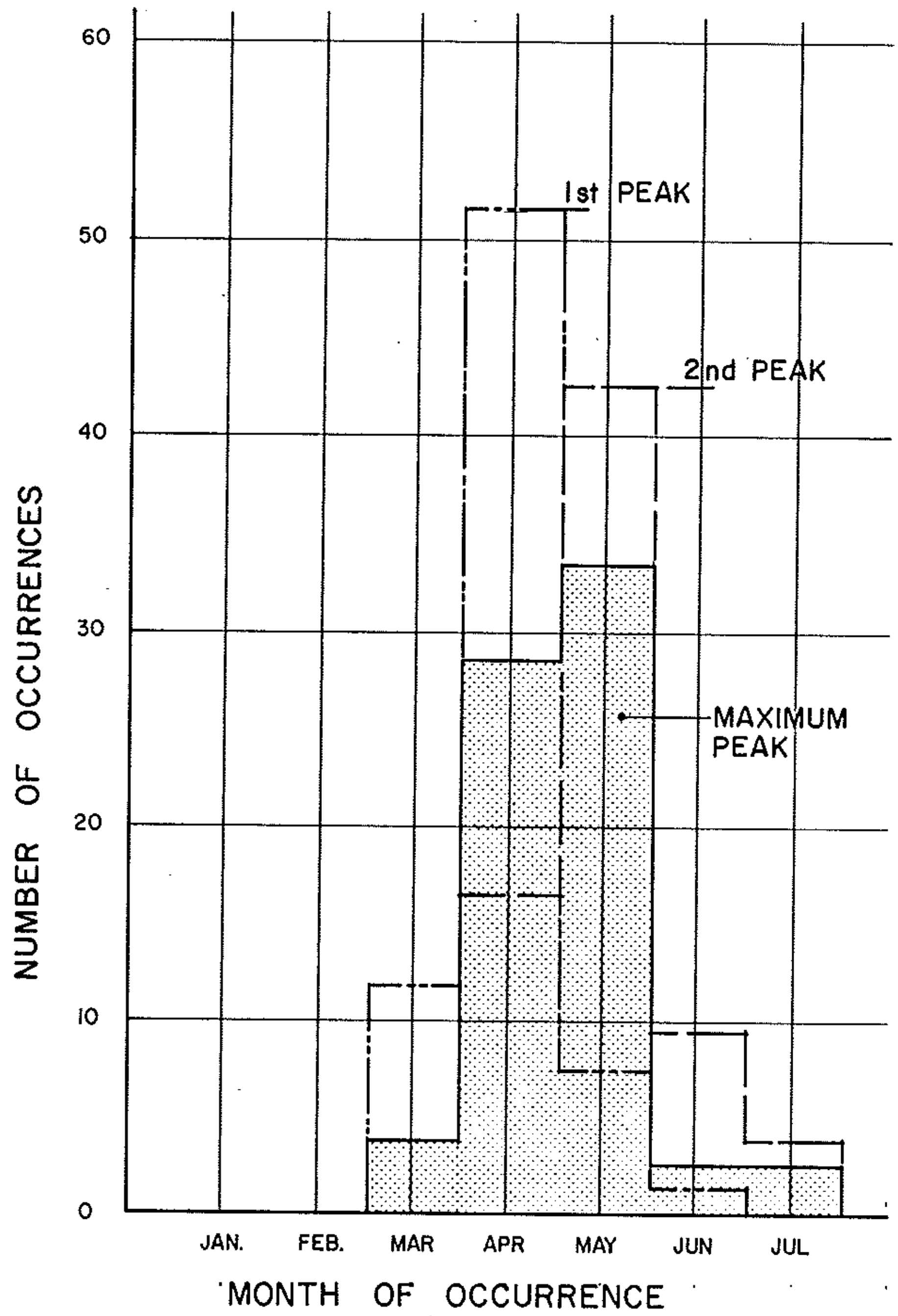
For this reason, the partitioning of the record was based primarily on the construction of dams.

2.2 DATA

Flow data, within the study reach as shown in Table 2.1, is available at three locations: (1) Grenville/Carillon Dam, (2) Britannia Bay, and (3) Chats Falls. The longest flow record available at Grenville/Carillon extends back to 1870. Since statistical analyses are highly dependent on the length of record, the most detailed analyses were done on Grenville data. Next in record length was Chats Falls, with flow data back to 1915, followed by Britannia with only 20 years of published flow data.

2.2.1 Grenville Data

For the reasons described in the previous section, the data was segmented into three periods: (1) 1870-1910, (2) 1911-1950, and (3) 1951-1980. In addition, the data was split by defining first and second spring peak of the year. Figure 2.2 shows a histogram of the data samples by the month of occurrence. The first peak of the year occurs generally in April while the second peak follows in May. The occurrence of the



HISTOGRAM OF PEAK FLOW DATA BY MONTH OF OCCURRENCE

FIGURE 2.2

TABLE 2.1

FLOW DATA AVAILABLE WITHIN THE STUDY REACH

(1) Grenville 1870 - present (drainage area 143,000 km^2)

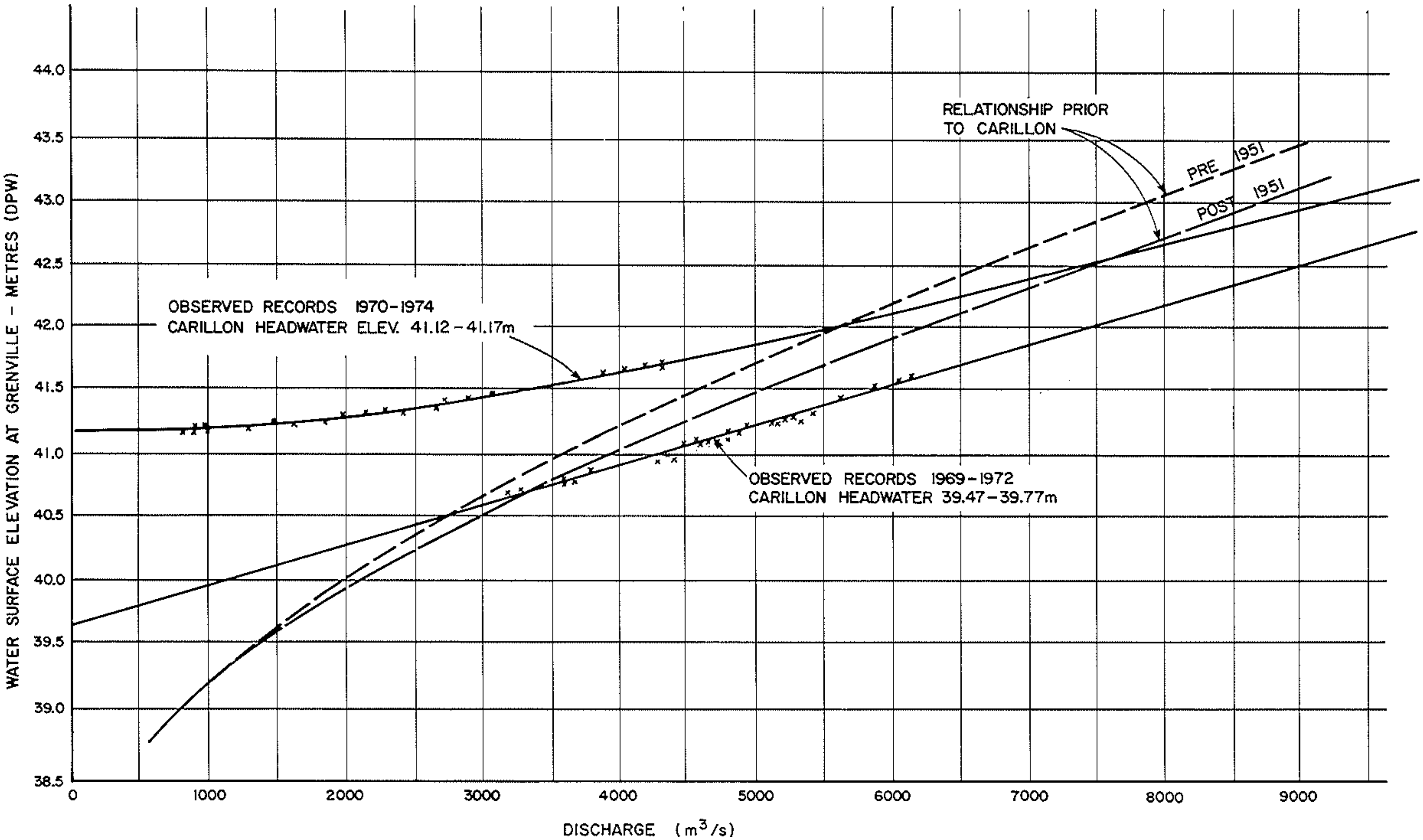
SOURCE	NOTES
Quebec Hydro	1960-present at Carillon Dam based on turbine rating plus spillage
Quebec Hydro	1951-1960 at Grenville based on rating curve No. 2 (See Figure 2.3)
Quebec Ministry of Natural Resources	1870-1950 at Grenville based on rating curve No. 1 (See Figure 2.3)

(2) Britannia Bay 1960 - present (drainage area 90,900 km^2)

Water Survey of Canada	1960-present based on rating curve published in 1978

(3) Chats Falls 1915 - present (drainage area 89,600 km^2)

Water Survey of Canada	1932-present from Ontario Hydro based on turbine rating plus spillage
	1915-1932 based on rating curve



OTTAWA RIVER
GRENVILLE
STAGE - DISCHARGE RELATIONSHIP
FIGURE 2.3

FILE A3

maximum annual peak is evenly divided between the first peak of the year and the second.

2.2.2 Chats Falls Data

Flow data at Chats Falls has been published since 1915. In 1930, the Chats Falls Dam was built and controlled for hydro-electric power generation. Both inflows and outflows for the facility were measured, and from an analysis of these records it may be stated that for the annual peak flow periods, there is no real difference between inflow and outflow, i.e. the dam operates as a "run-of-the-river" facility.

2.2.3 Britannia Bay Data

Although water level records date back to 1915, stage versus flow correlation was only begun in the middle 1960's. In 1978, a rating curve was developed and flow data was published back to 1960. The period of record is relatively short and flows at peak are not substantially different that those flows measured at Chats Falls.

As a result, the Britannia Bay record was dropped from the analysis as it was felt that no further information would be gained from the data.

2.3 RESULTS

2.3.1 Homogeneity of Grenville Data

Table 2.2 shows the results of the comparison of statistics for the various periods of data outlined earlier. A signifi-

TABLE 2.2

RESULTS OF HOMOGENEITY TESTS
OTTAWA RIVER AT GRENVILLE
ANNUAL PEAK MAXIMUM FLOWS

Statistical Comparison of Sub-Samples to
Determine if a Significant Difference Exists
between Periods of Record

Comparison of Sample A with		Sub-Sample A	
		1951-1980	1911-1950
SUB-SAMPLE B	1911 - 1950	$\bar{X}$, s^2, C_s, C_k) N/S	---
SUB-SAMPLE B	1870 - 1910	$\bar{X}$ S* S^2, C_s, C_k) N/S	$\bar{X}$ S* S^2, C_s, C_k) N/S

KEY:

$\bar{X}$ sample mean

S^2 sample standard deviation

C_s sample coefficient of skew

C_k sample coefficient of kurtosis

S* significant difference exists between sub-samples

N/S no significant difference exists between sub-samples cant difference in means was found between the annual maximum peak flows of the period 1870-1910 (6440 cms) when compared with the means of the periods 1951-1980 (5490 cms) and 1911-1950 (5340 cms). However, no significant difference was detected in the other statistical parameters. Thus, a significant downward shift in peak flow has occurred since 1870. It is postulated that this downward shift is caused by the construction of numerous dams along the river in the period 1911-1950, which would provide storage thereby reducing peak flows.

It is noted that no significant difference occurs between the means and other statistical parameters when the data for the period 1911-1950 is compared with the period 1951-1980. Thus, for statistical purposes, the data from 1911-1980 can be treated as a single homogeneous data set, keeping in mind that this period of data may be influenced by the regulation of dams. The period 1870-1910 can be considered a second set, which is free from regulation of dams along the river.

2.3.2 Independence and Trend of Grenville Data

The results of correlation tests for independence and trend are shown in Table 2.3. The results of the trend tests confirm that a reduction in peak flow is occurring. A significant downward trend has been found in the periods 1870-1950 and 1870-1980. The trend for the first peak of the year was compared with the second peak of the year. It was observed that the first peak has a fairly large trend component and, although it cannot be shown to be statistically significant, it is much larger than the trend observed for the second peak of the year.

TABLE 2.3

RESULTS OF INDEPENDENCE, TREND AND RANDOMNESS TESTS OTTAWA RIVER AT GRENVILLE

DATA	PERIOD	R^2 INDEPENDENCE	R^2 TREND	RANDOMNESS	REMARKS
Annual Peak Maximum	1951-1980	0.329 N/S	-0.118 N/S	NOT RANDOM	Sample too short for proper random test
Annual Peak Maximum	1911-1950	-0.304 N/S	0.033 N/S	RANDOM	
Annual Peak Maximum	1870-1910	-0.065 N/S	0.257 N/S	RANDOM	
Annual Peak Maximum	1870-1950	-0.024 N/S	0.353 S*	RANDOM	Significant trend is present: Q_p decreasing
Annual Peak Maximum	1911-1980	-0.099 N/S	-0.044 N/S	RANDOM	
Annual Peak Maximum	1870-1980	0.032 N/S	0.282 S*	RANDOM	Significant trend is present: Q_p decreasing
First Peak of the Year	1911-1980	0.0958 N/S	-0.270 N/S	RANDOM	Some trend probably due to fact reservoirs empty initially in spring
Second Peak of the Year	1911-1980	0.0173 N/S	-0.022 N/S	RANDOM	It would appear that trend is removed from sample if second peak is utilized
Second Peak of the Year	1870-1980	0.0279 N/S	-0.0126 N/S	RANDOM	

N/S - not statistically significant
S* - significant at 5% level

Although there may be some dependence of flows from one year to the next due to regulation of the river in the sub-samples 1951-1980 and 1911-1950, statistically this dependence has been shown to be negligible. All remaining samples show relatively small serial correlation coefficients and are also shown statistically to be independent. In particular, the results shown in Table 2.3 indicate that the period of record, 1911-1980, is statistically independent and has no significant trend component.

2.3.3 Flood Frequency Analysis at Grenville

Flood frequency analysis was done using the Environment Canada program FDRPFFA (revised version - September 1981). Table 2.4 shows the results of the flood frequency analyses performed on the various data periods. Sample statistics, shown in Table 2.5, and the graphical plots of return period versus flow (Appendix C) were used to evaluate the goodness-of-fit of the Gumbel, Log-Normal, 3 parameter Log-Normal and the Log-Pearson Type III distributions. Estimation of parameters for the distributions was done by the Maximum Likelihood Method. The best fit distributions for each period of data are shown by an asterisk beside the appropriate 1/100 year flood estimate. The goodness-of-fit varied between the 2 parameter and the 3 parameter Log Normal distributions, but in every case the Log-Pearson Type III provided adequate fit for the data, as well as very similar flood estimates as the other best-fit distribution (either 2 or 3 parameter Log Normal). Graphically the Log Pearson Type III distribution appeared to fit the data in each case examined. As a result, the Log Pearson Type III was chosen as the best overall distribution.

TABLE 2.4

RESULTS OF FLOOD FREQUENCY ANALYSIS OTTAWA RIVER AT GRENVILLE

	DATA	PERIOD	1/100 FLOOD ESTIMATE BY			
			GUMBEL (CMS)	LOGNORM (CMS)	3-LOG (CMS)	LOG PEARS (CMS)
1.	Annual Peak Maximum	1951-1980	9,880	9,460*	10,000	9,960*
2.	Annual Peak Maximum	1911-1950	10,400	9,990*	10,300	9,620*
3.	Annual Peak Maximum	1870-1910	12,200	11,300	10,300*	10,100
4.	Annual Peak Maximum	1870-1950	11,700	11,100	10,400*	9,810
5.	Annual Peak Maximum	1911-1980	10,200	9,750*	10,100	9,840*
6.	Annual Peak Maximum	1870-1980	11,000	10,600*	10,500*	10,000*
7.	First Peak of the Year	1911-1980	9,120	8,650	8,260*	8,110*
8.	Second Peak of the Year	1911-1980	10,100	9,840	9,630*	9,410*
9.	Second Peak of the Year	1870-1980	11,000	11,000	9,650*	9,140*

* best-fit distribution(s)

TABLE 2.5

COMPARISON OF SAMPLE STATISTICS OTTAWA RIVER AT GRENVILLE

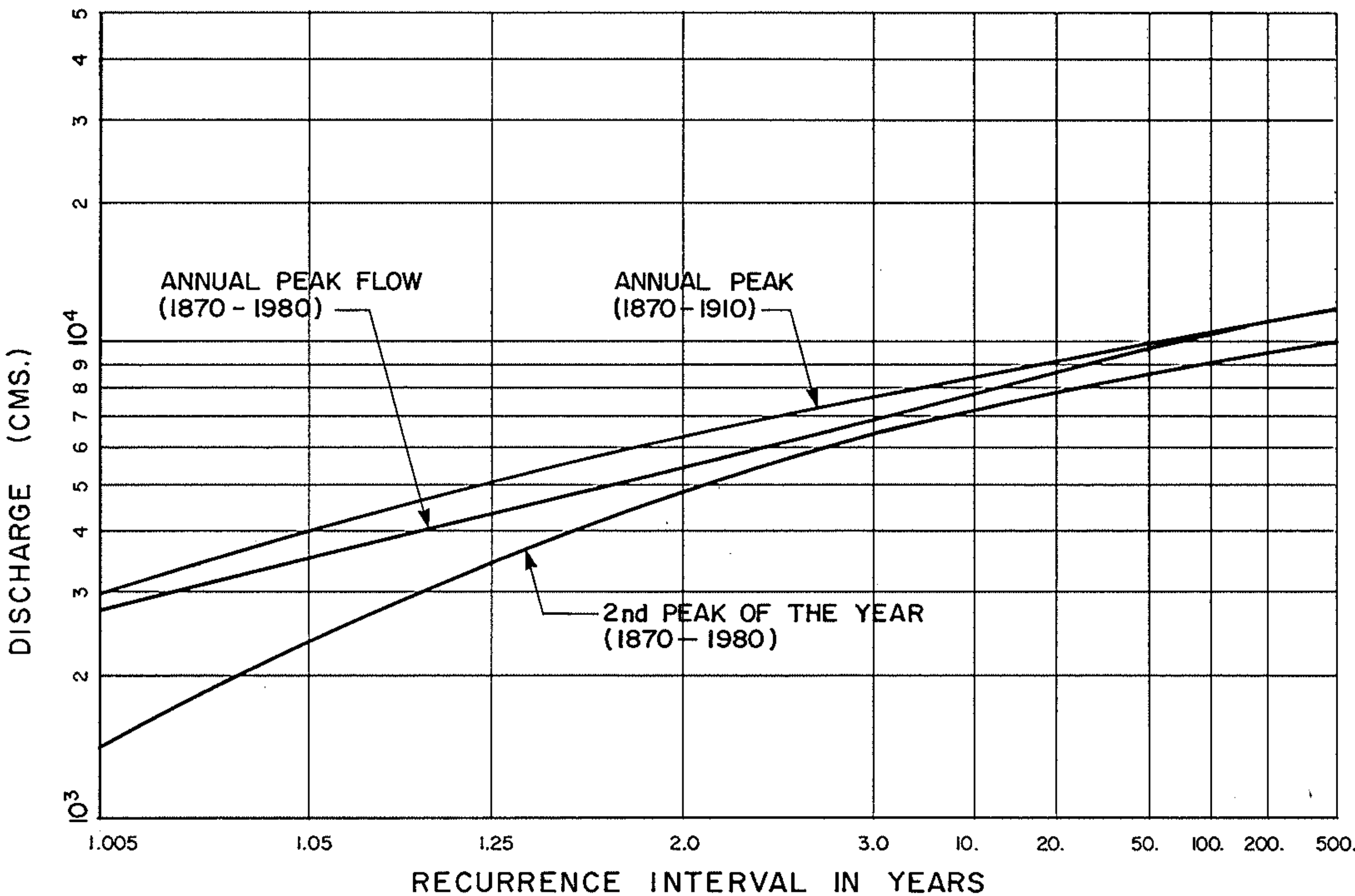
DATA	PERIOD	SAMPLE STATISTICS					
		MEAN (CMS)	STD.DEV.	C_s	C_k	LOG C_s (-A)*	LOG C_k (-A)*
Annual Peak Maximum	1951-1980	5486.	1411.	0.901	4.02	0.273 (-0.033)	3.24 (3.26)
Annual Peak Maximum	1911-1950	5343.	1516.	0.388	2.403	-0.035 (0.118)	2.25 (2.28)
Annual Peak Maximum	1870-1910	6441.	1578.	0.081	2.698	-0.373 (-0.012)	2.48 (2.58)
Annual Peak Maximum	1870-1950	5899.	1634.	0.219	2.387	-0.249 (-0.053)	2.25 (2.19)
Annual Peak Maximum	1911-1980	5404.	1463.	0.554	2.810	0.033 (-0.079)	2.45 (2.47)
Annual Peak Maximum	1870-1980	5779.	1562.	0.575	3.362	-0.112 (-0.073)	2.28 (2.26)
First Peak of the Year	1911-1980	4578.	1308.	0.521	3.306	0.237 (-0.023)	2.99 (2.89)
Second Peak of the Year	1911-1980	4960.	1553.	0.612	3.136	-0.126 (-0.037)	2.87 (2.79)
Second Peak of the Year	1870-1980	4974.	1707.	0.3511	2.651	-0.511 (-0.029)	3.15 (2.61)

* LOG C_s and C_k for 3-parameter lognormal distribution

2.3.4 Discussion of Results at Grenville

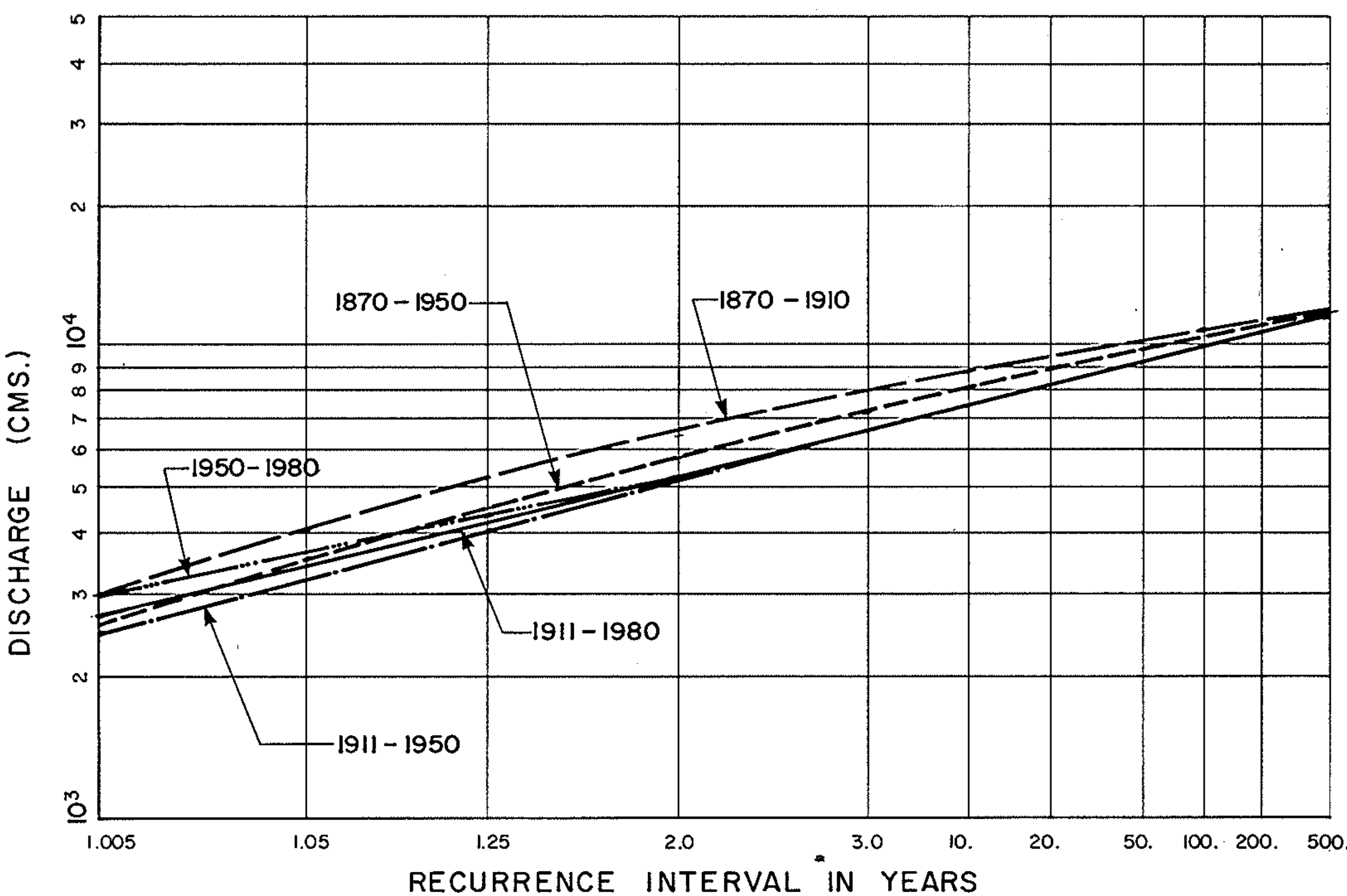
With reference to Figure 2.4 which shows the Log Pearson Type III Distribution Plot for the (a) Annual Peak 1870 to 1910, (b) Annual Peak 1870 to 1980, and (c) Second Peak of Year 1870 to 1980; the following observations were made. The two data sets which were found to be free of trend: annual peak (1870 to 1910) and second peak (1870 to 1980) appear to be 1500 cms apart no matter what the recurrence interval. The period 1870 to 1910 is measured under natural flow conditions, since none of the major storage reservoirs were in place during this period. The second peak of the year would also appear to be free of any trend. It is hypothosized that this is because reservoirs are near their capacity by the time the second peak of the year occurs and consequently the resultant peak is closer to the "run-of-the-river" situation. The separation of the first and second peaks was somewhat difficult due to the discernability of two separate hydrographs and occasionally the existence of only one significant peak in the record. As a result, interpretation of these results must be considered with care.

The flood estimates based on the Annual Peak 1870 to 1980 lies between the above two lines, as do the estimates from the annual peak 1870-1910, 1870-1950, 1911-1950, 1950-1980. These latter flood estimates are shown in Figure 2.5 with relation to the estimates using annual peak 1911-1980. As can be seen in Figure 2.5, the flood estimates vary considerably at the more frequent recurrence interval, however they still converge between the 1/100 year and 1/500 year return periods. This appears to be consistent with the explanation that the smaller



COMPARISON OF FLOOD FREQUENCY ANALYSES (A)
LOG PEARSON TYPE III DISTRIBUTION
PARAMETERS ESTIMATED BY MAXIMUM LIKELIHOOD

FIGURE 2.4



COMPARISON OF FLOOD FREQUENCY ANALYSES (B)
LOG PEARSON TYPE III DISTRIBUTION
PARAMETERS ESTIMATED BY MAXIMUM LIKELIHOOD

FIGURE 2.5

peak flows remain dependent on the degree of regulation while the greater flows cannot be controlled.

In addition to converging with each other at lower frequency recurrence interval, the estimates from the various periods of record also converge with the estimates based on annual peak flow 1870-1910; the period which is considered to be natural flow conditions. This is shown in Figure 2.4.

2.4 ANALYSIS OF CHATS FALLS RECORD

As with the Grenville data, the Chats Falls record was checked for independence, trend and homogeneity. For determining homogeneity, the data was partitioned at 1950.

The data was found to be statistically independent with the serial correlation coefficient of -0.17.

The calculated trend correlation coefficient of -0.025 was also found to be statistically insignificant.

The means, standard deviations and other statistical parameters between the two partitioned sub-samples were found not to be statistically different from one another.

As a result of these tests the entire sample of peak flows from 1915 to 1980 were considered to meet the assumptions discussed previously for flood frequency analysis. It must be remembered that the flows during this period are controlled.

Flood frequency analysis was completed for the Chats Falls annual peak flow record from 1915 to 1980. The best-fit distribution was determined to be the Log Pearson Type III.

Table 2.6 compares the flow estimates for the various return periods for Grenville and Chats Falls. In order to provide a clearer comparison between estimates, the flows for each return period are "normalized" by dividing through with the mean flow for the station. In this way, the normalized ratio can be compared directly with those of another location. The normalized flows for Grenville and Chats Falls show excellent agreement. Only approximately 3 percent difference exists between the normalized values. It may be assumed then that the peak flow estimates made at Grenville are consistent with those made at Chats Falls Dam.

2.5 SUMMARY AND CONCLUSIONS

Ottawa River data from the period 1911 to 1980 has been shown to be relatively independent and trend-free. As well, homogeneity has been statistically indicated during this period. The results of the flood frequency analysis are consistent between flow estimates made at Grenville and those at Chats Falls Dam.

Based on the above discussion, and the assumption that dam operation will continue in the same manner as it has historically, we recommend the flows tabulated in Table 2.6 be used in the hydraulic calculation of flood elevations.

TABLE 2.6

FLOW ESTIMATES FOR VARIOUS RETURN PERIODS

RETURN PERIOD	CHATS FALLS DA = 89,600 km^2		GRENVILLE DA - 143,000 km^2	
	Q(CMS)	$Q/\overline{X}$	Q(CMS)	$Q/\overline{X}$
2	3300	0.963	5200	0.962
5	4210	1.228	6520	1.207
10	4770	1.391	7360	1.362
20	5280	1.540	8130	1.504
50	5920	1.727	9120	1.688
100	6370	1.858	9840	1.821

3.0 HYDRAULICS

3.1 APPROACH

The purpose of the hydraulic computations was to obtain flood elevations along the river for flood events with return periods varying from 2 to 100 years. This was accomplished using a steady state numerical model of the river. Computer program HEC-2 developed by the U.S. Army Corps of Engineers was used to perform the computations. Topographic and hydrographic data to describe the geometry of the river was obtained from field surveys, topographic maps prepared by the National Capital Commission, hydrographic charts prepared by the Canadian Hydrographic Survey, specific hydrographic surveys carried out for this project and the 1:2000 scale topographic maps which are used to show the flood lines and fill lines.

The numerical model was calibrated using measured water levels from recent flood events. Bridges were modelled using information obtained from as-built drawings.

The results of the hydraulic analysis are documented in the computer output, cross section plots and the flood plain maps. Detailed discussion of the steps taken to produce these results are given in the following paragraphs.

3.2 STUDY AREA

The study area includes the Ottawa River from the east boundary of the Regional Municipality of Ottawa-Carlton near Cumberland to the west boundary of the Regional Municipality of Ottawa-Carlton. The flood plain in the study area is under the jurisdiction of two Conservation Authorities and the Regional Municipality.

Hydraulically the study area can be considered as four independent reaches. These are identified below:

- Downstream study limit to Chaudiere Dam
- Chaudiere Dam to Britannia Bay
- Britannia Bay to Chats Falls Dam
- Chats Falls Dam to upstream study limit

Each of these reaches is described separately in the following paragraphs.

3.3 CUMBERLAND TO CHAUDIERE DAM

The computations for this reach of the river were begun downstream of Rockland and extended upstream of the Parliment Buildings. Cross sections for this reach of river are numbered from 995 to 1044.

Two bridges cross this reach of the river downstream of the Parliment Buildings. These are the Macdonald-Cartier Bridge and the Alexandra Bridge. Two other bridges, the Portage Bridge and the Chaudiere Bridge cross the river immediately downstream of the Chaudiere Dam.

3.3.1 Design Flow Rates

The flow rates estimated for the various return periods are shown in Table 2.6. The data for Grenville was used along this reach. No reduction in flow rate was made to account for smaller watershed area in the upstream portion of this reach. However, during the calibration process, tests were made to assess the influence of flow rate variations along the river caused by tributary inflow. The results which are discussed later show that water levels are relatively insensitive to changes due to spatial variation of flow rate along the river.

3.3.2 Downstream Water Surface Elevations

The water surface elevation at the downstream limit of the computations (Section 995) was computed in the following manner. Water level at Carillon Dam was assumed to be at the elevation defined by the rule curve for that facility. Relationships which define water level as a function of flow rate and water level at Carillon Dam at the Cumberland gauge (Section 1007) and Grenville gauge have been developed by Ontario Hydro (see Reference 7). The data used to develop these relationships were observations made in the period 1970 - 1974. The water levels at the two gauge sites were estimated for each of the design flow rates and the water levels at the downstream end of the project were estimated by linearly interpolating between the two stations. Table 3.1 shows the estimated water levels at Carillon Dam, Grenville, Cumberland and Section 995. The computed water levels at Section 1007 were compared with those determined from the Ontario Hydro data. This was a check to confirm that linear interpolation of water levels between the Grenville and Cumberland gauges was reasonable.

TABLE 3.1

AT CARILLON DAM, GRENVILLE, CUMBERLAND AND SECTION 995

Return Period (yrs)	Flow Rate (cms)	Water Level (m)			
		Carillon Dam	Grenville	Cumberland	Section 995
2	5200	40.12	41.50	42.29	42.13
5	6520	40.56	42.03	42.95	42.77
10	7360	40.84	42.37	43.43	43.22
20	8130	40.84	42.51	43.77	43.52
50	9120	40.84	42.85	44.16	43.90
100	9840	40.84	43.07	44.44	44.17

3.3.3 Bridges

The MacDonald-Cartier Bridge and Alexandra Bridge were modelled using the special bridge routine due to the existence of piers in the river.

The Portage Bridge and Chaudiere Bridge cross over a confined channel below Chaudiere Dam. The rapid flow in this channel creates large waves which make hydrographic surveys very difficult. Thus, this short length of river and the two bridges over it were not modelled in the HEC-2 program. The floodlines were estimated by projecting upstream from the last section in the HEC-2 program. The channel sides are very steep to vertical, thus the position of the floodlines is not affected significantly by changes in water surface elevation.

3.3.4 Calibration

The numerical model of this reach of river was calibrated using the flood event of May 1974. This event was selected because flow data was available for the tributaries to estimate the distribution of flow along the river. Water level data was also available at the Rideau Locks to enable calibration at the upper end of the reach. Table 3.2 shows flow rates used for calibration of the model. The downstream water surface elevation was 43.40 meters elevation.

As a result of the calibration, the Mannings roughness coefficient for the channel was estimated to be 0.029. The coefficient for the flood plain/overbank areas was assumed to be 0.070. Most of the flow is confined to the channel and no sudden transitions in river width occur. Thus, the model will

TABLE 3.2

FLOW RATES USED FOR CALIBRATION OF HEC-2 MODEL
CUMBERLAND TO CHAUDIERE DAM
BASED ON FLOOD EVENT MAY 1974

Section Number	Flow Rate (m^3/s)
995	
	7825
1010	
	6983
1037	
	4517
1040	
	4440
1044	

Flow at Carillon Dam 8030 m^3/s

be relatively insensitive to the value of the roughness coefficient and expansion and contraction loss coefficients.

The differences between the measured surface water elevations and the estimated water elevations was ±0.01 meters at the Rideau locks and Cumberland gauge.

3.3.5 Flow Distribution

As noted previously the computations used to estimate flood elevations along the river assumed that the Grenville flow rate occurred along the total distance being studied. However, because of flow contributions from the Gatineau, Rideau, Lievre, Petit Nation, South Nation and Rouge Rivers; the flow rate at the upper end of the river will be lower than that at Grenville. Historical data shows that there is no fixed relationship between the flow in the Ottawa River and the flows supplied by the tributaries. To test the sensitivity of the computed water surface elevations to varying flow rates, computations were made assuming that flows for the various flood events were spatially distributed in portion to the May 1974 flood event. Table 3.3 shows a comparison of the computed water levels at Section 1044 assuming constant and distributed flow rates. Table 3.4 shows the flow rates at Section 1044 with these two conditions. Comparison of the water levels in Table 3.3 shows that water levels at Section 1044 would be lower by 0.18 to 0.20 meters if the flow rate was distributed in portion to the 1974 flood event. This difference is relatively small and does not significantly influence the position of the flood line. Thus, the constant flow rate along the reach is a reasonable assumption.

TABLE 3.3

COMPARISON OF FLOOD LEVELS AT SECTION 1044

Event Return Period (yrs)	Water Level (m)	
	Flow Varied*	Flow Constant
2	43.38	43.56
5	44.17	44.36
10	44.66	44.86
20	45.04	45.24
50	45.51	45.72
100	45.84	46.05

* Flow was assumed to be varied along the river in proportion to the flow variation of the 1974 flood event.

TABLE 3.4

COMPARISON OF FLOW RATES AT SECTION 1044

Event Return Period (yrs)	Flow Rate (cms)	
	Flow Varied*	Flow Constant
2	2951	5200
5	3700	6520
10	4177	7360
20	4613	8130
50	5175	9120
100	5583	9840

* Flow was assumed to be varied along the river in proportion to the flow variation of the 1974 flood event.

3.4 CHAUDIERE DAM TO BRITANNIA BAY

This short reach of river was the most challenging section of river in the Study Area. There are three sets of rapids, two bridges which span the river completely and two other bridges which span from the main land to Lemieux Island.

Hydrographic surveys of this section of river were carried out by Environment Canada. The presence of the rapids restricted the areas that could be surveyed. The lack of bathometric data at the crest of the rapids meant that artificial sections had to be deduced from available data. These data were used to simulate the control section that would form at the top of the rapids. This is discussed further in the following paragraphs.

3.4.1 Design Flow Rates

The design flow rates used for this reach of the river are those defined in Table 2.6 for Chats Falls. Analysis of flow records at the Britannia Bay gauge indicates that the flows at the two locations are virtually identical.

3.4.2 Downstream Water Surface Elevations

The downstream water surface elevation is controlled by the Chaudiere Dam. The facility is operated to give a headwater level of approximately 52.70 meters, regardless of flow rate.

3.4.3 Bridges

The pipe and road bridges to Lemieux Island and the Prince of Wales bridge across the Ottawa River were modelled using the normal bridge routine in the HEC-2 computer program. This approach was selected because the hydraulic profile would result in a low flow condition and the presence of Lemieux Island made the bridges equivalent to a multiple bridge opening situation. The special bridge routine was not used in this situation even though the bridge is supported on piers, because treatment of the Island as a wide pier using the semi-empirical Yarnell equation is not recommended for such flow conditions.

The Champlain Bridge was modelled using the special bridge routine because it is supported on piers.

3.4.4 Calibration

The Chaudiere and Remic Rapids cause hydraulic control sections to form. Thus, this reach of the Ottawa River is hydraulically three independent subreaches. Calibration of the hydraulic model was done using the 1979 flood event because peak flood elevations at the storm sewer outlets, measured by the City of Ottawa, were available.

Computation of the hydraulic profile from Chaudiere Dam to the foot of Chaudiere Rapids was done using a Mannings roughness coefficient of 0.030. It was found that the computed elevations were relatively insensitive to the roughness coefficient and the above noted value provided results which compared well with the measured peak flood elevations.

At Chaudiere Rapids, an artificial section (Section No. 2001.5) was assumed at the top of the rapids. The shape of the section was approximately trapezoidal with a deeper channel on the north side. This section was based on available hydrographic survey data obtained by Environment Canada in the area immediately upstream of the rapids. A series of trial and error computations were made in which the invert elevation of the artificial section was adjusted until the upstream water surface profile fit the measured data.

A similar process was used at the top of the Remic Rapids. In this case Section No. 2103.1 was a trapezoidal section based on information obtained from as-built drawings for the Champlain Bridge.

In both of the above cases the results were tested for sensitivity to the Mannings roughness coefficient. In both instances the computations were relatively insensitive to the roughness coefficient and a value of 0.030 was used.

The hydraulic profile in the rapids sections was not computed because geometric data was not available, the length of rapids is very short and flow conditions are complex in relation to the one dimensional capabilities of the HEC-2 computer program. From a flood plain management perspective, the profile in the rapids section is unnecessary.

3.5 BRITANNIA BAY TO CHATS FALLS DAM

The computations for this reach of the river begin at the top of the Deschenes Rapids and extends upstream to the foot of the Chats Falls Dam. Cross sections for this reach of the river are numbered from 2011 to 2059.

There are no bridges crossing the river in this reach.

3.5.1 Design Flow Rates

The flow rates estimated for the various flow rates are shown in Table 2.6. The data for Chats Falls was used along this reach of the river because of the close correlation between peak flow rates estimated at Chats Falls with those estimated at Britannia Bay.

3.5.2 Downstream Water Surface Elevations

The water surface elevation in Britannia Bay was estimated from the rating curve which the Water Survey of Canada had developed for the stream gauge in Britannia Bay (Station Number 02KF005). Table 3.5 shows the water surface elevations for each of the events which were considered in this study.

3.5.3 Calibration

The May 1974 flood event was used to calibrate the numerical model of this reach. Water level data was available at Britannia Bay, Quyon and the tailwater water of Chats Falls Dam. These results indicated that the Mannings roughness coefficient should be 0.024 for the main channel. The 1976 flood event was used to verify this result.

Most of the flood flow is conveyed in the main channel. Thus, the model will be relatively insensitive to the value of the Manning's roughness coefficient used for the overbank areas.

TABLE 3.5

WATER SURFACE ELEVATIONS IN BRITANNIA BAY
BASED ON WATER SURVEY OF CANADA RATING CURVE
FOR STATION NO. 02KF005

Return Period (yrs)	Flow Rate (cms)	Water Surface Elevation (m)
2	3300	59.48
5	4210	59.88
10	4770	60.12
20	5280	60.32
50	5920	60.61
100	6370	60.77

3.5.4 Flooding in Britannia Bay

The computations indicate that flooding will occur in the residential area east of the Britannia Pier. Water would flow overland and return to the Ottawa River downstream of the filtration plant.

Preliminary estimates of flow velocities through the residential area have been made. Based on our calculations, a flow velocity of approximately 1.50 m/s should be assumed for flood management purposes. The actual flow velocity at a particular location may be less than the above amount, depending on both local features and type of ground cover.

3.6 CHATS FALLS DAM TO STUDY LIMIT

This reach of river is the reservoir for Chats Falls Dam. A CN Rail bridge crosses the river at a point where the river narrows. Cross sections for this reach of the river are numbered from 3001 to 3011.

3.6.1 Design Flow Rates

Table 2.6 shows the design flow rates used for this reach of the river.

3.6.2 Downstream Water Surface Elevations

The water level at the downstream limit of this reach of river was determined from Ontario Hydro's forebay operating curves. Their policy is to control the forebay level so that Chats Lake water level is maintained at 74.22 meters elevation until

TABLE 3.6

WATER SURFACE ELEVATIONS IN CHATS FALLS
GENERATING STATION FOREBAY
BASED ON ONTARIO HYDRO DRAWING 146-1-1287
DATED MARCH 26, 1982

Return Period (yrs)	Flow Rate (cms)	Water Surface Elevation (m)
2	3300	73.21
5	4210	73.43
10	4770	73.50
20	5280	73.57
50	5920	73.67
100	6370	73.72

the flow rate exceeds 2180 m^3/s. When the flow rate exceeds 2180 m^3/s, the water level is allowed to rise so that the Chats Lake level follows the computed natural stage - discharge relationship at the Arnprior gauge. Table 3.6 shows the downstream water surface elevations for various return period flood events considered in this study.

3.6.3 Calibration

Calibration was carried out using the 1974 flood event and verified using the 1976 flood event. Water level data was available for the headwater of Chats Falls Dam and the Arnprior gauge. A Mannings roughness coefficient of 0.040 was used to obtain a satisfactory comparison between the measured and observed water levels.

4.0 WIND WAVE ANALYSIS

4.1 APPROACH

An analysis of wind generated waves and the potential flooding that would be caused by wave run-up during flood periods has been performed. This has been deemed necessary due to reports of flood damage due to waves at locations such as Britannia Bay.

The approach has been to assess the wind-wave conditions at four general locations in the study area to obtain guidelines suitable for planning purposes. The procedures used are documented in References 10, 21 and 22.

The steps in the process are:

- calculation of effective fetch
- determination of design wind speed
- estimation of wave height and period
- prediction of wave run-up height

Wave height is defined as the vertical distance between the wave trough and the wave crest. The wave period is the time between successive wave crests. The vertical distance which a wave travels up the slope above the still water height is defined as the wave run-up height.

These steps are described further in the following paragraphs.

The four areas of concern for wave run-up were Marshall Bay, Shirleys Bay, Constance Bay and Britannia Bay.

4.2 EFFECTIVE FETCH

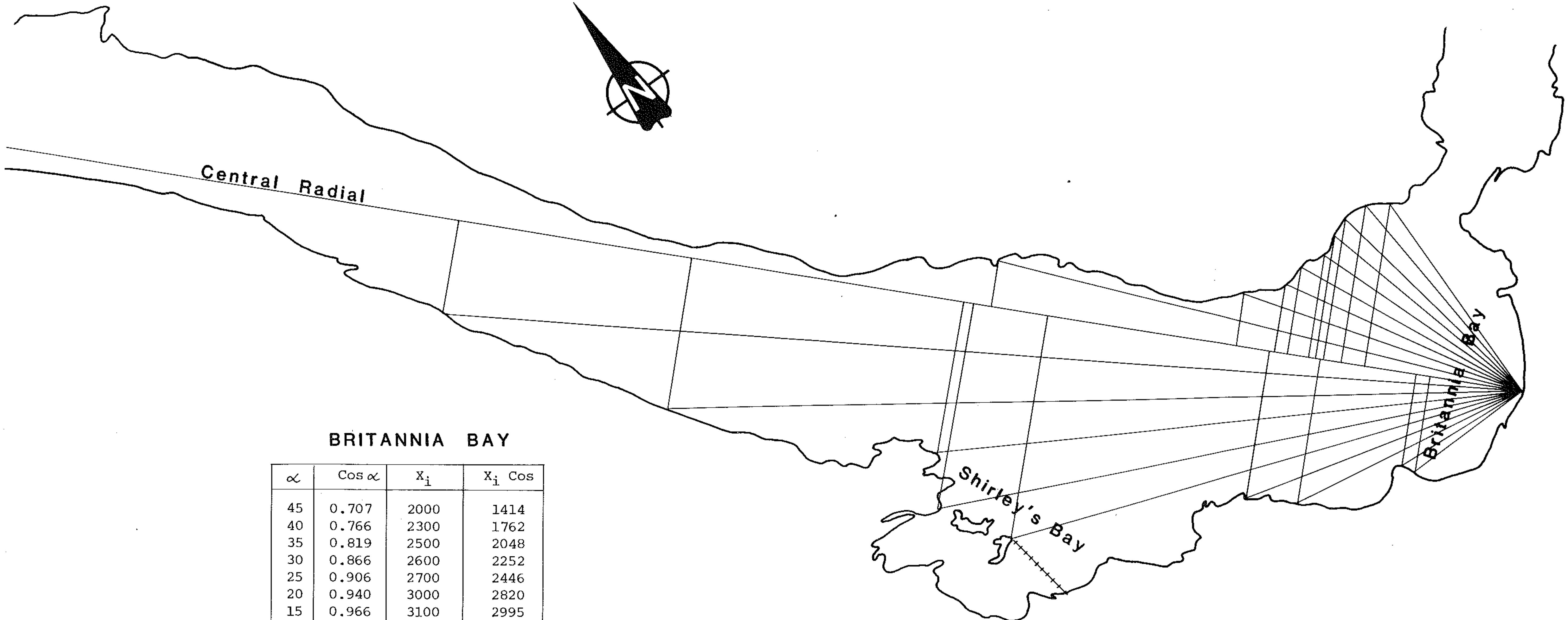
The effective fetch length is assumed to be a weighted average of the radial lines which extend from the point of interest upwind within an angle 45° angle on each side of the wind direction. The wind direction is generally assumed to occur in a direction which produces the longest effective fetch length. Figure 4.1 shows computation of effective fetch length for Britannia Bay.

The longest fetch lengths computed in each bay noted previously are shown in Table 4.1.

4.3 DESIGN WIND SPEED

There are a number of methods for selecting the design wind speed. One of these techniques is the use of joint probability analysis to assess the frequency of having high water levels and high wind speeds. This type of analysis would require considerable effort and may not provide realistic results due to data limitations. A second approach involves the somewhat arbitrary selection of a design wind speed. The simplicity of this method may be limited if there is not a substantial basis upon which to defend the selected velocity.

In this study we have obtained design wind speeds from the National Building Code for Ottawa. These wind speeds and the corresponding return periods are shown in Table 4.2.



BRITANNIA BAY

α	Cos α	X_i	X_i Cos
45	0.707	2000	1414
40	0.766	2300	1762
35	0.819	2500	2048
30	0.866	2600	2252
25	0.906	2700	2446
20	0.940	3000	2820
15	0.966	3100	2995
10	0.985	3600	3546
5	0.996	6700	6673
0	1.0	20700	20700
5	0.996	13400	13346
10	0.985	10500	10343
15	0.966	7100	6859
20	0.940	7000	6580
25	0.906	6000	5436
30	0.866	3100	2685
35	0.819	2500	2048
40	0.766	1350	1034
45	0.707	1150	813
	16.902		95800

EFFECTIVE FETCH

$$= \frac{95800}{16.902} = 5668 \text{ m}$$

COMPUTATION OF EFFECTIVE FETCH LENGTH BRITANNIA BAY

SCALE 1:50 000

FIGURE 4.1

TABLE 4.1

EFFECTIVE FETCH LENGTHS

Location	Fetch Length (m)
Brittania Bay	5670
Shirleys Bay	4700
Constance Bay	3430
Marshall Bay	3750

TABLE 4.2

DESIGN WIND SPEEDS
(1 hour duration)

Return Period (yrs)	Wind Speed (km/hr)
10	80
30	87
100	97

Sensitive tests were performed to assess the influence of selecting a different return period wind events. The methods used to predict wave heights are described below.

4.4 WAVE HEIGHT

Wave heights were estimated using forecasting curves for shallow water waves as presented in the "Shore Protection Manual". Figure 4.2 shows the curve for a constant water depth of 20 feet. The wave height and period are obtained by entering the graph with the fetch length and wind speed.

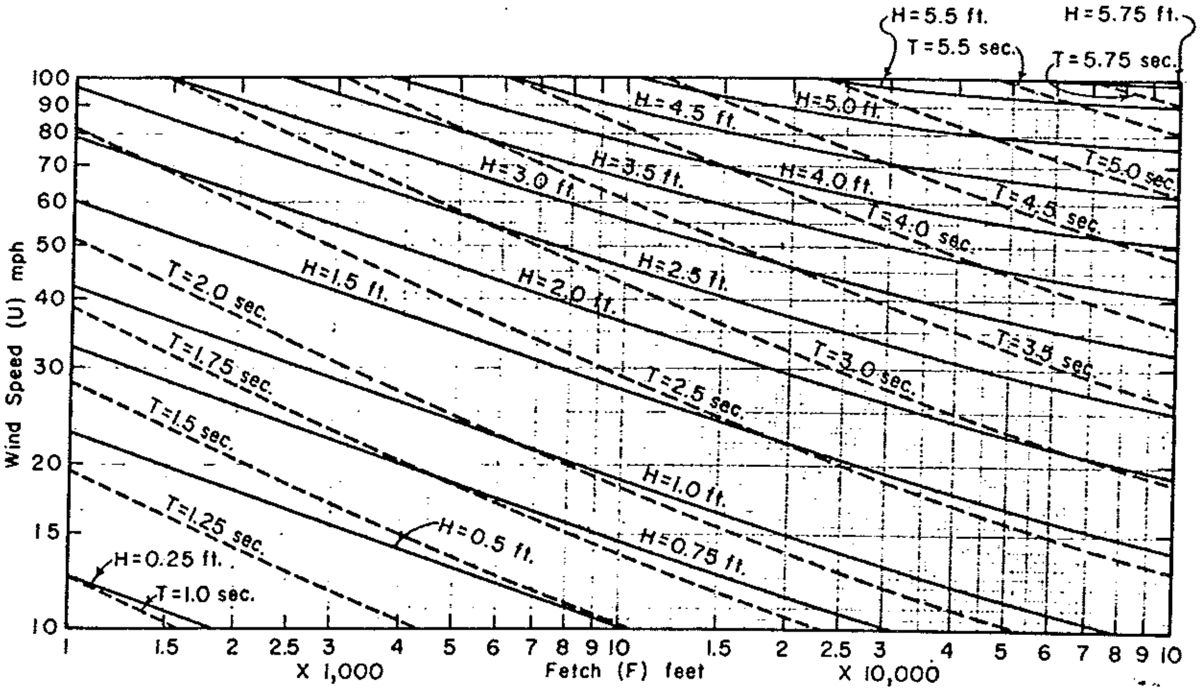
Using this figure, the wave height and period at the four areas of prime interest were estimated. These results are shown in Table 4.3.

4.5 WAVE RUN-UP

Wave run-up above the open water level occurs when the wave strikes the shore. The amount of run-up depends on the wave steepness (ratio of wave height to wave length), shore slope, roughness and permeability. In addition, the wave generated by the wind may be modified by shoaling and diffraction as it approaches the shoreline and this could also influence the amount of run-up.

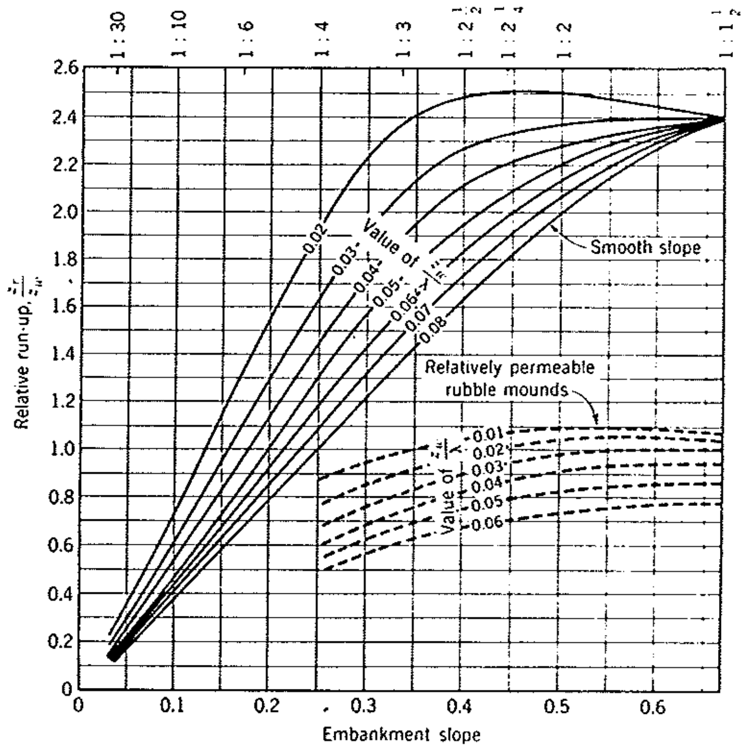
Figure 4.3 shows the influence of shoreline slope on wave run-up. A steep slope results in greater wave run-up. A rough permeable surface reduces the amount of run-up.

Estimates of wave run-up at the various locations of interest are shown in Table 4.4. These are based on typical shore



FORECASTING CURVES FOR SHALLOW WATER WAVES
CONSTANT DEPTH = 20 FEET
(FROM " SHORE PROTECTION MANUAL "
NOTE : IMPERIAL UNITS ARE SHOWN ON ABOVE FIGURE)

FIGURE 4.2



WAVE RUN-UP RATIOS VS. WAVE STEEPNESS AND EMBANKMENT SLOPES.
(FROM SAVILLE, McCLENDON AND COCHRAN)

Zr = WAVE RUN-UP (m)

Zw = WAVE HEIGHT (m)

λ = WAVE LENGTH (m)

= 1·56 T^2 (APPROXIMATELY)

T = WAVE PERIOD (S)

FIGURE 4.3

TABLE 4.3

WAVE HEIGHTS AND PERIOD

Location	Return Period (yrs)					
	10		30		100	
	Height (m)	Period (s)	Height (m)	Period (s)	Height (m)	Period (s)
Britannia Bay	0.97	3.6	1.03	3.7	1.14	3.9
Shirleys Bay	0.94	3.5	1.00	3.6	1.07	3.7
Constance Bay	0.85	3.3	0.91	3.4	1.04	3.6
Marshall Bay	0.85	3.3	0.91	3.4	1.04	3.6

TABLE 4.4

WAVE RUN-UP HEIGHT (m)

Shore Slope/ Location	30:1	10:1	4:1	2:1
Britannia Bay	0.17	0.51	1.53	2.50
Shirleys Bay	0.16	0.48	1.45	2.35
Constance Bay	0.15	0.47	1.40	2.30
Marshall Bay	0.15	0.47	1.40	2.30

slopes which exist along the river and the 1 in 100 year return period wave height. It has been assumed that shoaling is not influencing the wave height and that the shore slope is smooth and relatively impermeable.

4.6 APPLICATION OF WAVE RUN-UP CRITERIA

The application of wave run-up criteria in floodplain management should be done in the following manner. The wave run-up should be considered separately but in conjunction with the design flood level. The flood line has been established using only the 1 in 100 year flood level. No adjustment to the floodline has been made to include an allowance for wave uprush. If construction is proposed adjacent to the floodline, the effect of wave run-up above the floodline should be considered. The result may be that some type of flood proofing may be required. Also it should be noted that if a dike with relatively steep side slopes is used for flood protection purposes, additional dike height will be required to protect against wave run-up and overtopping. If dikes are used to protect against wave action on a shoreline with a flat slope, they should be set back from the edge of the flood fringe so that the waves can break on the flat foreshore prior to reaching the dikes.

No modification of the fill line criteria was necessary to account for wave up rush. The fill line is set back at least 30 metres from the floodline which is greater than the expected distance a wave would normally travel in land.

In some instances it may be appropriate to compute effective fetch, wave height and wave run-up on a site specific basis.

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