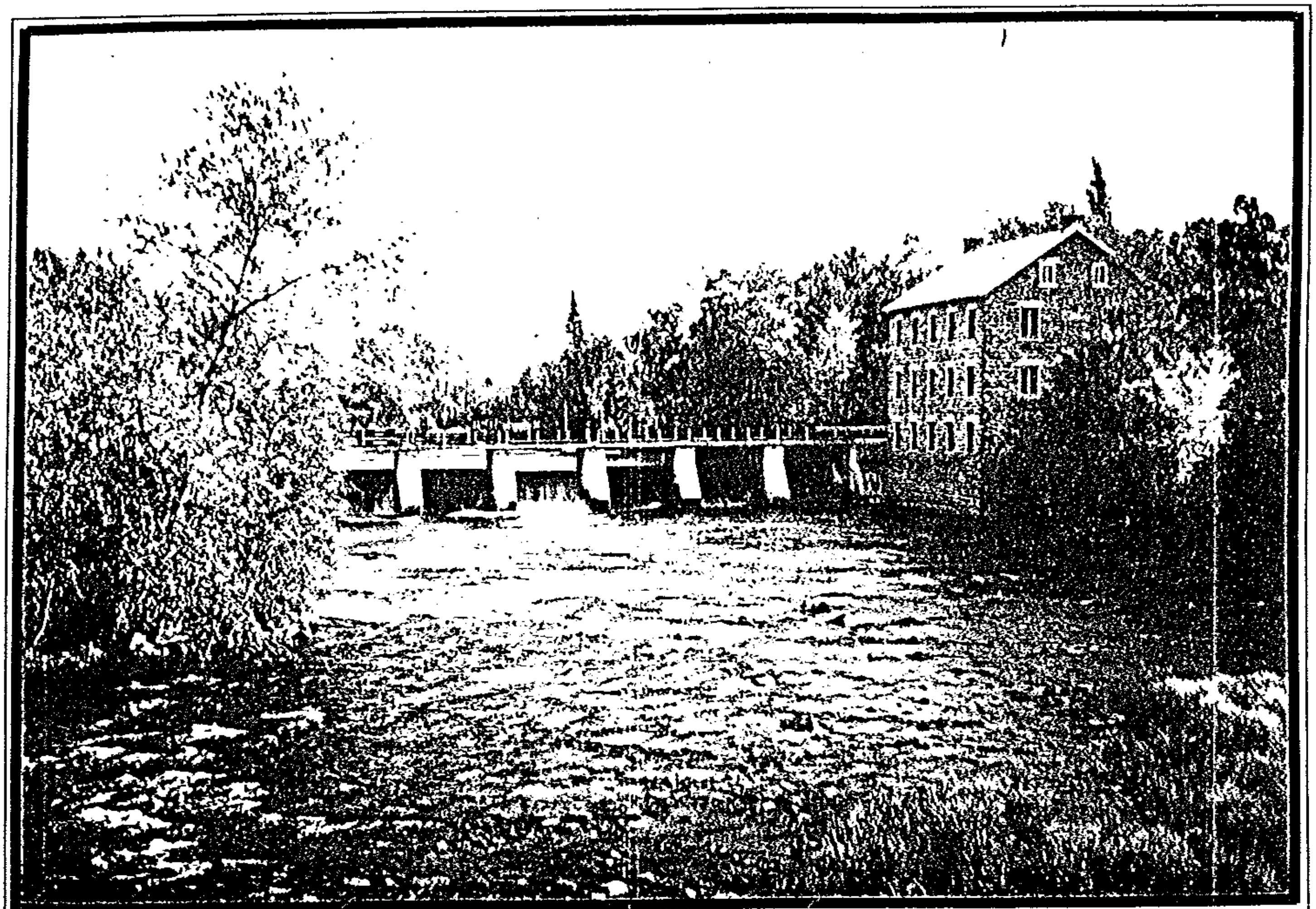
Ministry of Natural Resources
Ministère des Richesses naturelles
Ontario

Environment Canada
Environnement Canada

RIDEAU RIVER FLOOD RISK MAPPING STUDY

Mooney's Bay to Regional Road 6



CANADA / ONTARIO
FLOOD DAMAGE REDUCTION PROGRAM

Technical Report

RIDEAU VALLEY CONSERVATION AUTHORITY

DILLON

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Consulting Engineers • Planners
Environmental Scientists

OUR FILE: 1254-01
YOUR FILE:

21 February 1989

Rideau Valley Conservation
Authority
Box 599, Mill Street
Manotick, Ontario
K0A 2N0

Attention: Mr. Bruce Reid, P.Eng.

Rideau River Flood Risk Mapping Study - Mooney's Bay to Regional Road 6

Dear Sirs:

We are pleased to submit our Final Report for the above-mentioned study.

The report presents a comprehensive account of the hydrologic and hydraulic investigations undertaken, identification of the flood vulnerable areas, and appropriate recommendations for the reduction of flooding problems.

Yours truly,

M.M. DILLON LIMITED

Paul D. Holmes

PDH/mrb
Attach.

P.D. Holmes, P.Eng.
Project Engineer
for F.I. Lorant, P.Eng.
Project Manager

cc: R. Kallio, P.Eng.
L. Drennen, P.Eng.

M. M. DILLON LIMITED – 47 SHEPPARD AVENUE EAST, TORONTO, ONTARIO • TELEPHONE (416) 229-4646
MAIL: BOX 1850, STATION A, WILLOWDALE, ONTARIO, CANADA M2N 6H5 • TELEX 064-7540 • FAX (416) 229-4692

RIDEAU RIVER
FLOOD RISK MAPPING STUDY
MOONEY'S BAY TO REGIONAL ROAD 6
RIDEAU VALLEY CONSERVATION
AUTHORITY

1254-01
FEBRUARY 1989

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SUMMARY AND CONCLUSIONS

This study was undertaken as part of the Flood Damage Reduction Program established by the agreement signed by the Provincial and Federal Governments in 1978. The objective of the program is to promote proper management of the flood plain areas, and prevent or minimize future flood losses.

The study reach under consideration comprises approximately 29 km of the Rideau River, extending from Mooney's Bay (at Hog's Back Road in the City of Ottawa) to Regional Road 6 (between the Townships of Rideau and Osgoode) near the Village of Kars.

A description of the major components of the study is provided in the following.

Hydrologic Analysis

Design flows were computed for six events: the 2, 5, 10, 20, 50 and 100-year return periods. The flows were computed on the basis of a statistical frequency analysis of historical data recorded at two Water Survey of Canada hydrometric gauging stations.

Design flows resulting from the contribution of the Jock River tributary were determined by a frequency analysis of historical data and by a regional frequency analysis utilizing regression equations.

Design flows for the east and west branches around Long Island were determined in the hydraulic analysis by a series of assumed flows and computation of a balanced water level by utilizing the hydraulic model.

Hydraulic Analysis

Water surface profiles were produced for each of the above noted design events employing the HEC-2 computer program.

The necessary input data for the watercourse model was obtained from the following:

- above waterline cross-sectional data from 1:2000 scale topographic mapping produced by Airmap Ltd.;

- below waterline cross-sectional data from bathymetric soundings supplied by the Canadian Hydrographic Service;

- field surveys obtained from RVCA and conducted by M.M. Dillon Limited;

- general arrangement design drawings for various structures obtained from the Regional Municipality of Ottawa-Carleton, Canadian National Railways and the City of Ottawa.

Calibration of the HEC-2 model was conducted using flow and water level data supplied by RVCA for both the October 2, 1986 and March 27, 1988 events.

Sensitivity of predicted water level to river flow was examined for flows ranging from -15% to +15% of the predicted 100-year flow.

Flood Risk Maps

Regulatory Flood lines and water surface elevations associated with the 100-year flows are plotted on eighteen Flood Risk Maps which form part of this report. Also delineated is the Fill Line, to be applied for regulating any future development in the area.

The topographic mapping (1:2000 scale) was produced by Airmap Ltd. The maps were prepared based on 1:8000 scale aerial photography taken in April 1985.

In accordance with the specifications for flood plain mapping procedures, field survey work was undertaken by McElhanney Mapping Services Ltd. in June 1987 that verified both the horizontal and vertical accuracy of the mapping.

Flooding Concerns

The results of the hydrologic/hydraulic analyses indicate that several flood vulnerable areas exist throughout the Rideau River study reach.

The primary cause of flooding is due to the inadequate capacity of the existing channel resulting from low bank height associated in areas of flat topography. In addition, some isolated areas are prone to flooding due to culvert and ditch back-up.

On the Rideau River, within the study reach, a total of 259 structures are potentially at risk due to the Regulatory Flood (i.e. situated totally or partially within the 100-year flood plain). This included 142 residential structures such as cottages and seasonal dwellings and 117 ancillary structures such as sheds, boathouses, etc.

For most of the study reach, from Mooney's Bay to upstream of Long Island the number of structures prone to flooding is relatively low. Altogether 70 buildings are affected, they are primarily sheds, boat houses and seasonal cottages/residences. In the remaining 7 kilometres, extending upstream of Long Island to Regional Road 6, 190 buildings are affected. These buildings are primarily residential dwellings.

Almost all of the structures susceptible to flooding at the 100-year flood level are exposed to flood depths of less than 0.6 m (2 ft.) and in many cases less than 0.3 m (1 ft.). Flooding of many structures does not occur until the exceedence of the 5-year return period level.

Recommendations

Based on the findings of this study, the following recommendations were formulated.

1) The Rideau River Conservation Authority should make available the information contained on the Flood Risk Maps to the municipalities for inclusion into their Official Plan documents. In consultation with the Ministry of Natural Resources, the Authority should encourage the municipalities to develop policies for inclusion into the Official Plan which:

   - describe the flood susceptibility and risk associated with the flood plain areas;

- restrict new buildings or structures which are prone to flood damages or which may cause adverse impacts to existing development or lands;

- address additions or alterations to existing buildings or structures, and replacement of buildings or structures situated in the flood plain;

- describe the public and private works which may locate in the flood plain;

- advise property owners located in the flood plain of the flooding implications, and inform them of alternative floodproofing measures which can be implemented.

2. Based on an assessment of the flooding problems, and a preliminary examination of the various alternatives the Authority should investigate the possibility of:

   - quantifying "Average Annual Flood Damages" in the study reach as a precursor to investigations of flood damage reduction;

   - isolated berming in areas where land availability and drainage requirements permit and can be justified economically;

   - flood proofing of structures where development is scattered and the number of affected buildings is limited.

3. The Authority should continue its implementation of a flood forecasting effort involving flow monitoring, snow

and ice pack monitoring, etc. and implementation of a flood warning system through coordination with municipality officials.

4. Upon approval by the Province of the proposed revisions to Provincial Flood Plain Planning Policies, the Authority should conduct a review of the applicability of implementation of a Two Zone concept that defines the flood fringe and floodway.

5. The Authority should investigate the possibility of gathering additional water level elevations through surveys carried out during high flood flows. This is especially important along both branches of the Long Island flow split and in the vicinity of Regional Road 6, at the upstream study limit. Continued water level measurements during spring events would provide future confirmation (or improvement) of current flood predictions.

1. INTRODUCTION

In September 1986, M.M. Dillon Limited was retained by the Rideau Valley Conservation Authority to conduct a flood risk mapping study along 29 km of the Rideau River. Undertaken as part of the Canada/Ontario Flood Damage Reduction Program, the principal objective was to delineate the Regulatory Flood and Fill Lines along the watercourse.

In a continued effort of maintaining its primary role of flood plain management, the Authority has commissioned this flood and fill line study to be used in administering its "Fill, Construction and Alterations to Waterways Regulations", and in flood and land use planning and control thereof.

This report presents the details of the hydrologic and hydraulic investigations; it includes a comprehensive account of:

i) the relevant background data collected;

ii) the hydrologic analysis including a review of the hydrology of the Rideau River within the study reach culminating in computed flow values of the 2, 5, 10, 20, 50 and 100-year flood discharges;

iii) the hydraulic analysis including the assemblage of a hydraulic model of the study reach;

iv) subsequent delineation of flood lines and fill lines on flood risk maps;

v) all methods, assumptions and considerations employed throughout the analyses.

2. WATERSHED DESCRIPTION

2.1 General Features

The Rideau River watershed is located in Southeastern Ontario comprising portions of the Counties of Frontenac, Lanark and Leeds-Grenville, and the Regional Municipality of Ottawa-Carleton (refer to Figure 1). Oriented in a north-south direction, the basin is approximately 130 km in length and has an average width of 30 km (refer to Figure 2). The Rideau River drains an area of about 3,880 km^2 at its confluence with the Ottawa River.

The invert elevation of the Rideau River at its confluence with the Ottawa River is approximately 50 m, while the head-water regions reach an elevation of 213 m. The average river gradient is 0.44 m per kilometre.

2.2 Climatic Characteristics

The Rideau River watershed exhibits climatic characteristics typical of Southeastern Ontario and identifiable with those observed at the National Research Station located in Ottawa. The normal climatic characteristics of this station are summarized in Table 1.

2.3 Description of Study Reach

The limits of the study reach for the purpose of flood plain identification are depicted in Figure 2 and the locations of the relevant points of reference shown in Figure 3.

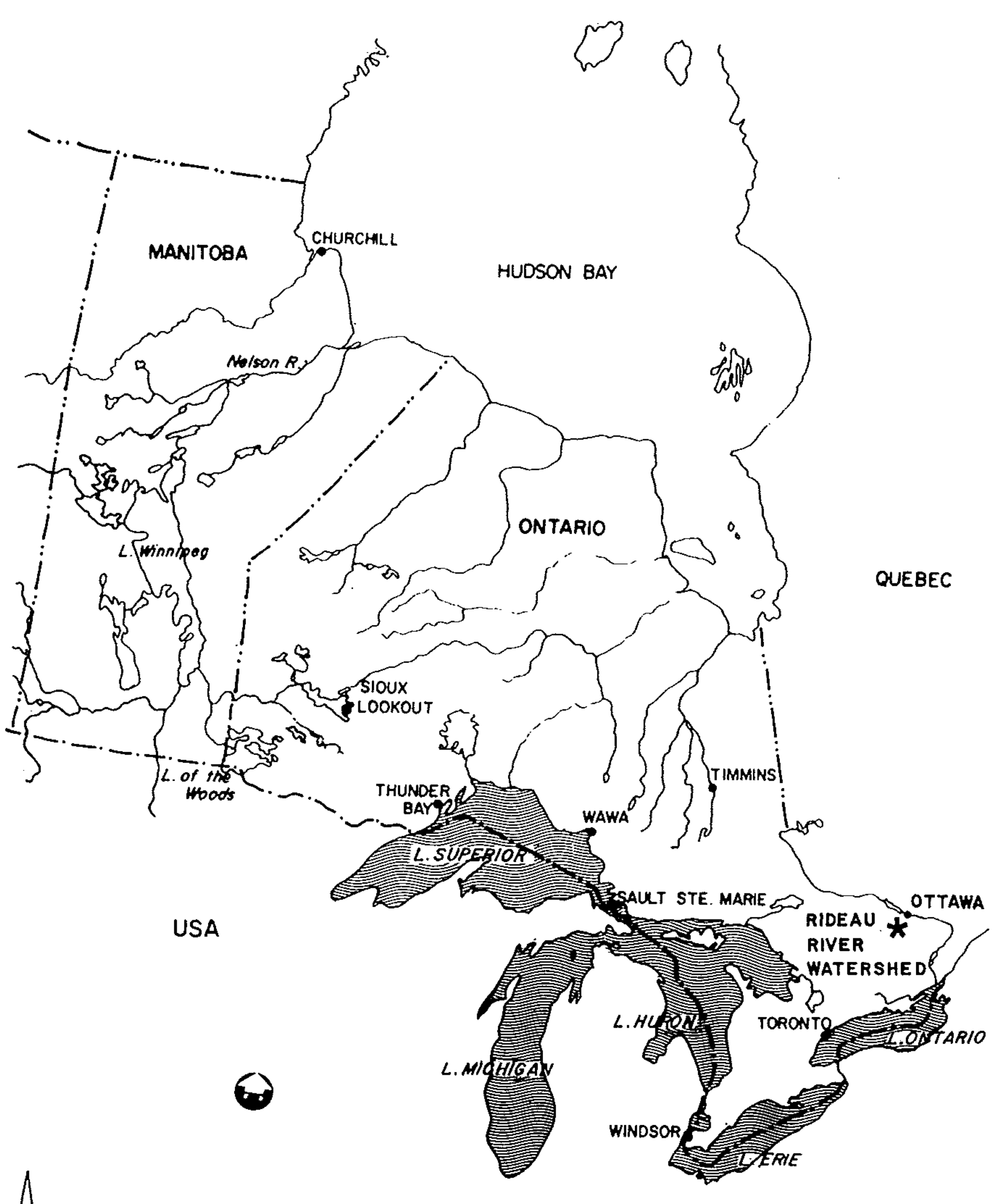
TABLE 1
RIDEAU RIVER WATERSHED - CLIMATIC CHARACTERISTICS*

Mean annual precipitation	875 mm
Mean annual rainfall	680 mm
Mean annual snowfall	195 cm
Mean daily temperature	6° Celsius
Mean daily temperature for July	21° Celsius
Mean daily temperature for January	-11° Celsius
Average Number of days with measurable precipitation	137
Average number of days with measurable rainfall	98
Average number of days with measurable snowfall	45

Greatest recorded 24-hour rainfall	71 mm
Greatest recorded 24-hour snowfall	33 cm

Snow cover forms in late November, early December.
Snow cover disappears in late March, early April.

* Obtained from National Research Centre, Ottawa Recording Station.



RIDEAU VALLEY
CONSERVATION AUTHORITY

Rideau River
Flood Risk Mapping Study

LOCATION PLAN

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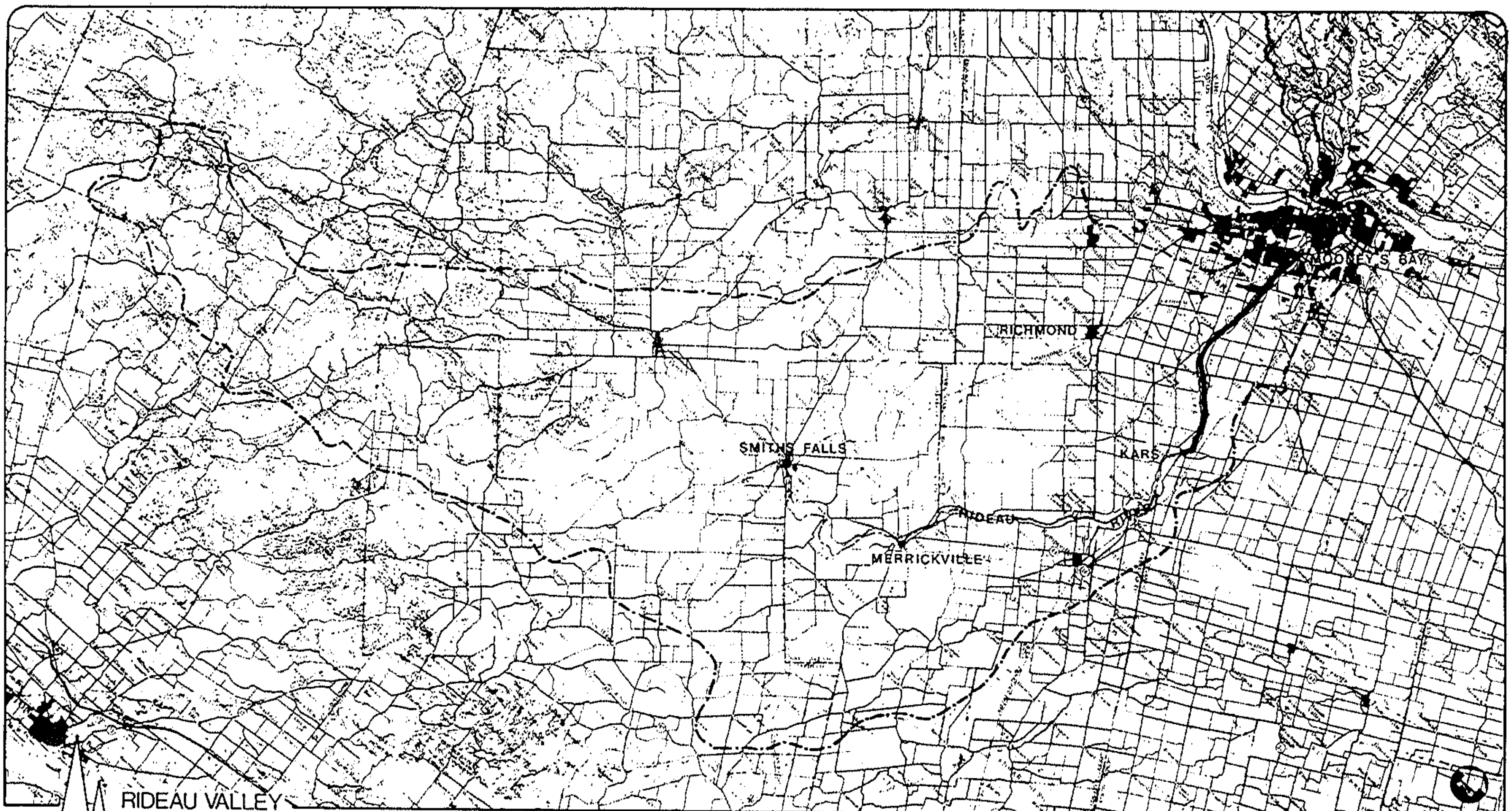
Figure 1



RIDEAU VALLEY CONSERVATION AUTHORITY

LEGEND

- Study Reach
- Watershed Boundary

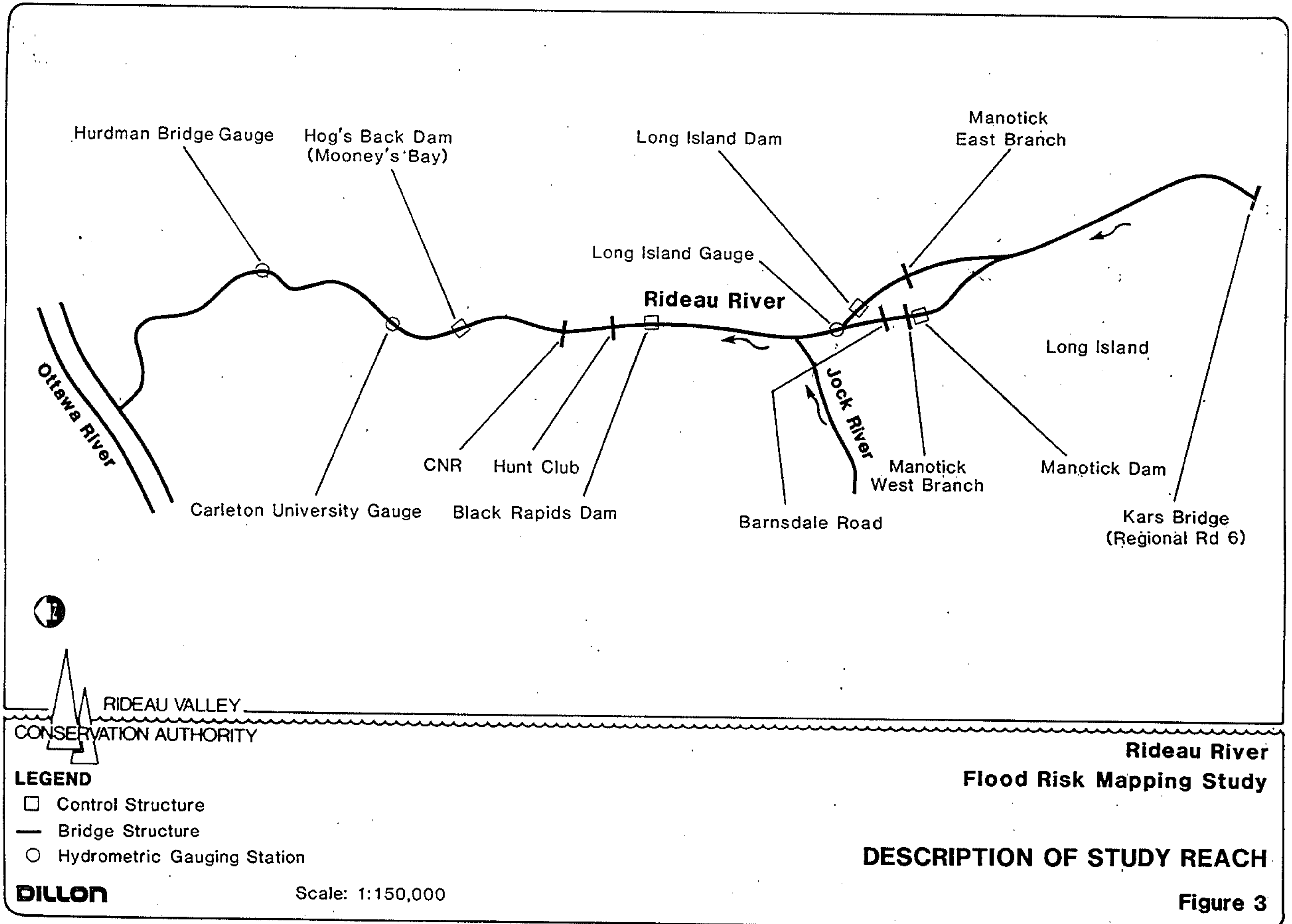
DILLON

Scale: 1:600,000

Rideau River
Flood Risk Mapping Study

RIDEAU RIVER WATERSHED & LOCATION OF STUDY REACH

Figure 2



DESCRIPTION OF STUDY REACH

Figure 3

The study reach comprises approximately 29 km of the Rideau River, extending from Mooney's Bay (at Hog's Back Road in the City of Ottawa) to Regional Road 6 (between the Townships of Rideau and Osgoode) near the Village of Kars. The Jock River, a major tributary with a drainage area of 559 km^2, joins the Rideau River about 14 km above Hog's Back.

Long Island, situated midway in the study reach, is approximately 6 km in length and effectively splits the Rideau River flow into two separate waterways; an east and west branch which rejoin approximately 20 km from Hog's Back.

Water levels are regulated by four control structures situated throughout the study reach. Hog's Back Dam, a large, eight-bay dam, is located at the downstream limit. Approximately 8 km upstream lies the Black Rapids Dam which is comprised of an ogee spillway crest that utilizes two small waste weirs to regulate levels. Long Island Dam, the second major control structure is located at the beginning of the east branch around Long Island. Flow control on the west branch around Long Island is maintained by the Manotick Dam located at the midpoint of the west branch.

In addition to the four control structures, seven major bridges are located throughout the reach. All of the structures have sufficient capacity to convey flows up to and including the 100-year event.

Steep slopes combined with dense vegetation comprise the river banks extending from Mooney's Bay to Black Rapids. Upstream of Manotick extending to Kars, the banks become predominantly flatter in relief. Composed almost entirely of clay-silt and clay loam complexes, the steeper river bank slopes are prone to erosion and failure as evident in the lower half of the river reach. For the most part, except in areas of low relief, the river banks are of sufficient steepness and height to effectively convey all flows including flood discharges.

The river itself, meanders only slightly and maintains a channel shape that is basically trapezoidal; bottom material consists of coarse gravel, cobbles and boulders. In addition, small wetlands supporting a variety of weed growth and waterfowl occupy isolated shoreline areas, and in some instances actually extend out into the river channel as small point bars or isolated islands.

3. HYDROLOGIC ANALYSIS

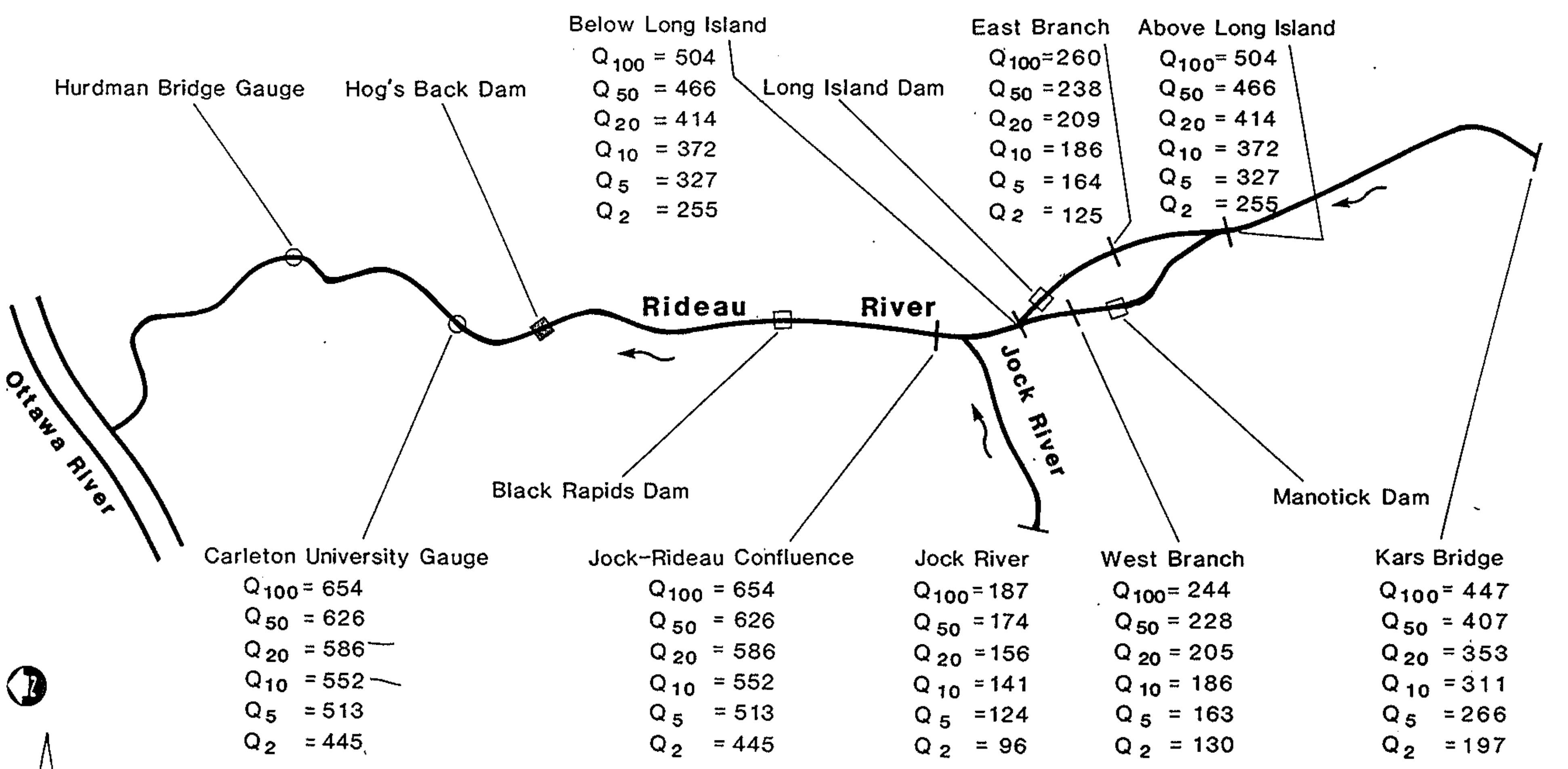
3.1 General

A number of reports have been prepared on the Rideau River flows since the first flood plain mapping report was completed by M.M. Dillon for the Rideau Valley Conservation Authority in 1972. Each report used the additional flow data available to extend the data base, and the latest statistical methods to predict the 100 year flows.

The Rideau River is one of the few Canadian Rivers for which flow data dates back several decades. Man-made activity such as flow regulation have had some effect on the observed flows, but the actual extent is unclear. As a result, where possible, adjustments have been made to include conversion of the maximum annual mean daily flows recorded to represent maximum instantaneous flows, and alteration of these flows to account for the change in operating procedures at the Poonamalie Dam structure.

The following hydrology study incorporates flow records observed up to 1986 and uses the latest CFA88 (Consolidated Frequency Analysis) statistical computer program. Released by Environment Canada in 1987, CFA88 computes flood peaks for given recurrence intervals by using a variety of statistical distributions.

Altogether, eight flow gauges (or dam records) provided data one time or another in the past on flows in the Rideau River. Table 2 lists the stations and the data available. Figure 2 displays the Rideau River watershed and relevant points of reference are depicted in Figure 3.



RIDEAU VALLEY CONSERVATION AUTHORITY

Rideau River

Flood Risk Mapping Study

RIDEAU RIVER DESIGN FLOWS
MAXIMUM INSTANTANEOUS PEAKS (m^3/sec)

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Scale: 1:150,000

Figure 4

TABLE 2
SUMMARY OF AVAILABLE FLOW RECORDS

Location	Drainage Area Square Kilometres	Source	Period
Rideau River at Ottawa (02LA002) (Hurdman Bridge)	3,860	Water Survey of Canada	1933 - '66
Rideau River at Ottawa (02LA004) (Carleton University)	3,830	Water Survey of Canada	1967 - Present
Rideau River at Long Island	3,120	Rideau Canal Office, Smiths Falls	1948 - '80
Rideau River Below Manotick (02LA012)	3,120	Water Survey of Canada	1981 - Present
Rideau River at Merrickville	1,920	Rideau Canal Office, Smith Falls	1942 - 1979
Rideau River below Merrickville (02LA011)	1,920	Water Survey of Canada	1980 - Present
Rideau River at Poonamalie	1,290	Rideau Canal Office, Smiths Falls	1944 - '71
Rideau River above Smiths Falls (02LA005)	1,290	Rideau Canal Office, Smiths Falls	1972 - Present
Jock River near Richmond (02LA007)	559	Water Survey of Canada	1970 - Present

In order to calculate flows for the entire study length, the following approach was adopted:

- For the downstream point at Hog's Back, the long-term Ottawa flow station data recorded at both the Hurdman Bridge and Carleton University gauges was used.

- For the downstream location at Long Island, the combined Long Island gauge records and Manotick flow station data was used.

- Tributary inflow was computed from recorded data for the Jock River and compared with results obtained from regression equations.

- Flows for the east and west branches around Long Island were determined in the hydraulic analysis by utilizing the hydraulic model.

- Flows for the upstream study limit at Kars were determined by transposition of the flows established at Long Island.

3.2 Single Station Frequency Analyses

3.2.1 Rideau River at Ottawa

The first task was to prepare a data base consisting of instantaneous peak flows.

Continuous, year-round maximum mean daily flows from 1947 to 1986 (with the exception of 1948) were utilized by combining recorded flows from both the Hurdman Bridge (1947-66) and subsequently the Carleton University (1967-1986)* sites.

Prior to 1971, the gauge recorded maximum mean daily flows only. The 1971-1986 flows, which contained both the mean daily and instantaneous peaks, were analyzed to derive a ratio between the two flows as given in Table 3.
A comparison of the two quantities showed that the ratio varies between 1.013 and 1.148 with an average of 1.06 (6%). The magnitude of flood peak does not influence the ratio, therefore, the average was accepted to be applicable for all flows prior to 1971. The recorded mean daily flows before 1971 and for 1974 (since the maximum instantaneous was absent) were increased by 6.0% to obtain instantaneous peak flows.

A second adjustment was carried out to account for the change in the operation of the Poonamalie structure, as described in the 1984 A.J. Robinson & Associates Inc. report "Flood Risk Mapping of the Rideau River". The operation of the control structure, which is located near the head water lakes, was changed in 1977 from a summer conservation rule to a spring flood control rule. This operational change provides increased control during the high runoff periods to reduce the flood peaks downstream.

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* In 1966 the gauging station was moved upstream from the Hurdman Bridge to Carleton University.

TABLE 3
RIDEAU RIVER AT OTTAWA, PEAK FLOWS, (02LA004)

	Maximum Instantaneous		Maximum Mean Daily		Maximum Instantaneous (Maximum mean Daily x 1.06)	
Year	m³/sec	(cfs)	m³/sec	(cfs)	m³/sec	(cfs)
1986	256	(9040)	233	(8230)	256*	(9040)
1985	276	(9750)	265	(9360)	276*	(9750)
1984	398	(14050)	385	(13600)	398*	(14050)
1983	246	(8690)	224	((7910)	246*	(8690)
1982	435	(15360)	397	(14020)	435*	(15360)
1981	446	(15750)	435	(15360)	446*	(15750)
1980	421	(14870)	385	(15600)	421*	(14870)
1979	423	(14940)	403	(14230)	423*	(14940)
1978	527	(18610)	487	(17200)	527*	(18610)
1977	473	(16700)	467	(16490)	473*	(16700)
1976	597	(21080)	583	(20590)	597*	(21080)
1975	413	(14590)	394	(13910)	413*	(14590)
1974			396	(13990)	420	(14830)
1973	464	(16390)	447	(15790)	464*	(16390)
1972	578	(20410)	535	(18890)	578*	(20410)
1971	513	(18210)	496	(17520)	513*	(18120)
1970			442	(15610)	469	(16560)
1969			328	(11580)	348	(12300)
1968			377	(13310)	400	(14130)
1967			311	(10980)	330	(11650)
1966			215	(7590)	228	(8050)
1965			146	(5160)	155	(5470)
1964			109	(3850)	116	(4100)
1963			442	(15610)	469	(16560)
1962			323	(11410)	342	(12080)
1961			193	(6820)	205	(7240)
1960			532	(18790)	564	(19920)
1959			413	(14590)	438	(15470)
1958			306	(10810)	324	(11440)
1957			133	(4700)	141	(4980)
1956			351	(12400)	372	(13140)
1955			493	(17410)	523	(18470)
1954			405	(14300)	429	(15150)
1953			331	(11690)	351	(12400)
1952			379	(13380)	402	(14200)
1951			419	(14800)	444	(15680)
1950			447	(15790)	474	(16740)
1949			379	(13380)	402	(14200)
1947			538	(19000)	570	(20130)

* Recorded maximum instantaneous values.

The Rideau River at Poonamalie drains an area of approximately one-third of the total drainage area at Ottawa. Ratios of annual mean daily flood peaks at the two locations in the 1977-1986 period averaged 4.7% compared to 9.6% in the 1972 to 1977 period, which illustrates the effect of the change in the rule curve operation depicted in Table 4. The difference between the two, approximately 5%, was used to increase the 1977-1986 Ottawa flows to account for the higher annual maxima which would have occurred had the operating procedure been discontinued or had the Rideau Canal Office been unable to control the discharge at Poonamalie.

It should be noted that the limits of the Canal's ability to exercise flow control at Poonamalie have not to date been determined. Therefore, in order to justify this adjustment of the flow series, it can be pointed out that since 1977 no major flood flow has been experienced. Of the ten post-1977 flows, eight were 2-year flow or less, one was less than 5-year flow, one was less than 10-year flow.

The maximum instantaneous flow series for the 39-year record used for the statistical analyses is presented in Table 5.

To estimate the return period flows requested in the Terms of Reference, the latest version of the CFA88 computer model was used. CFA88 employs a series of standard non-parametric tests for homogeneity, trend, independence and randomness on the sample data, and fits a series of probability distributions to the sample.

It also performs tests for the existence of high and/or low outliers. For the latter case, the existence of low outliers can, in some instances, affect the skewness of the sample and result in difficulty in fitting the distribution functions.

TABLE 4

EVENT BASED COMPARISON OF ANNUAL MAXIMUM MEAN DAILY FLOWS AT POONAMALIE AND OTTAWA

Date	Poonamalie m³/sec	Poonamalie (cfs)	Ottawa m³/sec	Ottawa (cfs)	Ratio (%)
1986 May 24	28.5	(1010)	223	(7870)	12.80
1985 March 15	26.1	(920)	265	(9360)	9.85
1984 April 16	18.1	(640)	385	(13600)	4.70
1983 March 20	4.2	(150)	224	(6910)	2.80
1982 April 1, 2	11.7	(410)	397	(14020)	2.95
1981 February 24	35.1	(1240)	435	(15360)	8.09
1980 March 22	2.3	(90)	385	(15600)	0.52
1979 March 25	3.4	(120)	403	(14230)	0.84
1978 April 14	21.4	(750)	487	(17200)	4.37
1977 March 15	0.0	(0)	467	(16490)	0.00
1976 March 28	55.8	(1970)	583	(20590)	9.57
1975 April 20	31.4	(1110)	394	(13910)	7.98
1974 April 6	55.2	(1950)	396	(13990)	13.94
1973 March 18	30.9	(1090)	447	(15790)	6.90
1972 April 21	51.8	(1830)	535	(18890)	9.69

Pre 1977		Post 1976	
	9.57		2.95
	7.98		8.09
	13.94		0.52
	6.90		0.84
	9.69		4.37
			0.00
	48.08 ÷ 5 = 9.62*		16.77 ÷ 6 = 2.80*

% DIFFERENCE = 9.62 - 2.80 = 6.82, say 7.0%*

* As stated in the AJ Robinson Study

Pre 1977 = 9.62 (as before)	Post 1976 =
	12.80
	9.85
	4.70
	2.80
	2.95
	8.09
	0.52
	0.84
	1.37
	0.00
	46.92 ÷ 10 = 4.69

% DIFFERENCE = 9.62 - 4.69 = 4.93, say 5%

TABLE 5
RIDEAU RIVER AT OTTAWA, 02LA004
MAXIMUM INSTANTANEOUS FLOWS USED FOR SINGLE STATION FREQUENCY ANALYSIS

Year	Month	FLOWS* m³/sec	FLOWS* (cfs)
1986	5	269	(9490)
1985	3	290	(10240)
1984	4	418	(14760)
1983	3	258	(9110)
1982	4	457	(16140)
1981	2	468	(16530)
1980	3	442	(15610)
1979	3	444	(15680)
1978	4	553	(19530)
1977	3	497	(17550)
1976	3	597	(21080)
1975	4	413	(14590)
1974	4	420	(14830)
1973	3	464	(16390)
1972	4	578	(20410)
1971	4	513	(18120)
1970	4	469	(16560)
1969	4	348	(12300)
1968	3	400	(14130)
1967	4	330	(11650)
1966	3	228	(8050)
1965	12	155	(5470)
1964	4	116	(4100)
1963	3	469	(16560)
1962	4	342	(12080)
1961	3	205	(7240)
1960	4	564	(19920)
1959	4	438	(15470)
1958	3	324	(11440)
1957	3	141	(4980)
1956	.4	372	(13140)
1955	4	523	(18470)
1954	4	429	(15150)
1953	3	351	(12400)
1952	4	402	(14200)
1951	4	444	(15680)
1950	4	474	(16740)
1949	3	402	(14200)
1947	4	570	(20130)

* Increased flows after 1976 by 5 percent reflects old Operating Rule (Pre 1977)

The presence of low outliers can cause the theoretical distribution to bend more steeply downward at the lower return periods. This is usually of no consequence since the range of interest is normally the higher return period events.

Once the low outliers have been identified, CFA88 employs a "mathematical retrofitting*" technique that overcomes the aforementioned drawbacks and improves the shape of the curve. This process is normally done automatically.

A visual observation of the plotted points identified possible low outliers. The CFA88 computer program confirmed the existence of one low outlier. To establish the significance of the low outliers on the predicted high flows, a sensitivity analysis was carried out, assuming three different conditions:

i) one outlier mathematically identified by the computer;

ii) five outliers identified (as determined in the 1984 A.J. Robinson study) but not removed from the data set;

iii) five outliers identified and manually removed from the data set (as done in the 1984 A.J. Robinson study).

The results of the sensitivity analysis using the three-parameter lognormal distribution are presented in Table 6. For the 100 year flow estimate the arbitrary removal of the five lowest recorded flows produced an increase in flows of 2.5%. Following discussions with

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* Mathematical retrofitting is employed only for the three-parameter lognomal, generalized extreme variable and Weibull distributions. The Log Pearson Type III uses a method based on synthetic statistics.

TABLE 6

RIDEAU RIVER AT OTTAWA

SENSITIVITY ANALYSIS OF LOW OUTLIERS

Average Return Period - Years	1984 A.J. Robinson Study 1947-1982, 5 Low Outliers Removed From Data Set	1 Low Outlier Identified Mathematically by CFA88	5 Low Outliers Identified Manually, Left in Data Set	5 Low Outliers Identified Manually Removed From Data Set
2	445	417	419	435
5	513	503	439	508
10	552	540	499	545
20	586	568	569	575
50	626	596	603	608
100	654	614	626	630

Note: All flows are instantaneous peaks in m^3/sec.
All tabulated flow estimates generated by three-parameter lognormal distributions.

Environment Canada personnel, it was decided not to remove the low outliers from the data set; such a procedure could have a tendency to make the data set an unrepresentative sample. In any case, the relative change in flow value at the 100-year level is insignificant.

Four statistical distributions were applied to generate flows:

- generalized extreme value (GEV);
- three-parameter lognormal (3PLN);
- Log Pearson Type III (LPIII);
- Wakeby.

A summary of the computed flows for selected frequencies are presented in Table 7 along with the past flow prediction determined in the 1984 Robinson Study.

TABLE 7
RIDEAU RIVER AT OTTAWA
COMPARISON OF STATISTICAL DISTRIBUTIONS

Average Return Period - Years	1987 M.M. Dillon Study 1947-1986 Flow Series Statistical Distribution				1984 A.J. Robinson Study 1947-1982 Flow Series
	GEV	3PLN	LPIII	Wakeby	3PLN
2	414	417	416	421	445
5	511	503	512	494	513
10	554	540	548	540	552
20	584	568	571	585	586
50	611	596	589	639	626
100	626	614	597	678	654

CS = -0.054
CK = 2.927

Note: Data base has one low flow outlier.
All flows are instantaneous peaks in m^3/sec.

From an examination of the flows predicted by the various statistical distributions presented in Table 7, it can be seen that the 100-year flow ranges from 597 m^3/s to 678 m^3/s with an average of 629 m^3/s. This represents a 4% lower estimate, or a 6% lower estimate if one were to compare the three-parameter lognormal distributions directly to the 1984 Robinson estimate of 654 m^3/s. Only the Wakeby distribution yielded a 100-year flow higher than the past estimate.

Based on the following considerations, the flows presented in the 1984 A. J. Robinson study were adopted for use:

- The flows predicted in the 1984 study were well within the range of values currently predicted.

- The 1984 study flows are conservative.

- Sensitivity testing (discussed in Section 4.5) of the hydraulic model indicated that a 5% increase in flow would not result in any appreciable increase in the horizontal extent of the 100-year flood line.

- The 1984 study flows provide for a continuity of flow through the lower study limit at Hog's Back Road and consistency with previous floodplain mapping.

The adopted flows are summarized in Table 8.

TABLE 8
RIDEAU RIVER AT OTTAWA
DESIGN FLOWS

Average Return Period-Years	1984 A.J. Robinson Study 3PLN (m^3/sec)
2	445
5	513
10	552
20	586
50	626
100	654

3.2.2 Rideau River Below Manotick

Observations on flows are available at Long Island just below the Long Island Dam, upstream of the confluence of the Jock and Rideau Rivers. The data for the original gauge had been collected by the Rideau Canal office for the period 1948 to 1980. In 1981 Water Survey of Canada assumed the collection of flow data at this location.

The Long Island data contains only mean daily flows. To convert the data to instantaneous flows, seven years of mean daily and instantaneous peak flows of Manotick data were analyzed, as shown in Table 9. Comparison of the two quantities revealed that the ratio varies from 1.05 to 1.28 with an average of 1.08 (8.0%). Therefore all Long Island mean daily flows prior to 1981 were increased by 8.0% to represent instantaneous peak flows.

Similar to the Ottawa data analysis, an additional adjustment was carried out to allow for the change in the Poonamalie structure operation. The mean ratio of the Poonamalie and Long Island flows was 15.8% for the period 1972-1976 and 7.6% for the period 1977-1986 as depicted in Table 10. The difference is approximately 8%, therefore, the post 1976 year flows at Long Island were increased by 8%, reflecting the previous rule curve operation at Poonamalie.

The maximum instantaneous flow series for the 39-year record used for the statistical analysis is presented in Table 11.

The CFA88 computer model again identified one low flow outlier. Similarly to the Ottawa gauge analysis, four distributions were computed, which are summarized in Table 12. In addition, computer generated plots of the distributions are contained in Appendix B.

TABLE 9
RIDEAU RIVER BELOW MANOTICK, PEAK FLOWS

Year	Maximum Instantaneous		Maximum Mean Daily		Maximum Instantaneous (Maximum mean Daily x 1.08)	
	m³/sec	(cfs)	m³/sec	(cfs)	m³/sec	(cfs)
1986	189	(6670)	148	(5230)	189*	(6670)
1985	183	(6460)	178	(6290)	183*	(6450)
1984	247	(8720)	243	(8580)	247*	(8720)
1983	200	(7060)	163	(5760)	200*	(7060)
1982	303	(10700)	296	(10450)	303*	(10700)
1981	312	(11020)	298	(10520)	312*	(11020)
1980			385	(13600)	416	(14690)
1979			252	(8900)	272	(9610)
1978			361	(12750)	390	(13770)
1977			289	(10210)	312	(11020)
1976			456	(16100)	493	(17410)
1975			242	(8550)	262	(9250)
1974			272	(9610)	294	(10380)
1973			262	(9250)	283	(9990)
1972			236	(8330)	255	(9000)
1971			193	(6820)	209	(7380)
1970			159	(5610)	172	(6070)
1967			136	(4800)	147	(5190)
1968			201	(7100)	217	(7660)
1967			221	(7800)	239	(8440)
1966			176	(6210)	190	(6710)
1965			142	(5010)	153	(5400)
1964			127	(4490)	137	(4840)
1963			252	(8900)	272	(9610)
1962			198	(6990)	214	(7560)
1961			159	(5610)	172	(6070)
1960			318	(11230)	344	(12150)
1959			272	(9610)	294	(10380)
1958			215	(7590)	232	(8190)
1957			71	(2500)	77	(2720)
1956			215	(7590)	232	(8190)
1955			317	(11190)	343	(12110)
1954			249	(8790)	269	(9500)
1953			229	(8090)	247	(8720)
1952			246	(8690)	266	(9390)
1951			275	(9710)	297	(10490)
1950			319	(11260)	345	(12180)
1949			266	(9390)	287	(10130)
1948			236	(8330)	255	(9000)

* Recorded maximum instantaneous values.

TABLE 10
EVENT BASED COMPARISON OF
ANNUAL MAXIMUM MEAN DAILY FLOWS AT
POONAMALIE AND LONG ISLAND

Date	Poonamalie m^3/s	Poonamalie (cfs)	Long Island m^3/s	Long Island (cfs)	Ratio (%)
1986 March 21	15.5	(550)	148	(5230)	10.47
1985 March 15	26.1	(920)	178	(6290)	14.66
1984 April 7	0.0	(0)	243	(8580)	0.00
1983 May 5	44.8	(1580)	163	(5760)	27.48
1982 April 1, 2	11.7	(410)	296	(10450)	3.95
1981 Feb 24	35.1	(1240)	298	(10520)	11.78
1980 March 22	2.3	(80)	385	(13600)	0.60
1979 March 25, 26	3.4	(120)	252	(8900)	1.35
1978 April 14	21.2	(750)	361	(12750)	5.87
1977 March 15	0.0	(0)	289	(10210)	0.00
1976 March 28	55.8	(1970)	456	(16100)	12.24
1975 April 20	31.4	(1110)	242	(8550)	12.98
1974 April 6	55.4	(1950)	272	(9610)	20.37
1973 March 18	30.9	(1090)	262	(9250)	11.79
1972 April 21	51.8	(1830)	236	(8330)	21.95

Pre 1977

12.24
12.98
20.37
11.79
21.95

79.33 ÷ 5 = 15.87

Post 1976

10.47
14.66
0.00
27.48
3.95
11.78
0.60
1.35
5.87
0.00

76.18 ÷ 10 = 7.62

% Difference = 15.87 ÷ 7.62 = 8.25, say 8.0%

TABLE 11

RIDEAU RIVER BELOW MANOTICK
MAXIMUM INSTANTANEOUS FLOWS USED FOR SINGLE STATION FREQUENCY ANALYSIS

Year	Month	FLOWS* m³/sec	(cfs)
1986	3	204	(7200)
1985	3	198	(6990)
1984	4	267	(9430)
1983	5	216	(7630)
1982	4	327	(11550)
1981	2	337	(11900)
1980	3	449	(15860)
1979	3	294	(10380)
1978	4	421	(14870)
1977	3	337	(11900)
1976	3	493	(17410)
1975	4	262	(9250)
1974	4	294	(10380)
1973	3	283	(9990)
1972	4	255	(9000)
1971	4	209	(7380)
1970	4	172	(6070)
1969	4	147	(5190)
1968	3	217	(7660)
1967	4	239	(8440)
1966	3	190	(6710)
1965	12	153	(5400)
1964	3	137	(4840)
1963	4	272	(9610)
1962	3	214	(7560)
1961	3	172	(6070)
1960	4	344	(12510)
1959	4	294	(10380)
1958	3	232	(8190)
1957	3	77	(2720)
1956	4	232	(8190)
1955	4	343	(12110)
1954	4	269	(9500)
1953	3	247	(8720)
1952	4	266	(9390)
1951	4	297	(10490)
1950	4	345	(12180)
1949	3	287	(10130)
1948	3	255	(9000)

* Increased flows after 1976 by 8 percent reflects Old Operating Rule (Pre 1977)

TABLE 12
RIDEAU RIVER BELOW MANOTICK
COMPARISON OF STATISTICAL DISTRIBUTIONS

Average Return Period - Years	Statistical Distribution			
	GEV	3PLN	LPIII	Wakeby
2	255	255	255	258
5	326	327	325	319
10	370	372	369	366
20	411	414	410	416
50	460	466	460	488
100	495	504	497	547

CS = 0.010
CK = 3.249

Note: Data base has one low flow outlier. All flows are instantaneous peaks in m^3/sec.

It is recommended that the flows predicted by the three-parameter lognormal distribution be accepted for use. The three-parameter lognormal distribution yielded a good fit in that the computed coefficients of skew (CS) and kurtosis (CK) of the transformed data are very close to the theoretical values of 0.0 and 3.0 respectively. In addition, this distribution yielded the highest predicted flows (excluding the Wakeby) which provides a relatively conservative estimate.

The predicted 100-year flow of 504 m^3/s can be compared with a 100 year value of 408 m^3/s presented in the 1976 James F. MacLaren Ltd. "Rideau River Floodline Mapping" report. The reason for such a significant difference in predicted design flows is somewhat due to the inclusion of additional years of flow data to the original sample base since 1976. Of the eleven years of additional flow data included, three flow values exceeded the previous predicted 100-year high value of 408 m^3/s.

In addition, the MacLaren flow series was developed from correlating Long Island Flows with flows available at the Ottawa gauge in order to extend the period of record 33 years back to 1916. It is difficult to assess whether or not this approach in itself is responsible for a lower estimate of flow, but rather demonstrates that a different flow series was used. The current Dillon estimate was based on a flow series with a sufficient number of local recorded flows; thus, there was no need for a synthetic (or composite) data base.

Therefore, for downstream of Long Island, immediately upstream of the confluence of Jock River and the Rideau River the following flows were adopted for use.

TABLE 13
RIDEAU RIVER BELOW MANOTICK
DESIGN FLOWS

Average Return Period - Years	Three-Parameter Lognormal Flow m^3/sec
2	255
5	327
10	372
20	414
50	466
100	504

3.2.3 Jock River Near Richmond

The Jock River with a drainage area of 559 km² is the largest tributary in the Rideau River system. It maintains the single largest lateral inflow into the Rideau River between Long Island and the downstream study limit at Hog's Back.

Observations on flows have been collected by Water Survey of Canada since 1970. In total, 17 years of maximum mean daily flows have been collected with only eight years of maximum instantaneous being recorded as shown in Table 14.

To account for the missing years of maximum instantaneous flow record the following procedure was adopted. The years of maximum instantaneous flow record that were present were compared to their corresponding maximum daily peaks. Comparison of the two quantities showed that the ratio varies between 1.01 and 1.11 with an average of 1.04 (4%). Therefore all Jock River mean daily flows, where the corresponding maximum instantaneous flows were absent, were increased by 4.0% to complete the entire record.

The maximum instantaneous flow series for the 17 year record used for the statistical analysis is presented in Table 14.

TABLE 14
JOCK RIVER NEAR RICHMOND, PEAK FLOWS (02LA007)

Year	Month	Maximum Instantaneous		Maximum Mean Daily		Maximum Instantaneous (Maximum mean Daily x 1.04)	
		m³/sec	(cfs)	m³/sec	(cfs)	m³/sec	(cfs)
1986	5	65	(2300)	62	(2190)	65*	(2300)
1985	3			59	(2080)	61	(2170)
1984	4	120	(4240)	118	(4170)	120*	(4240)
1983	3			50	(1770)	52	(1840)
1982	4			76	(2680)	79	(2790)
1981	2	111	(3920)	108	(3810)	111*	(3920)
1980	3			103	(3640)	107	(3780)
1979	3			114	(4030)	119	(4190)
1978	4	148	(5230)	133	(4700)	148*	(5230)
1977	3			117	(4130)	122	(4300)
1976	4	140	(4940)	137	(4840)	140*	(4940)
1975	4	123	(4340)	122	(4310)	123*	(4340)
1974	3			79	(2790)	82	(2900)
1973	3			119	(4200)	124	(4370)
1972	4			136	(4800)	141	(4990)
1971	4	116	(4100)	112	(3960)	116*	(4100)
1970	4	125	(4410)	121	(4270)	125*	(4410)

* Recorded maximum instantaneous values.

The CFA88 computer model identified that the sample was absent of low outliers. Four distributions were computed, as summarized in Table 15, with their corresponding graphical representations presented in Appendix C1.

TABLE 15
JOCK RIVER
COMPARISON OF STATISTICAL DISTRIBUTIONS

Average Return Period - Years	Statistical Distribution			
	GEV	3PLN	LPIII	Wakeby
2	111	114	109	116
5	135	133	135	130
10	146	140	147	135
20	154	145	156	142
50	161	149	164	158
100	164	152	169	177

CS = 1.116
CK = 3.165

Note: Data base was free of low outliers.
All flows are instantaneous peaks in m^3/sec.

No single distribution provides a superior fit, and all of the 100 year flows predicted are within 15% of one another. Due to the relatively short period of record it is difficult to select any one single distribution to predict the 100 year flood peak with any great degree of confidence purely from the frequency analysis. As a result, for the purpose of this examination the flows predicted by the Log Pearson Type III distribution were preferred, rather than the three-parameter lognormal, which would give the lowest flow.

As a comparison, a regional frequency analysis was conducted utilizing regression equations recently developed and recommended by the Ministry of Natural Resources, the results of which are tabulated in Table 16 and detailed in Appendix C2.

In addition, for comparison purposes, the flows predicted in the 1981 Acres Ltd. "Jock River Floodplain Mapping" report are included in Table 16.

TABLE 16
JOCK RIVER
COMPARISON OF PAST AND PRESENT FLOW PREDICTIONS

	1981 Acres Study	1987 Dillon Study	
Average Return Period - Years	Localized Regional Frequency Analysis m^3/sec	Single Station Frequency Analysis m^3/sec	MNR Regression Equations m^3/sec
2	96	109	135
5	124	135	143
10	141	147	147
20	156	156	150
50	174	164	166
100	187	169	178

From the comparison, it can be noted that the 100-year flows predicted from the various methods are all within 15% of one another. This variance, although not statistically insignificant, is difficult to compare with a "measured benchmark" due to the shortness of the recorded gauged data.

Nevertheless, both the single station frequency analysis and the use of the MNR regression equations confirm that the flows predicted in the 1981 Acres study are indeed representative.

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* Flood Plain Management in Ontario, Technical Guidelines.

For the purpose of establishing the Jock River flows, the flows derived in the 1981 Acres study were adopted for use since their derivation was based on an in-depth localized regional frequency analysis.

TABLE 17
JOCK RIVER
DESIGN FLOWS

Average Return Period - Years	Maximum Instantaneous Flow m^3/sec
2	96
5	124
10	141
20	156
50	174
100	187

3.3 Flows from Hog's Back to Below Jock-Rideau Confluence

An event based comparison of the 17 years of maximum daily flow record for the Jock River and corresponding Long Island flows was conducted to gain a better understanding of the relative timing of the peak flows at the confluence. Of the 17 years of record available for comparison, coincident peak flows (i.e. same day) were found in 10 years with the remaining seven years demonstrating peaks coinciding within an additional two days of one another.

It is therefore reasonable to assume that peak flows on the Jock River occur at approximately the same time as the flows peak on the Rideau River, downstream of Long Island.

From a second examination of the 10 years of coincident peak flows it was found that the numerical addition of the Jock River and the Long Island flood peaks for a given event resulted in a combined sum which accounted, on average, for approximately 93% of the total corresponding flood peaks recorded at Ottawa for the same event. This indicated that the peaks of the two hydrographs may not coincide exactly and/or the possibility of some minor lateral inflow occurs between the Jock River and Hog's Back locations.

It was therefore concluded that no additional investigations would be conducted on the remaining tributaries between the Jock-Rideau River confluence and Ottawa since their drainage area is small in comparison and their effects on the peak flood flows at Ottawa would be negligible.

Since the recommended peak flows for the different frequency events sum to a greater combined flow when added directly to the Long Island flows than is predicted at Ottawa, it appears that they may be somewhat high. In order to maintain the proper continuity upstream and downstream at the Jock-Rideau River confluence, it was decided that the Jock River flows not be altered, and that the flows recommended for the Ottawa gauge be maintained constant upstream to the Jock-Rideau River confluence.

3.4 Flows for East-West Branches Around Long Island

Due to the absence of any flow measuring stations on the east and west branches of the Rideau River at Long Island and upstream of Long Island itself, it is impossible to predict any flows from a purely hydrologic approach. Instead, the flows were determined during the hydraulic analysis by a trial and correction method.

This involved assuming an initial proportion of flow in each branch and running the hydraulic model for each branch separately from an initial starting water level to the upstream location where the branches rejoin. If the water level obtained at the upstream location was the same, the original assumption of flow proportion was deemed correct, if not, the proportion of flows was adjusted and the process repeated until a coinciding water level was obtained.

Flows were proportioned for all the selected return periods assuming both the Long Island Dam and Manotick Dam maintained an operational configuration with all stoplogs removed and the hydraulic gates fully open.

The results of the flow split around the east and west branches of Long Island are presented in Table 18.

TABLE 18
PROPORTION OF FLOWS FOR EAST-WEST BRANCHES AROUND LONG ISLAND

Average Return Period - Years	Rideau River Flow (m^3/sec)	West Branch Flow (m^3/sec)	West Branch Level* (m)	East Branch Flow (m^3/sec)	East Branch Level* (m)
2	255	130	85.56	125	85.59
5	327	163	85.85	164	85.89
10	372	186	86.04	186	86.03
20	414	205	85.18	209	86.16
50	466	228	86.36	238	86.35
100	504	244	86.47	260	86.51

* Level as established by HEC-2 computer model at Section 20.200.

As evident from this analysis an approximate 50/50 flow split was calculated for all return period flows.

3.5 Transposition of Flows Above Long Island to Upstream Study Limit

To develop a suitable estimate of flows at the upstream study limit in the absence of any measured flow data at this location, a flow-area relationship was employed. Flow-area relationships are often applied in transposing known flow data from a gauging location to other locations within the watershed where measured data is unavailable.

The recommended design flows above Long Island were transposed to the upstream study limit (Regional Road 6 below Kars) by using the following relationship.

$$\left(\frac{Q1}{Q2}\right) = \left(\frac{A1}{A2}\right)^{x}$$

Where Q1 = Design Flow at A1
Q2 = Design Flow at A2

A1 = 3,830 km^2
A2 = 3,120 km^2

x = exponent

For example, by utilizing the recommended 100 year design flow at Ottawa and Long Island of 654 m^3/sec and 504 m^3/sec respectively, and their corresponding drainage areas, the value for x was found to be 1.27.

Repeating the same procedure using this calculated value of x, the 100 year flow at Long Island and the corresponding drainage areas at Long Island and Kars (2,840 km^2), a 100 year flood flow of 447 m^3/sec was computed for Kars.

Similarly, this procedure was carried out for the all return periods.

The results of the analysis are presented in Table 19.

TABLE 19
TRANSPOSITION OF FLOWS ABOVE LONG ISLAND TO UPSTREAM STUDY LIMIT

Average Return Period - Years	Ottawa Flows (m^3/s)	Long Island Flows (m^3/s)	Flow Exponent x	Kars Flow (m^3/s)	Reduction (%)
2	445	255	2.72	197	29
5	513	327	2.20	266	23
10	552	372	1.92	311	20
20	586	414	1.69	353	17
50	626	466	1.44	407	15
100	654	504	1.27	447	13

Flows between Long Island and the upstream study limit were then proportioned (i.e. reduced) with distance along the remaining 8.5 km channel.

3.6 Summary of Design Flood Flows

The adopted design flood flows for the Rideau River from the downstream study limit at Hog's Back to the upstream study limit below Kars (Regional Road 6) are depicted in Figure 4.

4. HYDRAULIC ANALYSIS

4.1 General

The purpose of the hydraulic analysis was to establish the water surface profiles along the Rideau River for the various design events (i.e. 2 to 100-year events). Flood level profiles for these events were computed for the entire length of the study reach, from downstream at the Hog's Back Dam to the upstream limit at the Regional Road 6 bridge near Kars, including both branches around Long Island.

The results of the analysis provides information regarding flooding problems through the existing developed areas, and identifies the extent of the Regulatory Flood Plain.

The water surface profile computations were conducted using the most current version of the HEC-2 computer program. Developed by the U.S. Army Corps of Engineers, HEC-2 is a well documented, non-proprietary program which has become the most widely applied modelling technique for flood plain mapping purposes.

The flood lines produced by the Regulatory Storm (100-year) are plotted on the 18 flood risk maps which form part of this report. The area encompassed by these flood lines represents the flood prone lands where construction and/or placing of fill should be regulated.

4.2 HEC-2 Computer Modelling

The HEC-2 computer program is designed to simulate the steady-state response of river/watercourse systems, where uniform or gradually varied flow prevails. These flow conditions occur along moderately sloped channels, where the in-system storage is not significant relative to the flood hydrograph. The flood levels are, therefore, controlled primarily by the peak inflows, the bed slope and the influence of hydraulic structures along the stream.

The HEC-2 program can be applied along both natural and man-made channels, and can account for the energy losses caused by hydraulic structures, such as bridges, weirs, culverts, embankments, etc.

Along open channels, the one dimensional energy equation is solved using the standard step method, with energy losses due to friction evaluated by Manning's Formula. The losses associated with flow expansion and contraction, due to the non-uniformity of the watercourse geometry, are based on the variation in velocity from cross-section to cross-section.

The energy losses created by hydraulic structures are computed in two steps. First, the losses due to contraction and expansion on the upstream and downstream side of the structure are calculated; and then the losses through the structure are computed by one of two subroutines incorporated into the model: the normal or special bridge subroutine.

The former treats the bridge section in the same manner as an open river section, and is particularly applicable for bridges without piers, bridges under high submergence and for flow through culverts. The special bridge subroutine determines losses through structures for low flow, weir flow and pressure flow, or for any combination of these. A detailed description of the HEC-2 program is contained in the users manual, "HEC-2 Water Surface Profiles", and in "Training

Document No. 6". Both documents are published by the U.S. Army Corps of Engineers.

A brief description of the program input requirements and the data used for this study is provided in the following.

4.2.1 Watercourse Definition

A basic input requirement is the accurate description of the channel and flood plain, and the details of all structures along the watercourse. Cross-sections are normally obtained at representative locations throughout the watercourse reach to accurately define the physical characteristics of the conveyance system. These locations include; where there is an appreciable change in cross-sectional area, roughness and bed slope, and at all structures. In general, more cross-sections are needed to define energy losses in urban areas as opposed to rural areas; where steep slopes are encountered; and on smaller streams.

All hydraulic structures represent a potential obstruction to flow, which may produce a pronounced effect on flood levels. Therefore, the physical dimensions and elevations of all structures are needed. For bridges and culverts, the required data includes the flow area of the waterway opening, the size and shape of any piers, the elevation of the structure invert and soffit, and the roadway grade along the crossing.

For the Rideau River study a total of 175 cross-sections were used in the hydraulic modelling of the study reach. Cross-sectional information to adequately describe the geometry of both the channel and overbanks was comprised of both below waterline and above waterline information.

Waterline refers to that water level represented on the topographical mapping developed from the aerial photography.

Below waterline cross-sections were determined from sounding data obtained from the Canadian Hydrographic Service, Department of Fisheries and Oceans. The sounding data recorded in the summer of 1970 is stored on long paper rolls and is presented as a series of continuous sonar tracings. The sonar tracings present a detailed picture of the channel configuration relative to the water surface established on the day of survey.

The procedure adapted for transferring the tracings to an input format suitable for the hydraulic model involved the following procedure. A suitable number of points that would sufficiently and accurately describe the channel configuration was selected - usually in the order of 10-20 points to describe a channel width of 100-300 m.

From an examination of the survey field notes that accompany each sonar tracing, the width applicable for each cross-section was established. This involved subtracting the distances that the boat was from shore (this was contained in the field notes that accompany the sounding rolls) from the channel top width measured from waterline to waterline. Since the flood risk mapping was based on aerial photography exposed just prior to the beginning of navigation season when water levels were near their navigation levels (as was the conditions at the time of the depth sounding survey), it was concluded that the measured channel width was representative of actual conditions.

The width applicable for each sonar tracing was then divided by the number of points used to describe the channel in order to establish an even spacing for the depth readings read from the sounding rolls. Any abnormalities in the channel bottom such as peaks or valleys not identified by the established spacing were also included.

The depths were then converted to elevations by referencing them to the reduced chart datum (i.e. corrected water level) established on the day of survey for a specified controlled reach.

From a comparison of isolated sounding data conducted by Environment Canada in November of 1987, the cross-sections developed from the 1970 CHS survey (and used throughout the HEC-2 model) were confirmed as being representative and accurate.

Above waterline information to supplement the below waterline data was abstracted from the 1:2000 scale topographic mapping, the accuracy of which was confirmed in a separate field investigation (see Section 8, Topographic Mapping).

In addition, changes to ground contours since 1985 (date of aerial photography) were acknowledged in the abstraction of above waterline data from the map sheets. This was accomplished through a review of approved fill application permits 1985-1986 supplied by RVCA. The coding of the hydraulic cross-sections takes into account (where possible), the presence of fill and/or buildings. Although the mapping itself has not been amended and the plotted flood line does not account for changes in the horizontal extent, it was felt that any increases in water level due to the removal of potential flood plain storage would be compensated for.

A list of the relevant sections and corresponding treatment was sent to the Authority under separate cover.

4.2.2 Flow Regime and Boundary Conditions

The HEC-2 computer program is capable of computing water surface profiles for either subcritical or supercritical flow regimes. The former occurs along channels with mild gradients and/or where obstructions such as bridges/culverts create backwater effects.

When subcritical flow prevails, control is exerted by downstream conditions. To determine the water surface profile, calculations begin at the downstream limit of the watercourse and proceed upstream in a step-wise fashion, from cross-section to cross-section.

Supercritical flow (not present in the study reach) occurs along relatively steep channels, and control is exerted by upstream conditions. The same computational procedure is applied for this type of flow regime, except that the calculations are initiated at the upstream limit and proceed downstream. For both flow conditions a starting water level must be specified.

Cross-sections developed by A.J. Robinson and Associates Ltd. for the 1984 study, were input into the current model in order to define the boundary conditions immediately downstream of the Hog's Back Dam. Critical depth was assumed as the starting water level at the initial cross-section (as identified in the 1984 study) and occurred again through the Hog's Back Dam structure modelled with all stop logs removed and the hydraulic gates fully open.

4.2.3 Energy Loss Coefficients

The quantification of several hydraulic coefficients are necessary to carry out water surface profile computations; these include:

- Manning's "n" value to determine friction losses;
- contraction and expansion coefficients to evaluate transition or shock losses;
- bridge loss cofficients.

Manning's "n" is an indicator of the flow resistance exerted by the channel and flood plain, based on vegetation and channel roughness properties. These factors were determined from a field inspection, analysis of aerial photographs and the guidelines provided in Chow (1959). Coefficients were assigned to the main channel and the two overbank areas for each section.

A value of 0.03 was selected for the natural stream channel, and 0.08 for both overbank areas.

The contraction and expansion losses are accounted for in HEC-2 by multiplying the absolute difference in velocity head between successive cross-sections by a coefficient. The selection of the appropriate value for this factor was based on the values suggested in the users manual prepared by the U.S. Army Corps of Engineers.

A respective value of 0.5 and 0.3 was assigned to the expansion and contraction factor.

The HEC-2 program basically employs three different computational methods to assess the energy losses due to structures. The procedure selected depends on the type of flow conditions which occur at the structure, i.e. low flow, pressure flow, weir flow, or a combination of these. A loss coefficient is required for each of the three types of flow conditions. The friction losses associated with low flow are determined using an appropriate Manning "n" coefficient for the bridge/culvert material. The factor for pressure flow combines the friction losses through the structure with entrance and exit losses, and a coefficient of discharge is required to account for the losses produced when flow overtops the roadway.

All bridge and control structures throughout the study reach operate under low flow conditions for all return periods flows. Bridge hydraulic tables and photographs of all structures have been included in Appendix D.

4.2.4 Design Flood Flows

The required data consist of peak flood flows at various locations along the river/watercourse.

Water surface profiles were carried out for all the events for which flows were established: the 2, 5, 10, 20, 50, and 100-year events. The flow values derived as part of the hydrologic analysis, described in Section 3 of report, were used for the hydraulic analysis.

4.2.5 Control Structures

All flood level elevations computed from the computer model were established by modelling the four control structures (i.e. Hog's Back, Black Rapids, Long Island and Manotick Dams) with all stop logs removed and the hydraulic gates fully open. This assumption is consistent with the standard operating rule for the Rideau River study reach as discussed in Section 4.4.

4.3 Calibration

The following three calibration exercises (two quantitative analyses conducted from measured events and one qualitative analysis conducted from visual examination of aerial photography) were conducted to assess the relative accuracy of water levels predicted by the HEC-2 model.

In addition, a comparison of water levels (obtained and supplied by RVCA) above Long Island to Beckett's Landing is provided.

The points of calibration for the HEC-2 watercourse model include daily water level readings recorded at three of the four control structures, (Hog's Back Dam, Black Rapids Dam and Long Island Dam), a stage-discharge curve for the Long Island gauge station 02LA010 below Manotick (developed by WSC) and observed water levels recorded at the Regional Road 8 Bridge located on the west branch.

Water level records are kept for the three control structures throughout the summer navigation season, but are collected only on a random basis for Hog's Back Dam and Black Rapids Dam throughout the spring months. As a result, calibration of a large spring flood event on any continuous basis throughout the study reach is impossible without supplementary field measurements.

Since a great deal of variation in the Rideau River flows and levels is possible, both spatially and temporally, and since this is compounded by the existence of man-made control structures and features, such as the Long Island flow split, it is important that water measurements (for both levels and flows) be continued during spring events for future confirmation (or improvement) of current flooding predictions. Further areas of emphasis would include both branches of the Long Island flow split and the long flat reach extending from above Long Island to Kars.

4.3.1 October 2, 1986 Event

The approach adopted for the basis of calibration involved the selection of a flood peak which occurred on a date that provided sufficient recorded water levels at the three control structures. Following discussions with personnel from the Rideau Canal Office, a flood peak occuring on the 2 October, 1986 was selected for the model calibration.

The flood peak, although lower than the predicted mean annual flow, occurred at the end of the navigation season with the river channel near capacity. Since the purpose of the calibration is to adjust the Manning's roughness coefficient, and since the 100-year flood peak is contained almost entirely in channel, it was decided that using the low flood peak with the channel near capacity would be sufficient for and representative of conditions associated with the 100-year flood.

The calibration was conducted maintaining the Manning's roughness coefficient of 0.03 for the channel throughout the study reach. Results indicated a close agreement of calculated water levels to that of observed as shown in Table 20.

TABLE 20

CALIBRATION OF HEC-2 COMPUTER MODEL

HOG'S BACK DAM TO LONG ISLAND

OCTOBER 2, 1986 FLOWS

Location	Observed Flow (m^3/s)	Observed Water Level (m)		Calculated Water Level (m)	Relevant HEC-2 Cross-Section No.
Hog's Back Dam	185[1]	uppersill	74.95	74.95*	0.200
Black Rapids Dam					
(below)	185	lowersill	75.08	75.34	6.615
(above)	177[2]	uppersill	78.04	78.04*	6.650
Long Island Dam	137[3]	lowersill	78.46	78.46	14.305
WSC Gauge 02LA010 at Long Island	137		80.08	80.02	15.350
Regional Road 8 Bridge Gauge (west branch)	68[4]		82.26	82.27	18.490
Manotick Dam	68	lowersill	N/A	82.40	18.716
(west branch)	68	uppersill	N/A	84.88	18.795
Long Island Dam (east branch)	69[5]	uppersill	85.47	85.47*	15.425

* Assumed starting water level for next controlled reach upstream.

[1] Obtained from Carleton University Gauge flow.

[2] Obtained from summation of Long Island Gauge flow (137 m^3/s) and Jock River Gauge flow (40 m^3/s) from WSC records.

[3] Obtained from subtraction of Jock River flow (40 m^3/s).

[4] Obtained from HEC-2 computer model through balancing of water level upstream of Long Island (at Section 22.002) using known stop log configuration at Manotick Dam.

[5] Obtained from HEC-2 computer model through balancing of water level upstream of Long Island (at Section 22.002).

A difference in water level of 0.26 m was encountered downstream of Black Rapids Dam when observation was compared to calculated levels for the October 2 flow. In order to identify the nature of this difference, two points were investigated. Firstly, the model was rerun with successive changes to Manning's "n" until the observed water level of 78.04 m was matched. The resulting roughness coefficient required was found to be 0.015, a value uncharacteristic of natural channels.

Secondly, the model was run using a second event, October 8, 1986 with an observed starting flow of 138 m^3/s and level of 74.95 m. The observed water level at the lower sill of Black Rapids was 75.10 m while the computed level was found to be 75.16 m, a difference of only 0.06 m.

With the above two points in mind, it was decided that the model was predicting water levels representative of existing conditions with no further calibration or adjustment required. In addition, it was assumed that the difference in water levels could be attributable to small errors in observation reading or localized wind setup effects.

Upon conversation with the project team members, it was decided that further calibration of the model should be carried out from the reach extending from Long Island (Manotick) to the upstream study limit. This was prompted primarily due to an absence of water level data within this reach. The aquisition of additional data was to be conducted in the following Spring of 1988.

4.3.2 March 27, 1988 Event

From utilization of water level data collected and supplied by RVCA for the March 27, 1988 flow, results of additional calibration runs indicated a close agreement of calculated vs. observed water levels at locations on both the east and west branches around Long Island, immediately upstream of Long Island and at the upstream study limit at the Regional Road 6 bridge.

The calibration was conducted once again by maintaining the Manning's roughness coefficient of 0.03 for the channel throughout the study reach. Results indicated a close agreement of calculated water levels to that of observed except for the Mahogany Harbour location as shown in Table 21A. At Mahogany Harbour, a 0.19 m difference between measured and computed water levels is present. One explanation for such a discrepancy may lie in the knowledge that during the 26-27th of March, the operation of the Manotick Dam was altered; stop logs were removed resulting in a 0.40 m drop in water level from 85.20 m to 84.80 m.

Aside from the Mahogany Harbour location, generally the water level differences were small (i.e. approximately 0.05 m), and it was concluded that the model was predicting water levels representative of existing conditions with no further calibration or adjustment required.

4.3.3 March 29, 1976 Event

From utilization of aerial photography taken during flooding that occurred on March 29, 1976, the RVCA staff conducted a comparison of the extent of flooding shown on the 29 March, 1976 photos (peak flow on 28 March, 1976 of 597 m^3/sec at

TABLE 21A

CALIBRATION OF HEC-2 COMPUTER MODEL

LONG ISLAND TO REGIONAL ROAD 8

MARCH 27, 1988

Location	Observed Flow (m^3/s)	Observed Water Level (m)		Calculated Water Level (m)	Relevant HEC-2 Cross-Section No.
WSC Gauge 02LA010 at Long Island	209[1]		80.32	80.32*	15.350
Manotick Dam (West Branch)	109[2]	lowersill	N/A	82.78	18.716
	109	uppersill	N/A	84.29	18.795
Mahogany Harbour (West Branch)	109		84.80	84.99	19.360
Long Island Dam (East Branch)	100[3]	uppersill	84.77	84.77*	15.425
Regional Road 8 Bridge (East Branch)	100		85.25	85.26	18.185
Boat House above Long Island	209		85.44	85.40	20.515
Kellys Landing	209		85.68	85.73	23.780
Regional Road 6 Bridge	209		85.84	85.90	28.729

* Assumed starting water level for next controlled reach upstream.

[1] Obtained from WSC Gauge 02LA010 at Long Island.

[2] Obtained from HEC-2 computer model through balancing of water level upstream of Long Island (at Section 22.002) using known stop log configuration at Manotick Dam.

[3] Obtained from HEC-2 computer model through balancing of water level upstream of Long Island (at Section 22.002).

Ottawa) to the extent of flooding under the 20-year flood conditions (586 m^3/sec at Ottawa) as predicted by the HEC-2 model. The Authority concluded that the observed event and the modelled event compare favourably. Thus, the aerial photography does appear to provide some corroboration of the modelling results.

While it is acknowledged that the air photos are not as precise as field measured water level data, it is felt that they are useful for providing a basis for visual comparison of actual flooding to predicted flooding.

4.3.4 March 26 - April 6, 1988; Long Island to Becketts Landing

From utilization of water level data collected and supplied by RVCA for March 26 - April 6, 1988, a comparison of water levels above Long Island to Becketts Landing was made. Becketts Landing is approximately 15 km upstream of the Regional Road 6 Bridge located near Kars, and is equipped with a continuous water level gauge.

It was felt that an examination of water levels throughout this reach would yield a better understanding of the reach's characteristics and provide a basis for comparison of levels predicted by HEC-2 at the upstream study limit.

The measured water levels for the Long Island to Becketts reach are contained in Table 21B.

The difference in water level between the Kellys Landing and Becketts Landing reach is extremely small; generally less than 0.2 m decreasing to as little as 0.1 m, considering the

TABLE 21B

COMPARISON OF WATER LEVELS ABOVE LONG ISLAND TO BECKETTS LANDING MARCH, APRIL, 1988

	Percival B. House		Kellys Landing		Regional Road 6 Bridge (Kars Bridge)		Becketts Landing	
Date	Time	Level	Time	Level	Time	Level	Time	Level
03 26	13:30	85.27	13:45	85.41	-	-	12:00 13:00 14:00	85.33 85.35 85.37
03 27	16:26	85.44	15:23	85.68	15:15	85.84	15:00 16:00 17:00	85.77 85.77 85.80
03 28	9:20	85.37	9:35	85.60	-	-	-	-
03 29	16:15	85.37	16:00	85.48	-	-	-	-
04 05	13:47	85.38	14:04	85.51	-	-	13:00 14:00 15:00	85.67 85.67 85.66
04 06	15:35	85.29	14:45	85.45	14.15	85.55	14:00 15:00 16:00	85.62 85.62 85.62

length of the reach is approximately 20 km. This certainly demonstrates the presence of the long flat reach that is present above Long Island, extending to Becketts.

Noted discrepancies include higher observed water levels at Kellys Landing (26th) and Kars Bridge (27th) than observed upstream at Becketts. Explanations for which may include; inaccurate reading of the staff gauges, localized wind and/or wave setup effects and variations in the timing of observations combined with changes in dam operations (i.e. opening of gates, removal of stop logs).

Comparison of the HEC-2 model predictions for the March 27, 1988 observations indicates that the model may have slightly overpredicted water levels at the upstream study limit (by approximately 0.06 m). In light of the fact that both flows and levels were altered due to changes in dam operations during the period of observation and noting the other sources for discrepancies, it was concluded that no further changes in the modelling parameters (i.e. Mannings n, velocity coefficients, etc.) would be made in order to "force" a condition of matching water levels at the Kars Bridge location.

4.4 Sensitivity Analysis

Water levels predicted by the HEC-2 model were tested as to their sensitivity to various changes in flow. Simulations were conducted in which the 100-year flow was altered by ±5%, ±10% and ±15% in order to evaluate the resulting changes in water level.

The results are discussed below both in terms of vertical differences in water levels as well as any potential impacts on the horizontal extent of the 100-year flood line.

± 5% Flow Change

Simulation of the ±5 change in flows results in a corresponding change in water level of approximately ±0.1 m. This difference in water level represents a negligible change to the Regulatory Flood Line (i.e. <5 m lateral shift) for the study reach extending from Hog's Back to upstream of Long Island. Sensitivity of the flood line becomes more apparent in the reach extending from Long Island to the upstream study limit. Here horizontal changes can be as much as 20 m due to the flatter topography aligning the channel.

± 10% Flow Change

Simulation of the ±10% change in flows results in a corresponding change in water level of approximately ± 0.2 m. This difference in water level is generally negligible (<5 m) in the lower portions of the study reach, but becomes more evident towards the upper portion approaching the Regional Road 6 Bridge. Changes in the Regulatory Flood Line can be as much as 40 m in this location.

± 15% Flow Change

Simulation of the ±15% change in flow results in a corresponding change in water level of approximately ± 0.3 m. Although generally a visible change in the Flood Line is evident (<10 m lateral shift) it is minor in the Hog's Back to Long Island reach. For locations above Long Island, the water level becomes the most sensitive to changes in flow, with horizontal shifts in the Flood Line as much as 60 m.

In summary, it can be noted that the HEC-2 model is sensitive to variances in design flows. Results of sensitivity testing demonstrated a ± 0.1 m change in water level for each ±5% change in flow and that these changes in water levels were maintained for the entire study reach. The impact of the variances in water level upon the Regulatory(100-year) Flood Line are generally minor and could be assumed negligible in locations of steep bank height, but can be very evident in locations subject to flatter topography.

The results of the sensitivity analysis are tabulated in Tables 22A and 22B.

4.5 Dam Operations

From communications with personnel from the Rideau Canal office the following briefly describes the regulation of water levels at the four control structures.

Generally, navigation levels are maintained from May to mid-October when the beginning of non-navigation season occurs. Early in November a procedure of drawing down the river system is initiated by the removal of stop logs at each structure. The resulting drop in water level provides some operational flexibility as the freshet approaches.

A period of flood watch begins in February and continues through to April and May during spring melt. In addition to regular ice jam monitoring and blasting operations by the

TABLE 22A

1:100 YEAR WATER LEVEL SENSITIVITY TO CHANGES IN FLOW

HOG'S BACK DAM TO KARS BRIDGE

INCLUDES WEST BRANCH AROUND LONG ISLAND

Cross Section Number (Refer to Flood Risk Maps)	100-YEAR STORM		+5% Flow Change		-5% Flow Change		+10% Flow Change		-10% Flow Change		+15% Flow Change		-15% Flow Change	
	Flow (m³/s)	Elevation (m)	Flow (m³/s)	Elevation (m)	Flow (m³/s)	Elevation (m)	Flow (m³/s)	Elevation (m)	Flow (m³/s	Elevation (m)	Flow (m³/s)	Elevation (m)	Flow (m³/s)	Elevation (m)
HOG'S BACK DAM														
0.105	654	74.98	687	75.08	621	74.87	719	75.18	589	74.76	752	75.28	556	74.64
0.200	654	75.17	687	75.28	621	75.05	719	75.39	589	74.93	752	75.50	556	74.81
0.380	654	75.17	687	75.28	621	75.05	719	75.39	589	74.93	752	75.50	556	74.81
0.510	654	75.18	687	75.30	621	75.07	719	75.41	589	74.95	752	75.52	556	74.82
0.825	654	75.20	687	75.31	621	75.08	719	75.42	589	74.96	752	75.53	556	74.83
1.150	654	75.20	687	75.32	621	75.08	719	75.42	589	74.96	752	75.53	556	74.83
1.285	654	75.20	687	75.32	621	75.08	719	75.42	589	74.96	752	75.53	556	74.83
1.415	654	75.20	687	75.32	621	75.08	719	75.42	589	74.96	752	75.53	556	74.83
1.545	654	75.20	687	75.32	621	75.08	719	75.42	589	74.96	752	75.53	556	74.83
1.690	654	75.25	687	75.37	621	75.14	719	75.47	589	75.03	752	75.57	556	74.91
1.815	654	75.55	687	75.65	621	75.44	719	75.74	589	75.34	752	75.84	556	75.23
1.950	654	75.84	687	75.93	621	75.74	719	76.03	589	75.64	752	76.12	556	75.54
2.085	654	76.06	687	76.16	621	75.96	719	76.26	589	75.86	752	76.36	556	75.76
2.290	654	76.26	687	76.36	621	76.16	719	76.46	589	76.06	752	76.56	556	75.95
2.440	654	76.35	687	76.45	621	76.24	719	76.55	589	76.14	752	76.65	556	76.03
2.780	654	76.38	687	76.49	621	76.28	719	76.59	589	76.17	752	76.69	556	76.06
3.065	654	76.49	687	76.60	621	76.38	719	76.70	589	76.28	752	76.81	556	76.16
3.335	654	76.53	687	76.64	621	76.42	719	76.74	589	76.31	752	76.84	556	76.20
3.535	654	76.58	687	76.68	621	76.47	719	76.78	589	76.34	752	76.89	556	76.25
CANADIAN NATIONAL RAILWAY BRIDGE														
3.750	654	76.69	687	76.80	621	76.58	719	76.90	589	76.47	752	77.01	556	76.36
4.015	654	76.94	687	77.06	621	76.82	719	77.17	589	76.70	752	77.29	556	76.57
4.320	654	77.00	687	77.12	621	76.88	719	77.24	589	76.76	752	77.35	556	76.63
4.555	654	77.00	687	77.12	621	76.88	719	77.24	589	76.76	752	77.35	556	76.63
4.810	654	77.01	687	77.13	621	76.88	719	77.24	589	76.76	752	77.36	556	76.63
4.980	654	77.05	687	77.17	621	76.93	719	77.29	589	76.80	752	77.41	556	76.67
HUNT CLUB BRIDGE														
5.205	654	77.06	687	77.19	621	77.94	719	77.30	589	76.82	752	77.42	556	76.68
5.560	654	77.10	687	77.22	621	76.97	719	77.34	589	76.85	752	77.46	556	76.71
5.940	654	77.15	687	77.28	621	77.02	719	77.40	589	76.90	752	77.52	556	76.76
6.210	654	77.21	687	77.33	621	77.07	719	77.46	589	76.95	752	77.58	556	76.81
6.430	654	77.22	687	77.35	621	77.09	719	77.47	589	76.96	752	77.59	556	76.82
6.560	654	77.22	687	77.35	621	77.09	719	77.47	589	76.96	752	77.59	556	76.82
6.615	654	77.22	687	77.35	621	77.09	719	77.47	589	76.96	752	77.59	556	76.82
BLACK RAPIDS DAM														
6.755	654	79.02	687	79.09	621	78.95	719	79.15	589	78.88	752	79.22	556	78.80
6.955	654	79.04	687	79.11	621	78.96	719	79.18	589	78.89	752	79.25	556	78.81
7.260	654	79.08	687	79.15	621	79.01	719	79.22	589	78.93	752	79.29	556	78.85
7.500	654	79.19	687	79.26	621	79.11	719	79.34	589	79.03	752	79.41	556	78.94
7.725	654	79.26	687	79.34	621	79.18	719	79.42	589	79.09	752	79.50	556	79.00
7.915	654	79.27	687	79.36	621	79.19	719	79.44	589	79.11	752	79.52	556	79.02
8.060	654	79.29	687	79.37	621	79.21	719	79.45	589	79.12	752	79.54	556	79.03
8.245	654	79.31	687	79.39	621	79.22	719	79.47	589	79.14	752	79.56	556	79.05
8.325	654	79.32	687	79.40	621	79.23	719	79.48	589	79.15	752	79.57	556	79.06
8.400	654	79.33	687	79.41	621	79.24	719	79.49	589	79.16	752	79.58	556	79.06
8.590	654	79.33	687	79.41	621	79.24	719	79.49	589	79.16	752	79.58	556	79.06
8.840	654	79.34	687	79.43	621	79.26	719	79.51	589	79.17	752	79.59	556	79.08
8.960	654	79.37	687	79.46	621	79.29	719	79.54	589	79.20	752	79.62	556	79.11
9.200	654	79.40	687	79.48	621	79.31	719	79.57	589	79.22	752	79.65	556	79.13
9.410	654	79.42	687	79.51	621	79.33	719	79.59	589	79.24	752	79.67	556	79.14
9.665	654	79.46	687	79.56	621	79.37	719	79.64	589	79.28	752	79.73	556	79.18
9.860	654	79.47	687	79.56	621	79.38	719	79.65	589	79.29	752	79.74	556	79.19
9.955	654	79.48	687	79.57	621	79.39	719	79.66	589	79.29	752	79.75	556	79.19
10.055	654	79.49	687	79.58	621	79.39	719	79.67	589	79.30	752	79.76	556	79.20
10.105	654	79.51	687	79.60	621	79.41	719	79.69	589	79.32	752	79.78	556	79.22
10.365	654	79.52	687	79.62	621	79.43	719	79.70	589	79.33	752	79.79	556	79.23
10.575	654	79.54	687	79.63	621	79.44	719	79.72	589	79.35	752	79.81	556	79.25
10.895	654	79.56	687	79.66	621	79.47	719	79.75	589	79.37	752	79.84	556	79.27
11.215	654	79.59	687	79.69	621	79.49	719	79.78	589	79.40	752	79.87	556	79.29
11.480	654	79.61	687	79.71	621	79.52	719	79.80	589	79.42	752	79.90	556	79.31
11.795	654	79.65	687	79.75	621	79.55	719	79.84	589	79.45	752	79.94	556	79.34
12.100	654	79.68	687	79.78	621	79.57	719	79.87	589	79.47	752	79.97	556	79.37
12.315	654	79.70	687	79.80	621	79.60	719	79.89	589	79.50	752	79.99	556	79.39
12.510	654	79.70	687	79.80	621	79.60	719	79.89	589	79.50	752	79.99	556	79.39
12.685	654	79.74	687	79.83	621	79.64	719	79.93	589	79.54	752	80.02	556	79.43
12.855	654	79.94	687	80.05	621	79.83	719	80.15	589	79.73	752	80.26	556	79.61
13.045	654	79.95	687	80.05	621	79.84	719	80.15	589	79.73	752	80.25	556	79.61
13.255	654	80.18	687	80.30	621	80.07	719	80.40	589	79.95	752	80.51	556	79.83
13.465	654	80.21	687	80.33	621	80.09	719	80.43	589	79.98	752	80.54	556	79.85
13.730	654	80.27	687	80.38	621	80.15	719	80.50	589	80.03	752	80.61	556	79.90

TABLE 22A

1:100 YEAR WATER LEVEL SENSITIVITY TO CHANGES IN FLOW

HOG'S BACK DAM TO KARS BRIDGE

INCLUDES WEST BRANCH AROUND LONG ISLAND

(continued)

Cross Section Number (Refer to Flood Risk Maps)	100-YEAR STORM Flow (m³/s)	100-YEAR STORM Elevation (m)	+5% Flow Change Flow (m³/s)	+5% Flow Change Elevation (m)	-5% Flow Change Flow (m³/s)	-5% Flow Change Elevation (m)	+10% Flow Change Flow (m³/s)	+10% Flow Change Elevation (m)	-10% Flow Change Flow (m³/s)	-10% Flow Change Elevation (m)	+15% Flow Change Flow (m³/s)	+15% Flow Change Elevation (m)	-15% Flow Change Flow (m³/s)	-15% Flow Change Elevation (m)
JOCK RIVER TRIBUTARY														
13.920	504	80.29	529	80.41	479	80.17	554	80.52	453	80.05	580	80.63	428	79.93
14.060	504	80.29	529	80.41	479	80.17	554	80.52	453	80.05	580	80.63	428	79.93
14.210	504	80.41	529	80.53	479	80.29	554	80.64	453	80.17	580	80.75	428	80.04
14.305	504	80.47	529	80.58	479	80.34	554	80.70	453	80.22	580	80.81	428	80.09
14.400	504	80.47	529	80.58	479	80.34	554	80.70	453	80.22	580	80.81	428	80.09
14.625	504	80.51	529	80.63	479	80.39	554	80.74	453	80.27	580	80.85	428	80.14
14.875	504	80.69	529	80.81	479	80.56	554	80.93	453	80.43	580	81.05	428	80.29
15.080	504	80.72	529	80.85	479	80.60	554	80.97	453	80.47	580	81.09	428	80.33
15.190	504	80.72	529	80.85	479	80.60	554	80.97	453	80.47	580	81.09	428	80.33
15.260	504	80.72	529	80.85	479	80.60	554	80.97	453	80.47	580	81.09	428	80.33
WSC GAUGE BELOW MANOTICK														
15.350	504	81.74	529	81.82	479	81.66	554	81.91	453	81.57	580	81.99	428	81.48
15.420	244	82.11	259	82.20	234	82.01	272	82.29	222	81.91	285	82.39	210	81.81
15.600	244	82.25	259	82.35	234	82.16	272	82.44	222	82.06	285	82.53	210	81.96
15.740	244	82.37	259	82.46	234	82.28	272	82.55	222	82.18	285	82.65	210	82.08
BARNSDALE ROAD														
16.177	244	82.97	259	83.08	234	82.88	272	83.18	222	82.78	285	83.28	210	82.68
16.510	244	83.23	259	83.35	234	83.15	272	83.45	222	83.04	285	83.55	210	82.94
16.860	244	83.36	259	83.84	234	83.27	272	83.58	222	83.16	285	83.69	210	83.06
17.040	244	83.39	259	83.51	234	83.30	272	83.62	222	83.19	285	83.72	210	83.08
17.375	244	83.46	259	83.58	234	83.37	272	83.69	222	83.26	285	83.79	210	83.15
17.595	244	83.50	259	83.63	234	83.41	272	83.73	222	83.30	285	83.84	210	83.19
17.785	244	83.53	259	83.66	234	83.44	272	83.76	222	83.33	285	83.87	210	83.22
17.975	244	83.53	259	83.66	234	83.44	272	83.76	222	83.33	285	83.85	210	83.22
18.270	244	83.83	259	83.95	234	83.74	272	84.05	222	83.63	285	84.15	210	83.53
18.490	244	84.16	259	83.29	234	84.07	272	84.40	222	83.97	285	84.51	210	83.86
MANOTICK BRIDGE														
18.716	244	84.31	259	84.43	234	84.22	272	84.54	222	84.11	285	84.65	210	84.00
MANOTICK DAM														
18.885	244	85.38	259	85.48	234	85.31	272	85.57	222	85.22	285	85.65	210	85.14
19.025	244	85.61	259	85.71	234	85.54	272	85.80	222	85.46	285	85.88	210	85.37
19.360	244	85.90	259	86.01	234	85.83	272	86.10	222	85.74	285	86.19	210	85.65
19.530	244	85.94	259	86.05	234	85.87	272	86.15	222	85.78	285	86.24	210	85.69
19.690	244	85.94	259	86.05	234	85.87	272	86.15	222	85.78	285	86.24	210	85.69
19.815	244	86.00	259	86.11	234	85.93	272	86.20	222	85.84	285	86.28	210	85.76
19.945	244	86.22	259	86.33	234	86.15	272	86.42	222	86.06	285	86.51	210	85.98
20.090	244	86.41	259	86.52	234	86.34	272	86.61	222	86.25	285	86.71	210	86.16
EAST AND WEST BRANCHES COMBINED														
20.200	504	86.51	529	86.59	479	86.41	554	86.67	453	86.31	580	86.77	428	86.22
20.515	504	86.56	529	86.63	479	86.45	554	86.72	453	86.36	580	86.81	428	86.27
20.890	504	86.65	529	86.72	479	86.54	554	86.82	453	86.45	580	86.91	428	86.35
21.115	498	86.67	523	86.74	473	86.56	543	86.84	448	86.46	573	86.93	423	86.37
21.275	498	86.68	523	86.75	473	86.57	543	86.85	448	86.47	573	86.94	423	86.38
21.505	498	86.73	523	86.81	473	86.62	543	86.90	448	86.52	573	87.00	423	86.42
21.765	498	86.74	523	86.82	473	86.62	543	86.91	448	86.53	566	87.01	423	86.43
22.015	492	86.74	517	86.82	467	86.62	541	86.91	443	86.53	566	87.01	418	86.43
22.350	492	86.92	517	87.01	467	86.81	541	87.10	443	86.71	566	87.20	418	86.61
22.840	492	87.07	517	87.16	467	86.95	541	87.27	443	86.85	555	87.37	418	86.74
23.130	492	87.10	517	87.19	467	86.98	541	87.30	443	86.87	555	87.40	418	86.76
23.400	483	87.10	507	87.19	459	86.98	531	87.30	435	86.87	555	87.40	411	86.76
23.615	483	87.20	507	87.30	459	87.08	531	87.40	435	86.97	555	87.50	411	86.86
23.780	483	87.23	507	87.32	459	87.11	531	87.42	435	87.00	555	87.53	411	86.89
24.000	483	87.30	507	87.40	459	87.18	531	87.50	435	87.07	549	87.61	411	86.96
24.165	483	87.37	507	87.47	459	87.25	531	87.58	435	87.14	549	87.69	411	87.02
24.350	477	87.39	501	87.49	453	87.26	525	87.59	429	87.15	549	87.70	405	87.03
24.560	477	87.43	501	87.54	453	87.31	525	87.65	429	87.19	549	87.76	405	87.07
24.680	477	87.44	501	87.54	453	87.31	525	87.65	429	87.20	542	87.76	405	87.08
25.110	477	87.48	501	87.59	453	87.36	525	87.70	429	87.24	542	87.81	405	87.12
25.300	471	87.49	495	87.59	447	87.36	518	87.71	424	87.25	542	87.82	400	87.12
25.500	471	87.50	495	87.60	447	87.37	518	87.71	424	87.25	542	87.82	400	87.13
25.810	471	87.50	495	87.61	447	87.38	518	87.72	424	87.26	535	87.83	400	87.14
26.060	471	87.51	495	87.62	447	87.38	518	87.73	424	87.27	535	87.84	400	87.14
26.230	465	87.51	488	87.62	442	87.39	512	87.73	419	87.27	535	87.84	395	87.15
26.545	465	87.53	488	87.64	442	87.40	512	87.75	419	87.28	528	87.86	395	87.16

TABLE 22A

1:100 YEAR WATER LEVEL SENSITIVITY TO CHANGES IN FLOW

HOG'S BACK DAM TO KARS BRIDGE

INCLUDES WEST BRANCH AROUND LONG ISLAND

(continued)

Cross Section Number (Refer to Flood Risk Maps)	100-YEAR STORM		+5% Flow Change		-5% Flow Change		+10% Flow Change		-10% Flow Change		+15% Flow Change		-15% Flow Change	
	Flow (m^3/s)	Elevation (m)	Flow (m^3/s)	Elevation (m)	Flow (m^3/s)	Elevation (m)	Flow (m^3/s)	Elevation (m)	Flow (m^3/s	Elevation (m)	Flow (m^3/s)	Elevation (m)	Flow (m^3/s)	Elevatio (m)
EAST AND WEST BRANCHES COMBINED														
26.830	465	87.54	488	87.65	442	87.41	512	87.76	419	87.29	535	87.88	395	87.17
27.160	459	87.55	482	87.66	436	87.42	505	87.78	413	87.31	528	87.89	390	87.18
27.540	459	87.57	482	87.67	436	87.44	505	87.79	413	87.32	528	87.90	390	87.19
27.870	459	87.59	482	87.70	436	87.46	505	87.81	413	87.34	528	87.93	390	87.21
28.075	453	87.60	476	87.71	430	87.47	498	87.82	408	87.34	521	87.94	385	87.22
28.245	453	87.60	476	87.71	430	87.47	498	87.83	408	87.35	521	87.94	385	87.22
28.435	453	87.61	476	87.72	430	87.48	498	87.83	408	87.35	521	87.95	385	87.23
KARS BRIDGE (REGIONAL ROAD 8)														

TABLE 22B

1:100 YEAR WATER LEVEL SENSITIVITY TO CHANGES IN FLOW

EAST BRANCH AROUND LONG ISLAND

Cross Section Number (Refer to Flood Risk Maps)	100-YEAR STORM		+5% Flow Change		-5% Flow Change		+10% Flow Change		-10% Flow Change		+15% Flow Change		-15% Flow Change	
	Flow (m^3/s)	Elevation (m)	Flow (m^3/s)	Elevation (m)	Flow (m^3/s)	Elevation (m)	Flow (m^3/s)	Elevation (m)	Flow (m^3/s	Elevation (m)	Flow (m^3/s)	Elevation (m)	Flow (m^3/s)	Elevation (m)
LONG ISLAND DAM														
15.570	260	85.68	270	85.77	245	85.55	282	85.86	231	85.43	295	85.97	218	85.32
15.770	260	85.68	270	85.77	245	85.55	282	85.86	231	85.43	295	85.97	218	85.32
16.030	260	85.68	270	85.77	245	85.56	282	85.87	231	85.43	295	85.97	218	85.32
16.130	260	85.68	270	85.77	245	85.56	282	85.87	231	85.43	295	85.97	218	85.32
16.430	260	85.69	270	85.77	245	85.56	282	85.87	231	85.44	295	85.98	218	85.33
16.515	260	85.69	270	85.77	245	85.56	282	85.87	231	85.44	295	85.98	218	85.33
16.820	260	85.72	270	85.80	245	85.59	282	85.90	231	85.47	295	86.00	218	85.36
17.170	260	85.78	270	85.87	245	85.66	282	85.96	231	85.54	295	86.07	218	85.43
17.460	260	85.85	270	85.93	245	85.73	282	86.03	231	85.62	295	86.13	218	85.51
17.720	260	85.91	270	85.99	245	85.79	282	86.09	231	85.67	295	86.19	218	85.56
OLD WHITEHORSE DAM														
18.135	260	86.29	270	86.36	245	86.19	282	86.44	231	86.10	295	86.53	218	86.02
MANOTICK BRIDGE - EAST BRANCH														
18.325	260	86.35	270	86.43	245	86.25	282	86.51	231	86.16	295	86.60	218	86.08
18.695	260	86.39	270	86.46	245	86.28	282	86.54	231	86.19	295	86.64	218	86.11
18.870	260	86.40	270	86.47	245	86.29	282	86.56	231	86.20	295	86.65	218	86.12
19.180	260	86.43	270	86.50	245	86.33	282	86.59	231	86.23	295	86.68	218	86.15
19.490	260	86.44	270	86.51	245	86.33	282	86.59	231	86.24	295	86.69	218	86.15
19.715	260	86.50	270	86.57	245	86.39	282	86.66	231	86.29	295	86.75	218	86.21
19.890	260	86.51	270	86.59	245	86.41	282	86.67	231	86.31	295	86.77	218	86.22
EAST AND WEST BRANCHES COMBINED														
20.200	504	86.51	529	86. 59	479	86.41	554	86.67	453	86.31	580	86.77	428	86.22

City of Ottawa a manned crew is on duty, ready to remove all of the remaining stop logs and open the hydraulic gates in advance of a major flood peak. After passage of a flood peak the logs are replaced and the procedure repeated if required.

The following table describes the operational ranges for the controlled water levels at the four structures.

TABLE 23
OPERATIONAL WATER LEVELS

Control Structure	Navigation Elevation (msl)	Non-Navigation Elevation (msl)
Hog's Back Dam	74.90 - 74.95	72.71
Black Rapids Dam	77.78 - 77.83	75.44
Long Island Dam	85.45 - 85.50	85.09 - 85.14
Manotick Dam	85.45 - 85.50	85.09 - 85.14

4.6 Results

4.6.1 General

The results of the hydraulic analysis are summarized in Tables 24A and 24B. The Regulatory Flood Lines are plotted on the accompanying Flood Risk Maps.

A comparison of water levels predicted herein to that of previously predicted flood levels (see Dillon, 1972) was conducted to assess the reasonableness of the current (or past) results.

Generally, the currently predicted 100-year results are higher than past predictions by approximately 0.3 m. As illustrated in the sensitivity analysis, changes in water level in this range do not generally result in a very noticeable change in the flood line for the Hog's Back to Long Island reach. For locations above Long Island, the flood line becomes more sensitive to changes in flow.

Some possible explanations for differences in the predicted 100-year flood levels presented herein may include:

- Extension of long-term flow records that resulted in increased flows predicted at Manotick (24%) and Kars (10%), compared to flow predictions in the 1976 MacLaren study.

- Increased improvements (i.e. accuracy) in hydraulic models and their application.

- Increased improvements in the photo mosaic mapping base resulting in a more accurate representation of the flood plain.

- Use of numerous below water cross-section obtained from hydrographic sounding surveys resulting in a more accurate representation of the river channel.

TABLE 24A

SUMMARY OF CALCULATED WATER SURFACE ELEVATIONS

HOG'S BACK DAM TO KARS BRIDGE - INCLUDES WEST BRANCH AROUND LONG ISLAND

Cross Section Number (Refer to Flood Risk Maps)	2-YEAR STORM		5-YEAR STORM		10-YEAR STORM		20-YEAR STORM		50-YEAR STORM		100-YEAR STORM	
	Flow (m^3/s)	Elevation (m)	Flow (m^3/s)	Elevation (m)	Flow (m^3/s)	Elevation (m)	Flow (m^3/s	Elevation (m)	Flow (m^3/s)	Elevation (m)	Flow (m^3/s)	Elevation (m)
HOG'S BACK DAM												
0.105	445	74.23	513	74.49	552	74.63	586	74.75	626	74.88	654	74.98
0.200	445	74.36	513	74.64	552	74.79	586	74.92	626	75.07	654	75.17
0.380	445	74.36	513	74.64	552	74.79	586	74.92	626	75.07	654	75.17
0.510	445	74.37	513	74.65	552	74.81	586	74.94	626	75.08	654	75.18
0.825	445	74.38	513	74.66	552	74.81	586	74.95	626	75.09	654	75.20
1.150	445	74.38	513	74.66	552	74.82	586	74.95	626	75.10	654	75.20
1.285	445	74.38	513	74.66	552	74.82	586	74.95	626	75.10	654	75.20
1.415	445	74.37	513	74.66	552	74.82	586	74.95	626	75.10	654	75.20
1.545	445	74.37	513	74.66	552	74.82	586	74.95	626	75.10	654	75.20
1.690	445	74.48	513	74.75	552	74.90	586	75.02	626	75.16	654	75.25
1.815	445	74.86	513	75.09	552	75.22	586	75.33	626	75.46	654	75.55
1.950	445	75.19	513	75.41	552	75.53	586	75.63	626	75.75	654	75.84
2.085	445	75.40	513	75.62	552	75.75	586	75.85	626	75.98	654	76.06
2.290	445	75.58	513	75.81	552	75.94	586	76.05	626	76.17	654	76.26
2.440	445	75.64	513	75.88	552	76.01	586	76.13	626	76.26	654	76.35
2.780	445	75.67	513	75.91	552	76.05	586	76.16	626	76.29	654	76.38
3.065	445	75.76	513	76.01	552	76.15	586	76.27	626	76.40	654	76.49
3.335	445	75.80	513	76.05	552	76.19	586	76.30	626	76.44	654	76.53
3.535	445	75.85	513	76.09	552	76.23	586	76.35	626	76.48	654	76.58
CANADIAN NATIONAL RAILWAY BRIDGE												
3.750	445	75.95	513	76.20	552	76.34	586	76.46	626	76.60	654	76.69
4.015	445	76.13	513	76.40	552	76.56	586	76.69	626	76.84	654	76.94
4.320	445	76.17	513	76.45	552	76.61	586	76.74	626	76.90	654	77.00
4.555	445	76.17	513	76.46	552	76.61	586	76.75	626	76.90	654	77.00
4.810	445	76.18	513	76.46	552	76.62	586	76.75	626	76.90	654	77.01
4.980	445	76.21	513	76.50	552	76.66	586	76.79	626	76.94	654	77.05
HUNT CLUB BRIDGE												
5.205	445	76.22	513	76.51	552	76.67	586	76.80	626	76.96	654	77.06
5.560	445	76.24	513	76.54	552	76.70	586	76.83	626	76.99	654	77.10
5.940	445	76.29	513	76.58	552	76.75	586	76.89	626	77.04	654	77.15
6.210	445	76.32	513	76.62	552	76.79	586	76.93	626	77.09	654	77.21
6.430	445	76.33	513	76.63	552	76.80	586	76.94	626	77.11	654	77.22
6.560	445	76.33	513	76.63	552	76.80	586	76.94	626	77.11	654	77.22
6.615	445	76.33	513	76.63	552	76.80	586	76.94	626	77.11	654	77.22
BLACK RAPIDS DAM												
6.755	445	78.54	513	78.72	552	78.79	586	78.87	626	78.96	654	79.02
6.955	445	78.55	513	78.74	552	78.80	586	78.89	626	78.97	654	79.04
7.260	445	78.58	513	78.77	552	78.84	586	78.93	626	79.02	654	79.08
7.500	445	78.65	513	78.86	552	78.93	586	79.02	626	79.12	654	79.19
7.725	445	78.70	513	78.91	552	78.99	586	79.09	626	79.19	654	79.26
7.915	445	78.71	513	78.92	552	79.00	586	79.10	626	79.20	654	79.27
8.060	445	78.73	513	78.94	552	79.02	586	79.12	626	79.22	654	79.29
8.245	445	78.74	513	78.95	552	79.04	586	79.13	626	79.24	654	79.31
8.325	445	78.74	513	78.96	552	79.04	586	79.14	626	79.25	654	79.32
8.400	445	78.75	513	78.97	552	79.05	586	79.15	626	79.25	654	79.33
8.590	445	78.75	513	78.97	552	79.05	586	79.15	626	79.25	654	79.33
8.840	445	78.77	513	78.98	552	79.07	586	79.17	626	79.27	654	79.34
8.960	445	78.79	513	79.01	552	79.10	586	79.19	626	79.30	654	79.37
9.200	445	78.81	513	79.03	552	79.12	586	79.22	626	79.32	654	79.40
9.410	445	78.82	513	79.04	552	79.13	586	79.23	626	79.34	654	79.42
9.665	445	78.85	513	79.07	552	79.17	586	79.27	626	79.39	654	79.46
9.860	445	78.85	513	79.08	552	79.18	586	79.28	626	79.39	654	79.47
9.955	445	78.86	513	79.09	552	79.18	586	79.29	626	79.40	654	79.48
10.055	445	78.86	513	79.09	552	79.19	586	79.29	626	79.41	654	79.49
10.105	445	78.88	513	79.11	552	79.21	586	79.31	626	79.43	654	79.51
10.365	445	78.89	513	79.12	552	79.22	586	79.33	626	79.44	654	79.52
10.575	445	78.90	513	79.13	552	79.23	586	79.34	626	79.46	654	79.54
10.895	445	78.92	513	79.15	552	79.26	586	79.36	626	79.48	654	79.56
11.215	445	78.94	513	79.18	552	79.28	586	79.39	626	79.51	654	79.59
11.480	445	78.95	513	79.19	552	79.30	586	79.41	626	79.53	654	79.61
11.795	445	78.98	513	79.22	552	79.33	586	79.44	626	79.57	654	79.65
12.100	445	79.00	513	79.24	552	79.35	586	79.47	626	79.59	654	79.68
12.315	445	79.02	513	79.27	552	79.38	586	79.49	626	79.61	654	79.70
12.510	445	79.03	513	79.27	552	79.38	586	79.49	626	79.61	654	79.70
12.685	445	79.06	513	79.31	552	79.42	586	79.53	626	79.65	654	79.74
12.855	445	79.21	513	79.47	552	79.60	586	79.72	626	79.85	654	79.94
13.045	445	79.22	513	79.48	552	79.60	586	79.72	626	79.85	654	79.95
13.255	445	79.39	513	79.68	552	79.81	586	79.94	626	80.08	654	80.18
13.455	445	79.41	513	79.70	552	79.84	586	79.97	626	80.11	654	80.21
13.730	445	79.46	513	79.75	552	79.89	586	80.02	626	80.17	654	80.27

TABLE 24A

SUMMARY OF CALCULATED WATER SURFACE ELEVATIONS

HOG'S BACK DAM TO KARS BRIDGE - INCLUDES WEST BRANCH AROUND LONG ISLAND

(continued)

Cross Section Number (Refer to Flood Risk Maps)	2-YEAR STORM		5-YEAR STORM		10-YEAR STORM		20-YEAR STORM		50-YEAR STORM		100-YEAR STORM	
	Flow (m^3/s)	Elevation (m)	Flow (m^3/s)	Elevation (m)	Flow (m^3/s)	Elevation (m)	Flow (m^3/s	Elevation (m)	Flow (m^3/s)	Elevation (m)	Flow (m^3/s)	Elevation (m)
JOCK RIVER TRIBUTARY												
13.920	255	79.48	327	79.77	372	79.91	414	80.04	466	80.19	504	80.29
14.060	255	79.48	327	79.77	372	79.91	414	80.04	466	80.19	504	80.29
14.210	255	79.54	327	79.85	372	80.00	414	80.14	466	80.30	504	80.41
14.305	255	79.57	327	79.88	372	80.04	414	80.19	466	80.35	504	80.47
14.400	255	79.57	327	79.88	372	80.04	414	80.19	466	80.35	504	80.47
14.625	255	79.59	327	79.92	372	80.08	414	80.23	466	80.40	504	80.51
14.875	255	79.67	327	80.02	372	80.20	414	80.36	466	80.55	504	80.69
15.080	255	79.69	327	80.04	372	80.23	414	80.40	466	80.59	504	80.72
15.190	255	79.69	327	80.04	372	80.23	414	80.40	466	80.59	504	80.72
15.260	255	79.77	327	80.05	372	80.23	414	80.40	466	80.59	504	80.72
WSC GAUGE BELOW MANOTICK												
15.350	255	80.70	327	81.02	372	81.21	414	81.39	466	81.59	504	81.74
15.420	130	80.96	163	81.31	186	81.53	205	81.72	228	81.95	244	82.11
15.600	130	81.15	163	81.49	186	81.69	205	81.88	228	82.10	244	82.25
15.740	130	81.31	163	81.63	186	81.84	205	82.01	228	82.22	244	82.37
BARNSDALE DRIVE BRIDGE												
16.177	130	81.90	163	82.23	186	82.44	205	82.62	228	82.82	244	82.97
16.510	130	82.15	163	82.49	186	82.70	205	82.88	228	83.09	244	83.23
16.860	130	82.24	163	82.59	186	82.82	205	83.00	228	83.21	244	83.36
17.040	130	82.26	163	82.62	186	82.84	205	83.03	228	83.24	244	83.39
17.375	130	82.32	163	82.68	186	82.91	205	83.10	228	83.31	244	83.46
17.595	130	82.35	163	82.72	186	82.95	205	83.14	228	83.36	244	83.50
17.785	130	82.37	163	82.74	186	82.97	205	83.16	228	83.38	244	83.53
17.975	130	82.39	163	82.75	186	82.98	205	83.16	228	83.38	244	83.53
18.270	130	82.72	163	83.07	186	83.29	205	83.47	228	83.68	244	83.83
18.490	130	83.04	163	83.39	186	83.62	205	83.81	228	84.02	244	84.16
MANOTICK BRIDGE WEST BRANCH												
18.716	130	83.18	163	83.53	186	83.76	205	83.95	228	84.16	244	84.31
MANOTICK DAM												
18.885	130	84.51	163	84.78	186	84.96	205	85.10	228	85.27	244	85.38
19.025	130	84.74	163	85.02	186	85.20	205	85.34	228	85.50	244	85.61
19.360	130	84.97	163	85.28	186	85.47	205	85.62	228	85.79	244	85.90
19.530	130	85.00	163	85.31	186	85.50	205	85.65	228	85.83	244	85.94
19.690	130	85.00	163	85.31	186	85.50	205	85.65	228	85.83	244	85.94
19.815	130	85.10	163	85.39	186	85.58	205	85.72	228	85.89	244	86.00
19.945	130	85.31	163	85.60	186	85.79	205	85.94	228	86.11	244	86.22
20.090	130	85.49	163	85.78	186	85.97	205	86.12	228	86.30	244	86.41
EAST AND WEST BRANCHES COMBINED												
20.200	255	85.59	327	85.89	372	86.04	414	86.18	466	86.36	504	86.51
20.515	255	85.63	327	85.93	372	86.09	414	86.23	466	86.41	504	86.56
20.890	255	85.68	327	86.00	372	86.16	414	86.31	466	86.50	504	86.65
21.115	249	85.69	321	86.01	366	86.17	408	86.32	460	86.51	498	86.67
21.275	249	85.70	321	86.02	366	86.18	408	86.33	460	86.53	498	86.68
21.505	249	85.72	321	86.05	366	86.22	408	86.38	460	86.57	498	86.73
21.765	249	85.72	321	86.05	366	86.22	408	86.38	460	86.58	498	86.74
22.015	243	85.72	315	86.05	360	86.22	402	86.38	454	86.58	492	86.74
22.350	243	85.85	315	86.20	360	86.39	402	86.56	454	86.76	492	86.92
22.840	243	85.93	315	86.30	360	86.50	402	86.68	454	86.90	492	87.07
23.130	243	85.94	315	86.32	360	86.52	402	86.70	454	86.93	492	87.10
23.400	233	85.94	305	86.32	350	86.52	392	86.70	444	86.93	483	87.10
23.615	233	86.02	305	86.40	350	86.61	392	86.80	444	87.03	483	87.20
23.780	233	86.04	305	86.43	350	86.63	392	86.82	444	87.05	483	87.23
24.000	233	86.08	305	86.48	350	86.70	392	86.89	444	87.12	483	87.30
24.165	233	86.12	305	86.53	350	86.75	392	86.95	444	87.19	483	87.37
24.350	226	86.13	298	86.54	343	86.76	385	86.96	437	87.20	477	87.39
24.560	226	86.15	298	86.57	343	86.80	385	87.00	437	87.25	477	87.43
24.680	226	86.15	298	86.57	343	86.80	385	87.00	437	87.25	477	87.44
25.110	226	86.18	298	86.60	343	86.84	385	87.04	437	87.30	477	87.48
25.300	219	86.18	291	86.61	336	86.84	378	87.05	430	87.30	471	87.49
25.500	219	86.18	291	86.61	336	86.84	378	87.05	430	87.31	471	87.50
25.810	219	86.19	291	86.62	336	86.85	378	87.06	430	87.31	471	87.50
26.060	219	86.19	291	86.62	336	86.86	378	87.07	430	87.32	471	87.51
26.230	213	86.19	284	86.62	329	86.86	371	87.07	424	87.32	465	87.51
26.545	213	86.20	284	86.63	329	86.87	371	87.08	424	87.34	465	87.53

TABLE 24A

SUMMARY OF CALCULATED WATER SURFACE ELEVATIONS

HOG'S BACK TO KARS BRIDGE - INCLUDES WEST BRANCH AROUND LONG ISLAND

(continued)

Cross Section Number (Refer to Flood Risk Maps)	2-YEAR STORM		5-YEAR STORM		10-YEAR STORM		20-YEAR STORM		50-YEAR STORM		100-YEAR STORM	
	Flow (m^3/s)	Elevation (m)	Flow (m^3/s)	Elevation (m)	Flow (m^3/s)	Elevation (m)	Flow (m^3/s	Elevation (m)	Flow (m^3/s)	Elevation (m)	Flow (m^3/s)	Elevation (m)
EAST AND WEST BRANCHES COMBINED (Continued)												
26.830	213	86.20	284	86.64	329	86.88	371	87.09	424	87.35	465	87.54
27.160	207	86.21	277	86.65	322	86.89	364	87.10	418	87.36	459	87.55
27.540	207	86.22	277	86.66	322	86.90	364	87.11	418	87.37	459	87.57
27.870	207	86.22	277	86.67	322	86.91	364	87.13	418	87.39	459	87.59
28.075	201	86.23	271	86.67	316	86.92	358	87.14	412	87.40	453	87.60
28.245	201	86.23	271	86.67	316	86.92	358	87.14	412	87.40	453	87.60
28.435	201	86.23	271	86.68	316	86.92	358	87.14	412	87.41	453	87.61
KARS BRIDGE (REGIONAL ROAD 6)												

TABLE 24B

SUMMARY OF CALCULATED WATER SURFACE ELEVATIONS

EAST BRANCH AROUND LONG ISLAND

Cross Section Number (Refer to Flood Risk Maps)	2-YEAR STORM		5-YEAR STORM		10-YEAR STORM		20-YEAR STORM		50-YEAR STORM		100-YEAR STORM	
	Flow (m^3/s)	Elevation (m)	Flow (m^3/s)	Elevation (m)	Flow (m^3/s)	Elevation (m)	Flow (m^3/s	Elevation (m)	Flow (m^3/s)	Elevation (m)	Flow (m^3/s)	Elevation (m)
LONG ISLAND DAM												
15.570	125	84.41	164	84.81	186	85.02	209	85.24	238	85.49	260	85.68
15.770	125	84.41	164	84.82	186	85.03	209	85.24	238	85.49	260	85.68
16.030	125	84.42	164	84.82	186	85.03	209	85.24	238	85.50	260	85.68
16.130	125	84.42	164	84.82	186	85.03	209	85.24	238	85.50	260	85.68
16.430	125	84.42	164	84.82	186	85.03	209	85.24	238	85.60	260	85.69
16.515	125	84.43	164	84.83	186	84.04	209	85.25	238	85.50	260	85.69
16.820	125	84.45	164	84.86	186	84.07	209	85.28	238	85.53	260	85.72
17.170	125	84.55	164	84.95	186	85.15	209	85.36	238	85.60	260	85.78
17.460	125	84.63	164	85.03	186	85.23	209	85.43	238	85.67	260	85.85
17.720	125	84.68	164	85.08	186	85.29	209	85.49	238	85.73	260	85.91
OLD WHITEHORSE DAM												
18.135	125	85.47	164	85.73	186	85.86	209	85.97	238	86.14	260	86.29
MANOTICK BRIDGE - EAST BRANCH												
18.325	125	85.49	164	85.77	186	85.91	209	86.02	238	86.20	260	86.35
18.695	125	85.52	164	85.80	186	85.94	209	86.05	238	86.24	260	86.39
18.870	125	85.53	164	85.81	186	85.95	209	86.07	238	86.25	260	86.40
19.180	125	85.54	164	85.83	186	85.97	209	86.09	238	86.28	260	86.43
19.490	125	85.55	164	85.83	186	85.98	209	86.10	238	86.28	260	86.44
19.715	125	85.58	164	85.87	186	86.02	209	86.15	238	86.34	260	86.50
19.890	125	85.59	164	85.88	186	86.04	209	86.17	238	86.36	260	86.51
EAST AND WEST BRANCHES COMBINED												
20.200	255	85.59	327	85.89	372	86.04	414	86.18	466	86.36	504	86.51

4.6.2 Description of Flooding

Flooding concerns and a description of the flood vulnerable areas are discussed according to four reaches. A summary of the number of structures within the 100-year flood plain is provided in Table 25.

Results indicate that several flood vulnerable areas exist throughout the Rideau River study reach. A total of 259 structures are potentially at risk due to the Regulatory Flood (i.e. "situated totally or partially within the 100-year flood plain").

i) Hog's Back to Black Rapids
Distance: 6.6 km Sheets: 1-4

No flooding of any residential structures is visible, although a total of five sheds/boathouses are affected somewhat. The 100-year flood level is contained almost entirely in the channel except in areas of low bank height, which results in an inundation of the overbank areas of approximately 100 m.

ii) Black Rapids to Downstream Long Island
Distance: 7.8 km Sheets: 4-9

Several flood vulnerable areas exist throughout this reach. The first of which lies immediately upstream of Black Rapids Dam. Eight structures (five cottages and three sheds/boathouses) are prone to minor flooding which begins at the 5-year return period.

TABLE 25
SUMMARY OF STRUCTURES PRONE TO FLOODING

Sheet No.	NUMBER OF STRUCTURES WITHIN THE 100-YEAR FLOOD PLAIN		
	Residential Structures	Ancillary Structures	Total Structures
1	-	-	-
2	-	2	2
3	-	2	2
4	5	3	8
5	8	8	16
6	-	3	3
7	5	7	12
8	1	1	2
9	9	5	14
10	-	1	1
11	1	-	1
12	-	2	2
13	2	7	9
14	4	6	10
15	12	8	20
16	15	10	25
17	40	27	67
18	40	25	65
	142	117	259

The second vulnerable area is situated approximately 500 m upstream of Black Rapids Dam. Flooding at this location begins at the 5-year peak flow and affects 11 structures (eight cottages/houses and three sheds at the 100-year level.

Continuing upstream through this reach the 100-year flood level is generally contained where steep banks provide sufficient elevation. Inundation of the flood plain of approximately 150 m is present in areas of flatter topography. As a result, the flooding of 17 structures that occurs through this section does so in scattered isolated areas leading up to Long Island.

A third flood vulnerable area is situated at the downstream end of Long Island. A total of 14 structures (nine houses/-cottages and five ancillary buildings) are situated within the 100-year flood plain.

iii) Downstream Long Island to Upstream Long Island
Distance: 5.8 km Sheets: 9-13

No serious flooding is encountered throughout this reach which includes both branches around Long Island. A total of nine structures are prone to minor flooding at the 100-year level which is entirely contained within the steep banks that line each side of the channel throughout this reach.

iv) Upstream Long Island to Regional Road 6
Distance: 8.8 km Sheets: 13-18

It is in this remaining reach that the most substantial flooding occurs. One such flood vulnerable area includes the east bank extending 1.6 km from Kilby Lane to the Manotick Marina. Thirty structures (19 residential and 11 ancillary buildings) are exposed to flooding at the 100-year level. Overbank flooding and inundation of structures begins at the 5-year level.

In the final 5 km of the reach, 60% of the total number of structures situated within the 100-year flood plain are present. Containing 157 buildings (95 residential and 62 ancillary structures) the flood vulnerable areas occupy both sides of the channel along Pine Avenue, Marina Drive and River Road, extending to Regional Road 6. Of the total, 18 residential dwellings are exposed to flooding due to a combination of overtopping of Marina Drive at the intersection of Fairway Drive and culvert back-up.

It should be noted that almost all of the structures susceptible to flooding at the 100-year flood level are exposed to flood depths of less than 0.6 m (2 ft.) and in many cases less than 0.3 m (1 ft.). With this in mind, the actual nature of the flood risk may be more accurately determined through site specific surveys of "borderline" (or marginal) cases of flood susceptible structures. Assessments of the need for or feasibility of floodproofing measures or of the appropriateness of renovation/reconstruction proposals will rely on the completion of such site specific surveys of building elevation and surrounding topography.

Flooding of many structures does not occur until exceedence of the 5-year return period level and at this level, although the number of structures affected is substantial, most of the structures are made up of sheds and boathouses, and the resulting damages could be assumed as being relatively minor.

4.7 Ice Conditions

Ice accumulations have been observed at various locations within the study reach. This reach include:

- the entrance to Mooney's Bay
- downstream of Black Rapids Dam
- downstream of Long Island Dam
- the confluence of the Jock-Rideau River
- Rideau Narrows

Generally, ice accumulations are self-clearing with little or no ice jamming occurring and potential for flooding. It should be noted that this is in contrast to the lower portion of the River (below the study limit) from Hog's Back to the outlet at the Ottawa River, where regular blasting operations are conducted to alleviate potential ice jams and the possibility of flooding.

Adverse ice formation is not regarded as a serious problem within the study range and as a result has not been considered as an influencing factor with the 100-year flow to dictate the Regulatory Flood level.

5. FILL LINE

5.1 General

The fill line is an administrative line, which defines the area over which the Authority has jurisdiction to restrict and/or prohibit development. It generally includes an area outside the flood line which:

i) may be susceptible to flooding and/or erosion problems;

ii) would increase flood and/or erosion problems if altered;

iii) is hazardous to development, or;

iv) may have detrimental effects on the environment if infilled.

The fill line does not preclude development or alteration, but is intended to be a warning signal that additional consideration may be warranted in order to ensure that the necessary precautions are implemented to eliminate any hazards.

Of prime interest to the Authority in the given study reach is to include within the fill lines all areas of potential instability (areas in which the indiscriminant placing of fill might serve to further reduce the stability of slopes).

This concern has been confirmed in a 1976 paper entitled "Slope Stability Study of the Regional Municipality of Ottawa - Carleton" (Klugman and Chung) that suggested that all slopes higher than 10 ft. (3 m) or with a grade of 1:4 [vertical:horizontal] or more should be examined for stability.

5.2 Fill Line Criteria

The specific criteria applied along the Rideau River to establish the fill line varies with location, and incorporates the results of the hydrologic/hydraulic analyses and information furnished by the RVCA. The general guidelines formulated to place the fill line are:

i) Bank Stability Concerns:

Condition	Criterion
• H less than 3 metres, flood line above top of bank	15 metre setback from flood line
• H less than 3 metres, flood line below top of bank	15 metre setback from top of bank
• H greater than 3 metres, i flatter than 4:1	15 metre setback from flood line
• H greater than 3 metres, i steeper than 4:1	farther from river of: - 15 metre setback from top slope, or - intersection of existing grade and imaginary 4:1 grade rising from toe of slope

Where: H = Bank Height
i = Slope Inclination [horizontal:vertical distance]

ii) For ease of location in the field, straight lines have been used whenever possible for the fill lines. Physical features such as fences, roads, etc. have been used where possible.

iii) Areas of swamp, environmentally sensitive areas and reaches susceptible to erosion or bank instability have been included within the fill line.

iv) Consideration has been given to excluding existing buildings from within the fill line wherever possible. Where flexibility as to the fill line location exists, the fill line has been located to the benefit of the landowners.

The resulting fill line along the Rideau River has been plotted on the accompanying Flood Risk Maps.

6. ALTERNATIVE FLOOD CONTROL MEASURES

6.1 General

The water surface profile calculations revealed that flooding generally initiates at the 5-year level. The primary cause of flooding is due to the inadequate capacity of the existing channel resulting from low bank height associated in areas of flat topography. In addition, some isolated areas are prone to flooding due to culvert and ditch back up.

A number of alternative remedial measures are available to reduce or eliminate future flood losses. Generally, all the alternatives belong to one of the three following categories:

i) Modify the flood.

ii) Modify the susceptibility to flooding.

iii) Modify the loss burden.

A description of the various flood control options for each category is provided in Table 26.

A preliminary examination of the various alternatives indicated that the three most practical and economically feasible solutions for the Rideau River study reach are:

i) Isolated berming in areas where land availability and drainage requirements permit and can be justified economically.

TABLE 26

ALTERNATIVES FOR REDUCING FLOOD LOSSES

GOAL	CORRECTIVE MEASURES: Structural	CORRECTIVE MEASURES: Non-Structural	PREVENTIVE MEASURES	OTHER
Modify the flood	- Dams and reservoirs levees or wall, channel improvements, stream diversion, storm drainage system.	- Watershed treatment meteorological modification, snow management prevention or removal of ice jams.		
Modify the susceptibility to damage	- Floodproofing, fill and/or elevate new structures, relocation.	- Evacuation and emergency flood fighting measures. Flood forecasting and warning systems. Urban redevelopment.	Regulations - to disclose flood hazard in real estate transactions; - for subdivision development; - for sanitary and health - for encroachment and fill Zoning by-laws. Building policies. Warning signs and education programs. Public and private purchase of open space for easements. Tax adjustments for open space.	
				- Flood insurance tax write-offs, relief and rehabilitation, protection and looting.

ii) Floodproofing of structures where development is scattered and the number of affected buildings is limited.

iii) Continued flood forecasting effort involving flow monitoring, snow and ice pack monitoring, etc. and implementation of a flood warning system through coordination with municipality officials.

As a precursor to the investigations into the possibility of flood damage reduction, the Authority should examine the prospect of quantifying average annual flood damages in the study reach. A brief discussion of procedures for estimating average annual flood damages is provided.

6.2 Flood Damage Reduction

Generally, a flood damage analysis is undertaken in order to determine the magnitude of a flooding problem and the extent of flood protection work that could be justified through a benefit-cost analysis. This analysis compares the net present value of benefits (reduction in flood damages) to costs for the structural and non-structural measures based on an assessment of the average annual damages discounted over the project life. The average annual damages are simply the area under a flood damage frequency curve which shows damages for selected return periods.

There are basically two methods to develop a damage-frequency curve. One uses damages recorded for real flood events, while the other uses synthetic flood elevation and depth-damages curves.

In the absence of any recorded damage figures for observed flood events, the procedure for estimating the total and expected average annual flood damages involves the following:

i) Determination of all structures flooded within the Regulatory Flood Line.

ii) Field reconnaissance survey of each structure identified in i) in order to determine:

- ° structure type and condition
- ° distance from ground to first flood
- ° distance from ground to invert of lowest opening
- ° address

iii) Selection of depth-damage curves that are representative of the various structure types identified in ii). The curves should represent both structural and content damages.

iv) Determination of flood elevations for selected return periods.

v) Determination of total damages (i.e., the summation of direct and indirect damages) by using the flood elevations determined in iv) and the depth-damages curves selected in iii).

vi) Calculation of the average annual damages by summation of the product of total damages and probability.

7. FLOOD PLAIN MANAGEMENT ALTERNATIVES

7.1 General

Current Flood Plain Management policy in the Province of Ontario encourages municipalities to incorporate flood plain lands into their official plans, together with appropriate policies to address new development.

In consultation with the local Conservation Authority or the Ministry of Natural Resources, where no Conservation Authority exists, municipalities should develop policies for inclusion into the Official plan which:

i) describe the flood susceptibility and risk associated with the flood plain areas;

ii) restrict new buildings or structures which are prone to flood damages or which may cause adverse impacts to existing development or lands;

iii) address additions or alterations to existing buildings or structures, and replacement of building or structures situated in the flood plain;

iv) describe the public and private works which may locate in the flood plain.

v) advise property owners located in the flood plain of the flooding implications, and inform them of alternative floodproofing measures which can be implemented.

Under proposed flood plain management criteria, there are two options which the Authority can adopt to identify and/or regulate flood plain areas, as described in the following:

i) Regulatory Flood Standard

As previously discussed, the standard for defining the regulatory flood plain limits in Eastern Ontario is based on the 100-year flood.

ii) Two Zone Concept

The two zone concept recognizes that there are two components to the flood plain: the floodway and the flood fringe. The former represents the more hazardous portion of the flood plain: the area which conveys the majority of the flow and where the highest velocities are experienced. New development in the floodway is to be prohibited or restricted.

The flood fringe represents the area of the flood plain outside the floodway, where generally shallow depths and low velocities prevail. New development may be permitted in the flood fringe, however, protection must be provided to the Regulatory Flood level.

The two zone concept cannot be applied selectively based on individual applications, but can be defined for entire reaches of watercourses, sub-catchments or watersheds with due consideration of local conditions.

7.2 Discussion

Under current Provincial policy in Eastern Ontario, the floodway is defined as the 1:100 year flood plain. In the case of the Rideau River the Regulatory Flood is based on the same 100-year criterion, therefore, at present, the two zone concept is not applicable for the Rideau River.

The proposed flood plain implementation guidelines released in July 1986 by MNR would permit the Conservation Authority to select its own criteria for the floodway. It is suggested, that the Authority carry out a review of the applicability of the two zone concept for the Rideau River based on the information presented on the flood plain maps, at such time as the proposed policies become the official policies of the Province of Ontario.

The review should consider the following:

1. Limit of the two zone concept to be applied: entire watershed or selective areas.

2. Criteria to be used for the definition of the floodway.

3. Type of development permitted in the Flood Fringe, new development, re-development, residential, recreational, etc.

4. Impact of future development on upstream and downstream riparian owners.

5. Floodproofing criteria for development in the Flood Fringe.

6. Method of implementation (MNR, C.A. Municipal roles).

8. TOPOGRAPHIC MAPPING

8.1 General

The topographic mapping, provided by Airmap Limited was produced from 1:8 000 scale aerial photography flown on April 26, 1985. In all, eighteen map sheets were completed to cover the 29 kilometre study reach. Flood risk mapping coverage and sheet layout is depicted in Figure 5. The maps are at a scale of 1:2 000, with a 1 m contour interval and a 0.5 m machine interpolated auxiliary contour. Prepared in conformity with the Ontario Base Mapping specifications, each map sheet has interior neat line dimensions of 75 cm by 75 cm and covers a maximum area of 225 hectares.

8.2 Ground Control

In accordance with the specifications for flood plain mapping procedures, field survey work was undertaken to confirm both the vertical and horizontal accuracy of the mapping. The procedure and criteria for evaluating the adequacy of the mapping, as described in the specification document (Schedule C) for flood plain mapping, is provided below.

8.2.1 Vertical Control

"Select ten spot elevations and ten identifiable contour crossings with roads, railways, etc., per map to be inspected. The contour crossings should be located in relatively flat and horizontal terrain (slope less than 5%).

The spot elevations and contour crossings should be evenly distributed throughout the map. Using existing bench marks as datum, compare the map elevation with the field elevation for the selected points.

The map meets the required accuracy standards if 90% of the spot elevations checked are within 1/3 of the contour interval and if 90% of the contour crossings are within 1/2 the contour interval of the map."

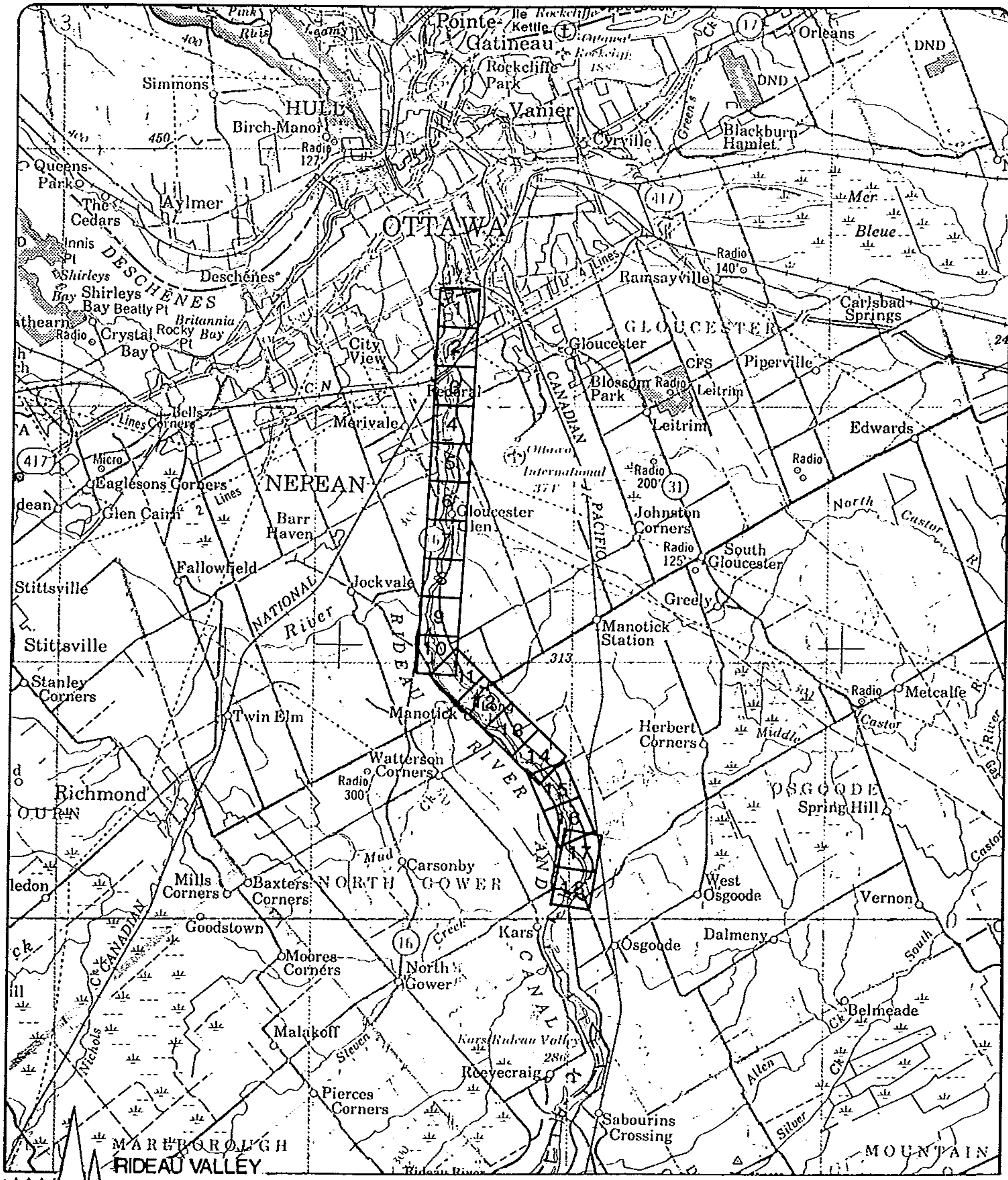
8.2.2 Horizontal Control

"Select three well-defined, identifiable and accessible features from the map. These three points should be at least 20 cm from each other at the scale of the map. Using monumented survey stations, establish the true position of the selected points by field survey methods.

The map meets the required accuracy standards if the map points are within a 0.5 mm (line map) or 1.0 mm (orthophotomap) radius of their true position."

8.3 Field Surveys

The vertical and horizontal control points to be field inspected were selected by the Technical Sub-Committee Canada - Ontario FDRP. For this project, map sheets Nos. 12 and 18 were chosen for field verification.
The field survey work was performed by McElhanney Mapping Services Ltd. of Nepean, Ontario, in June, 1987. The results of the investigations are contained in their report entitled "Flood Plain Mapping Check, Rideau River", August, 1987, confirmed that the mapping satisfies the required accuracy standards as outlined in Section 8.2.



CONSERVATION AUTHORITY

Rideau River

Flood Risk Mapping Study

FLOOD RISK MAPPING COVERAGE (AND SHEET LAYOUT)

DILLON

Figure 5

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Acres Consulting Services Limited, Jock River Flood Plain Mapping for the Rideau Valley Conservation Authority, November 1981.

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U.S. Army Corps of Engineers, Volume 6, Water Surface Profiles, The Hydrologic Engineering Centre, July 1975.

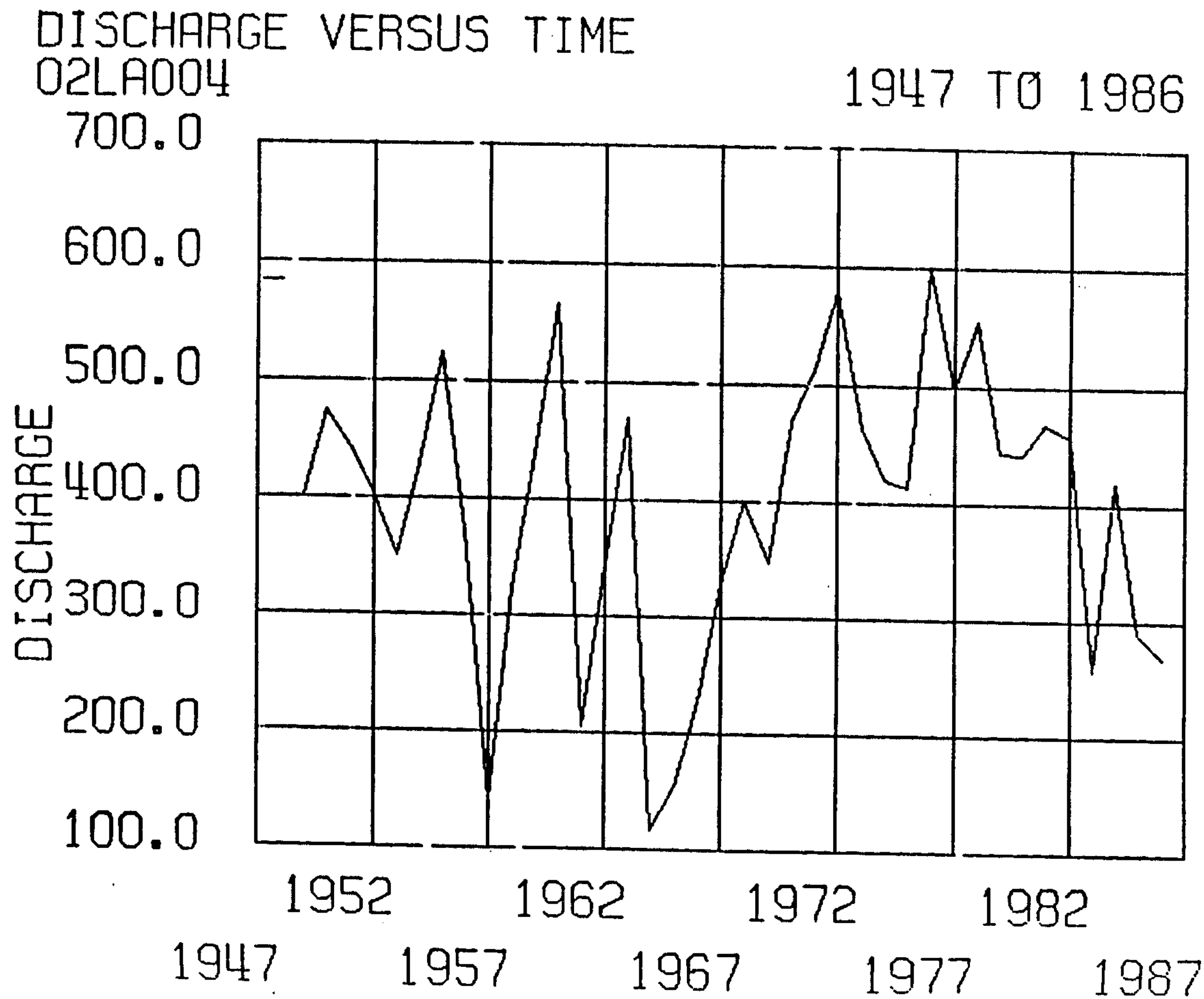
F. Ivan Lorant, P.Eng.
Project Manager

Paul D. Holmes, P.Eng.
Project Engineer

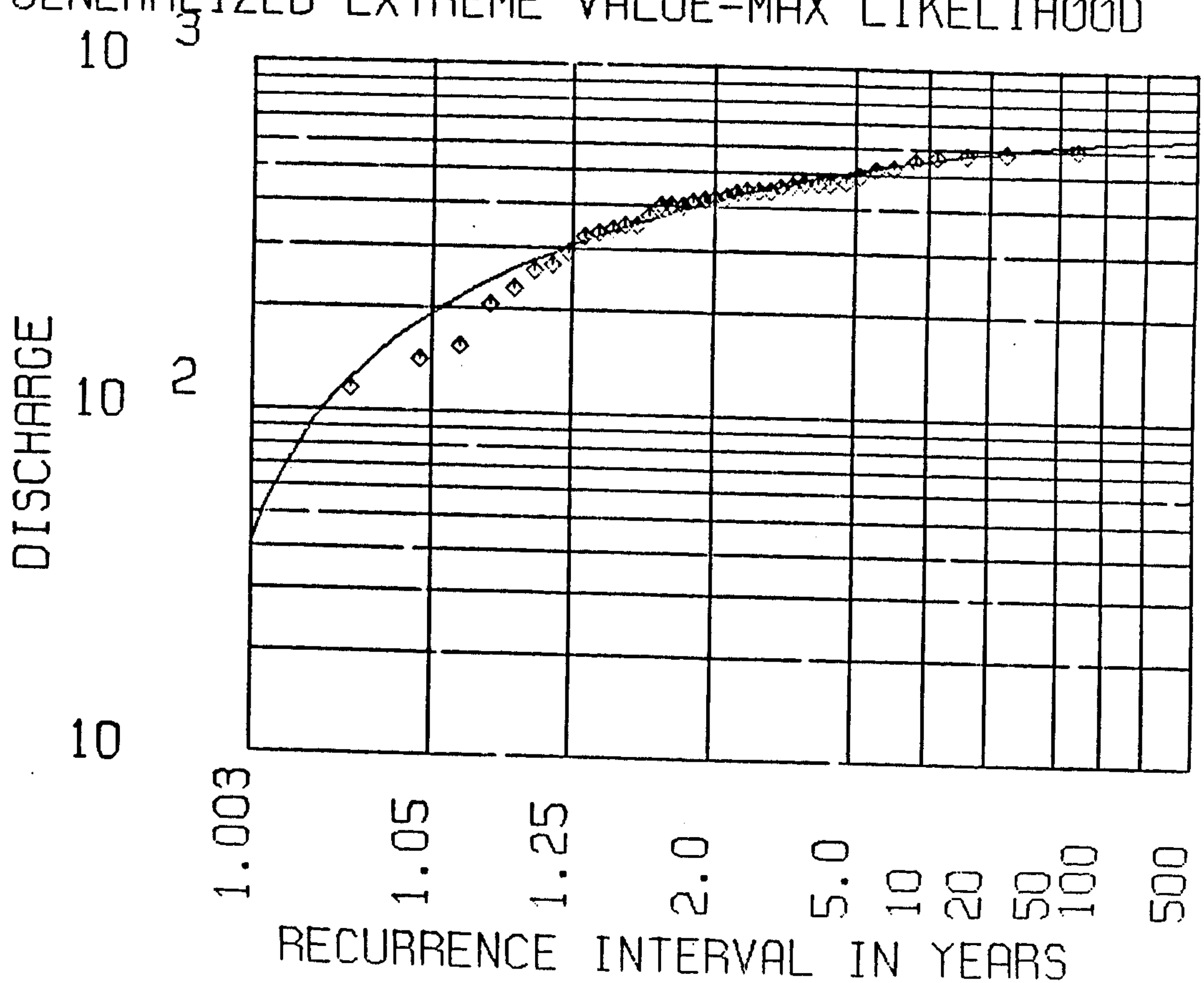
M.M. DILLON LIMITED
CONSULTING ENGINEERS, PLANNERS & SCIENTISTS

appendices

APPENDIX A
CFA88 COMPUTER PLOTS
RIDEAU RIVER AT OTTAWA
STATION 02LA004


DISCHARGE VERSUS TIME
02LA004
1947 TO 1986
DISCHARGE
700.0
600.0
500.0
400.0
300.0
200.0
100.0
1947
1952
1957
1962
1967
1972
1977
1982
1987

FREQUENCY ANALYSIS - 02LA004
GENERALIZED EXTREME VALUE-MAX LIKELIHOOD



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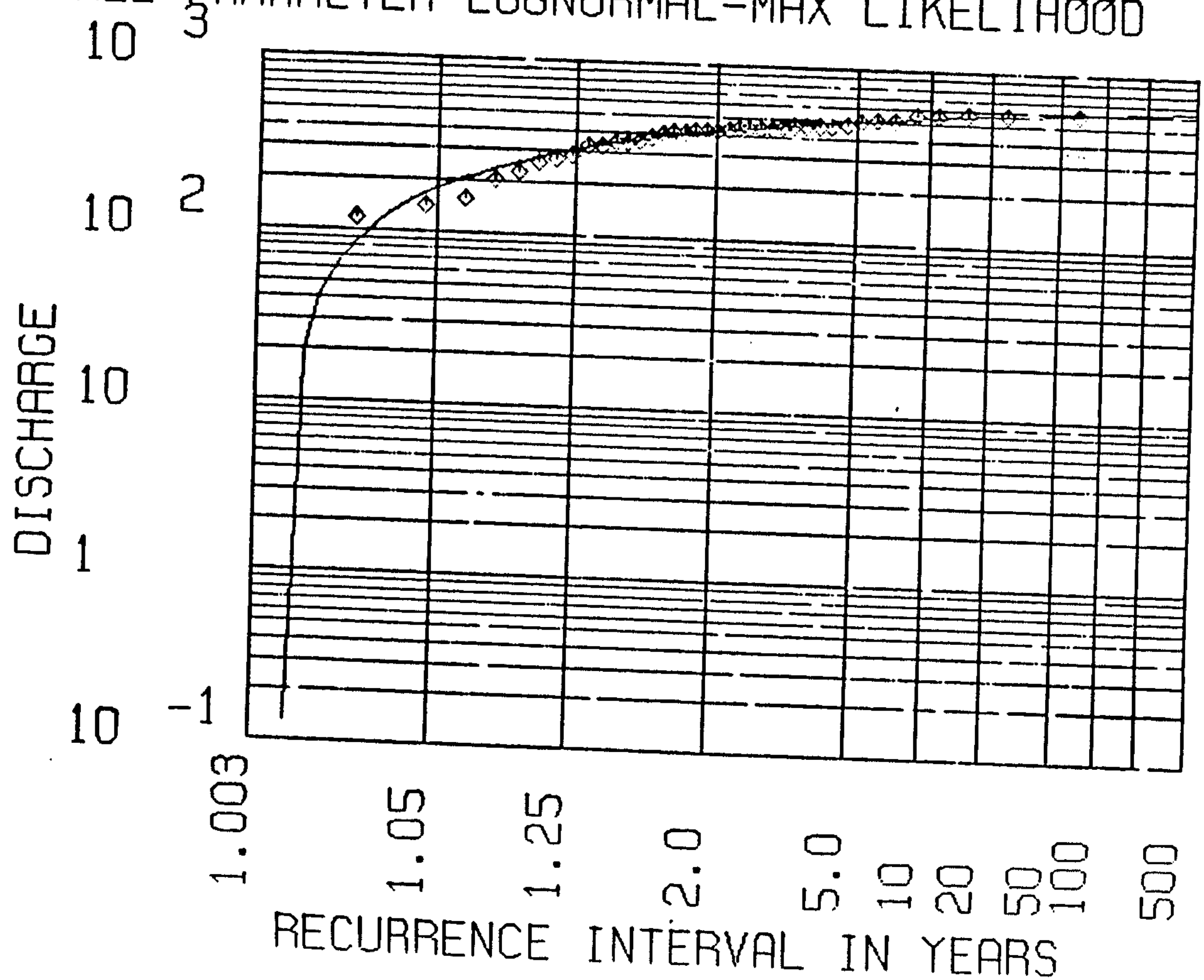
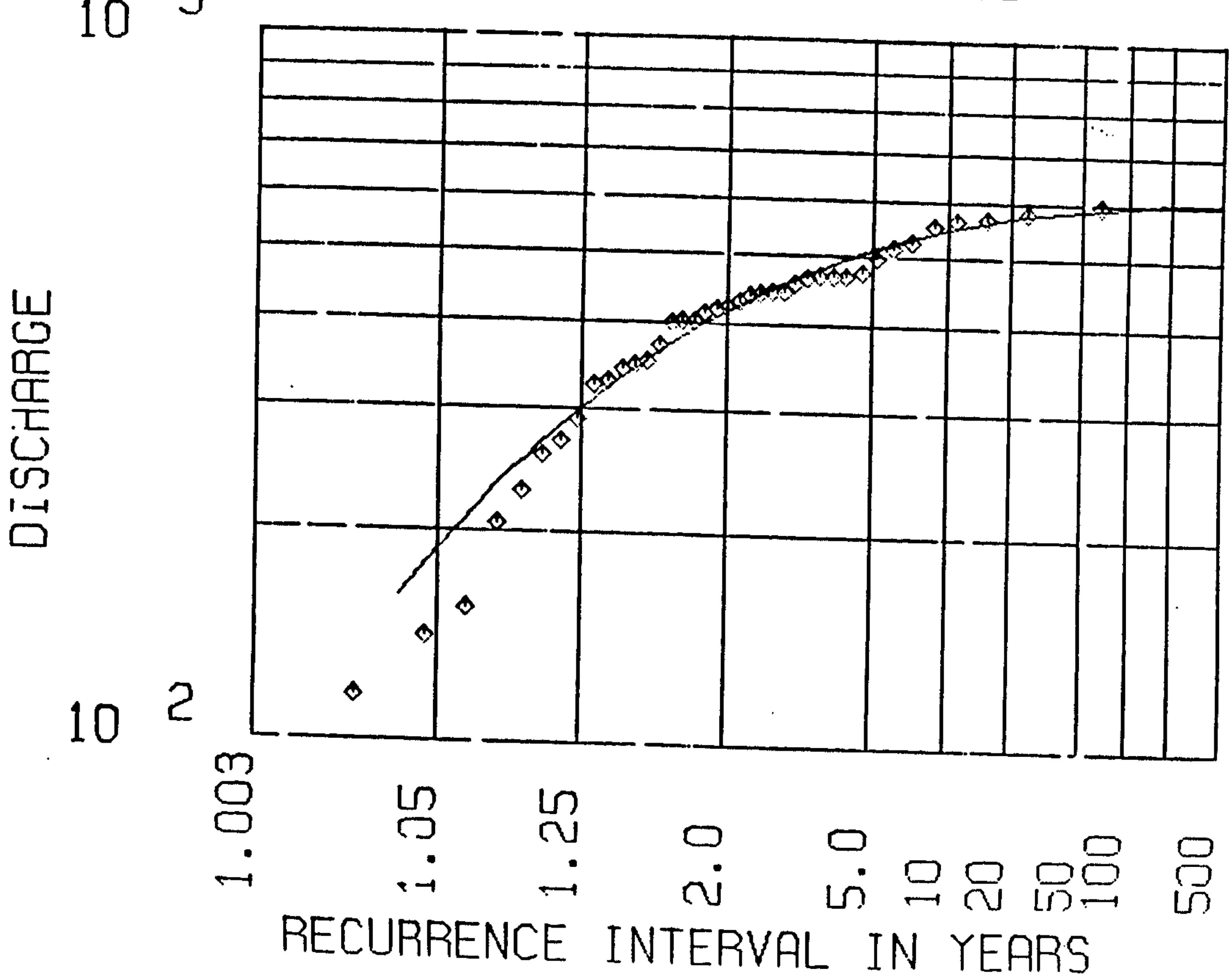
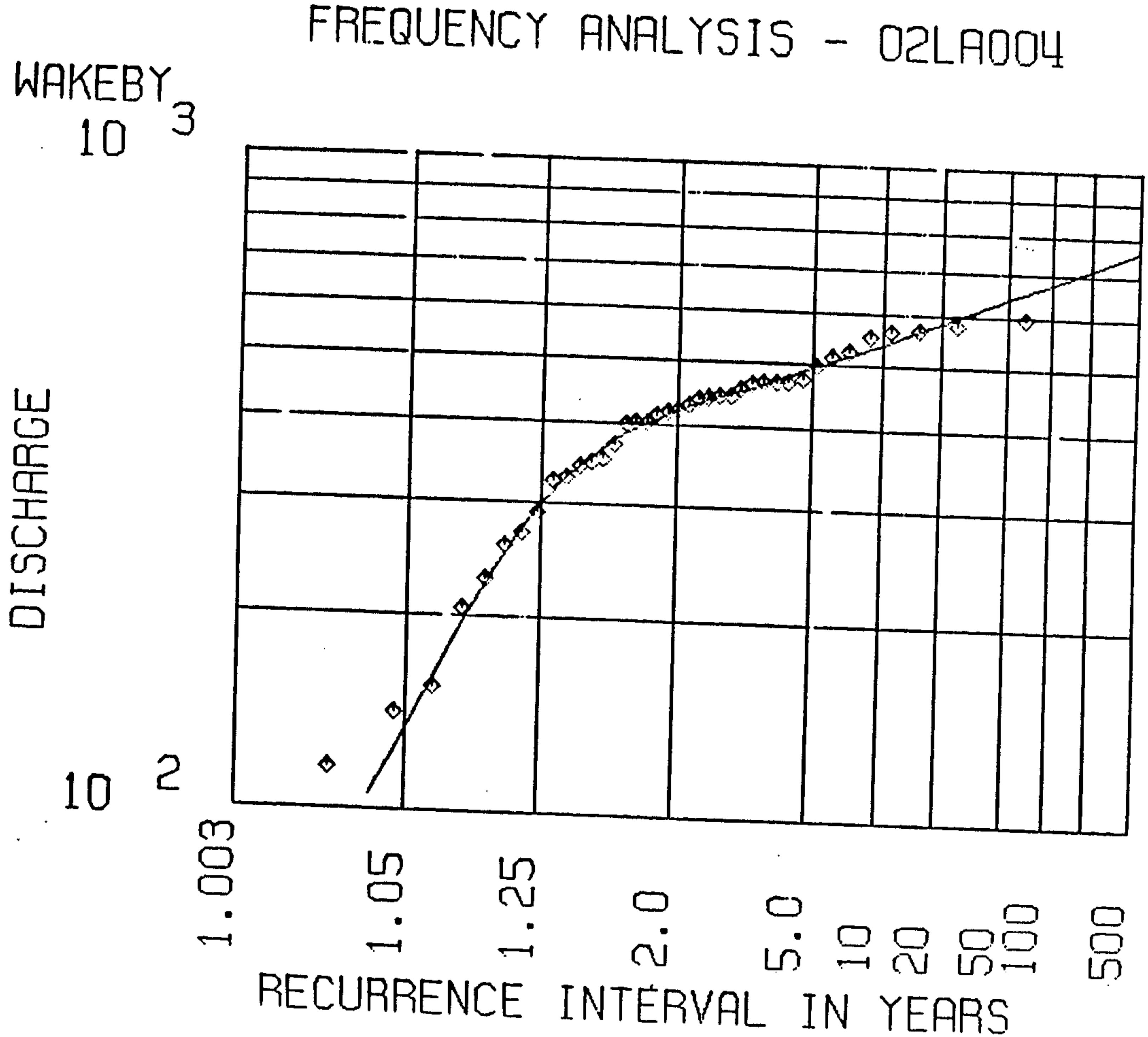
THREE PARAMETER LOGNORMAL-MAX LIKELIHOOD



FREQUENCY ANALYSIS - 02LA004

LOG PEARSON TYPE III-MAX LIKELIHOOD



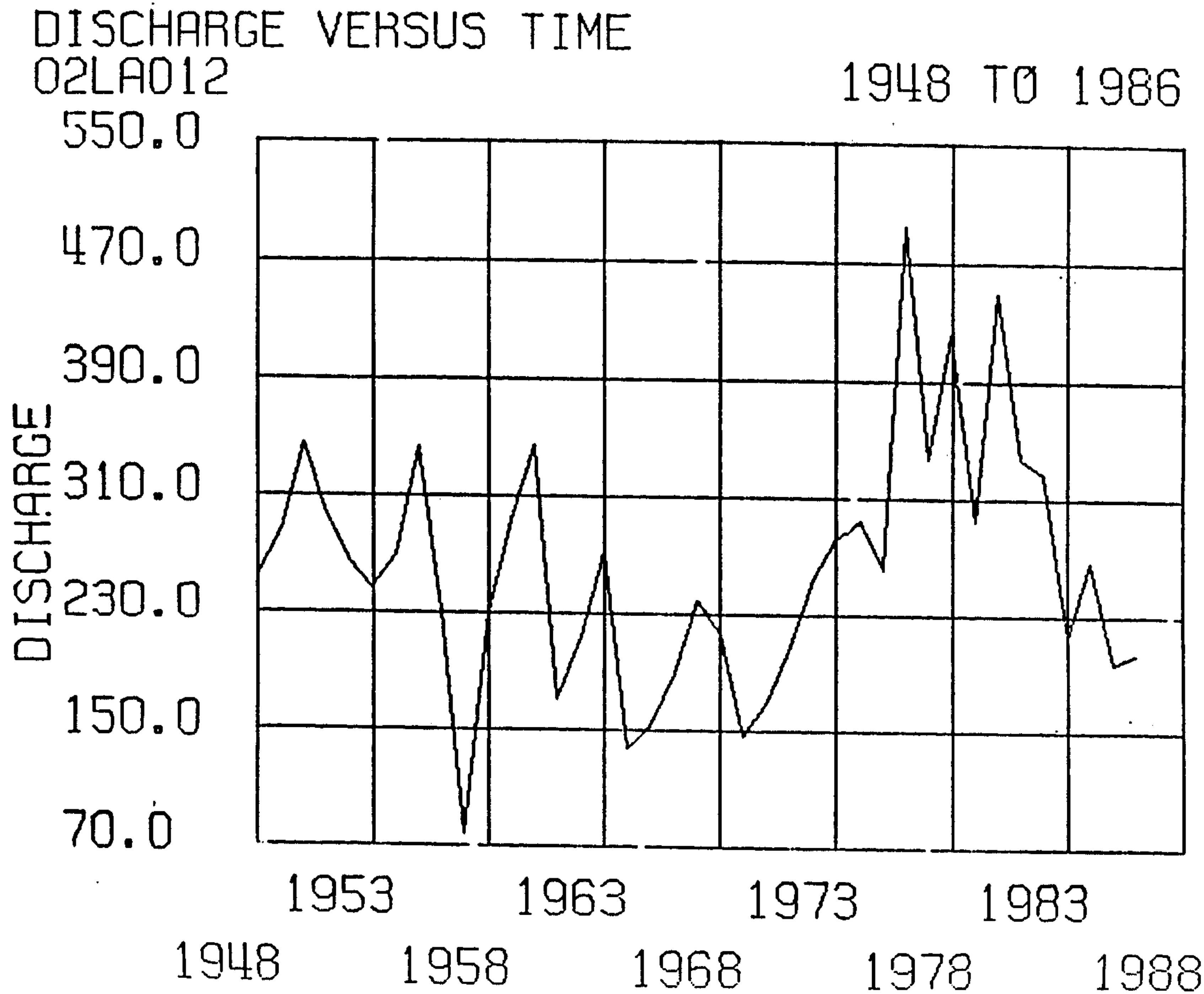

FREQUENCY ANALYSIS - 02LA004
WAKEBY
10^3
DISCHARGE
10^2
1.003
1.05
1.25
2.0
5.0
10
20
50
100
500
RECURRENCE INTERVAL IN YEARS

APPENDIX B

CFA88 COMPUTER PLOTS

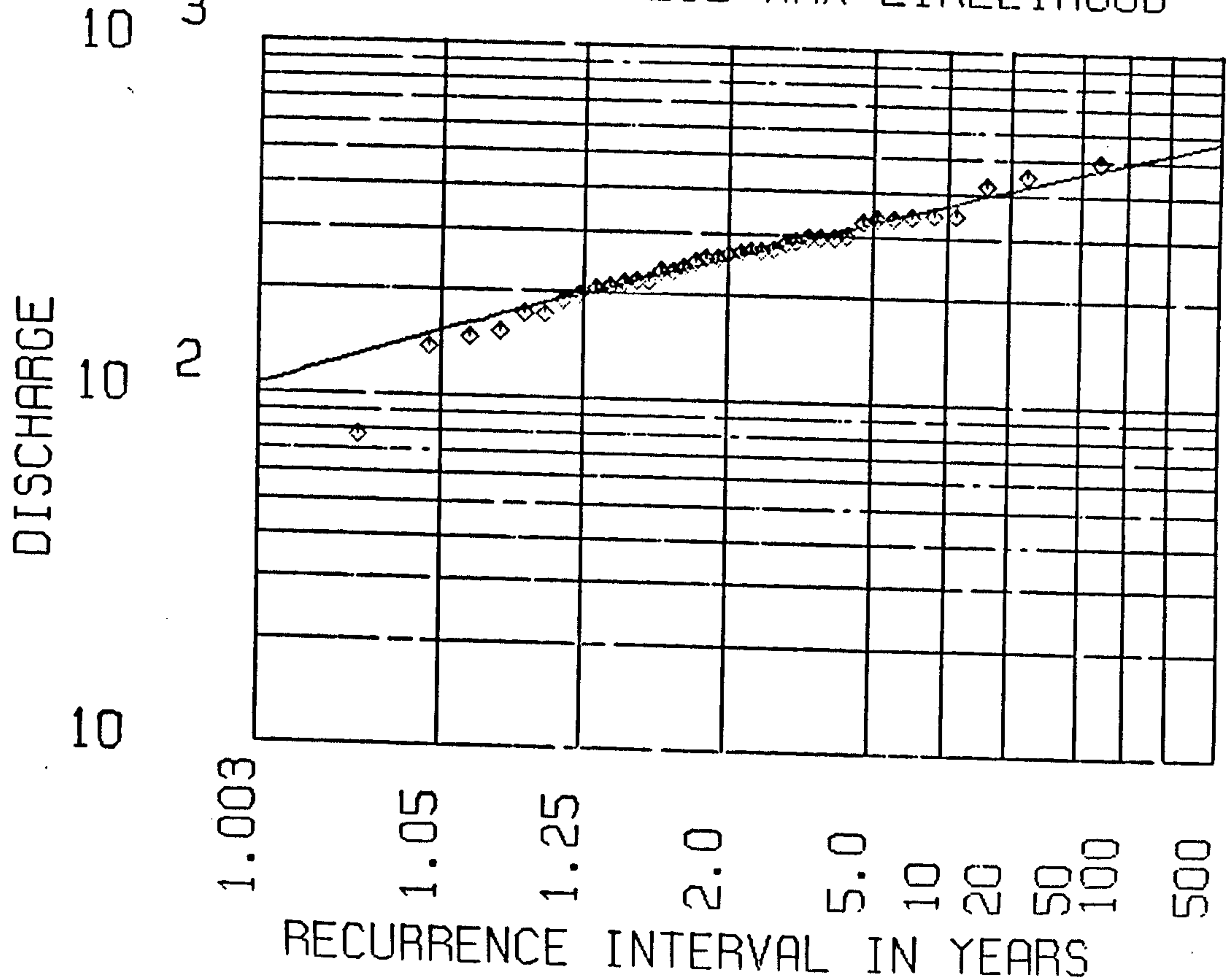
RIDEAU RIVER BELOW MANOTICK

STATION 02LA012


DISCHARGE VERSUS TIME
02LA012
1948 TØ 1986
DISCHARGE
550.0
470.0
390.0
310.0
230.0
150.0
70.0
1948
1953
1958
1963
1968
1973
1978
1983
1988

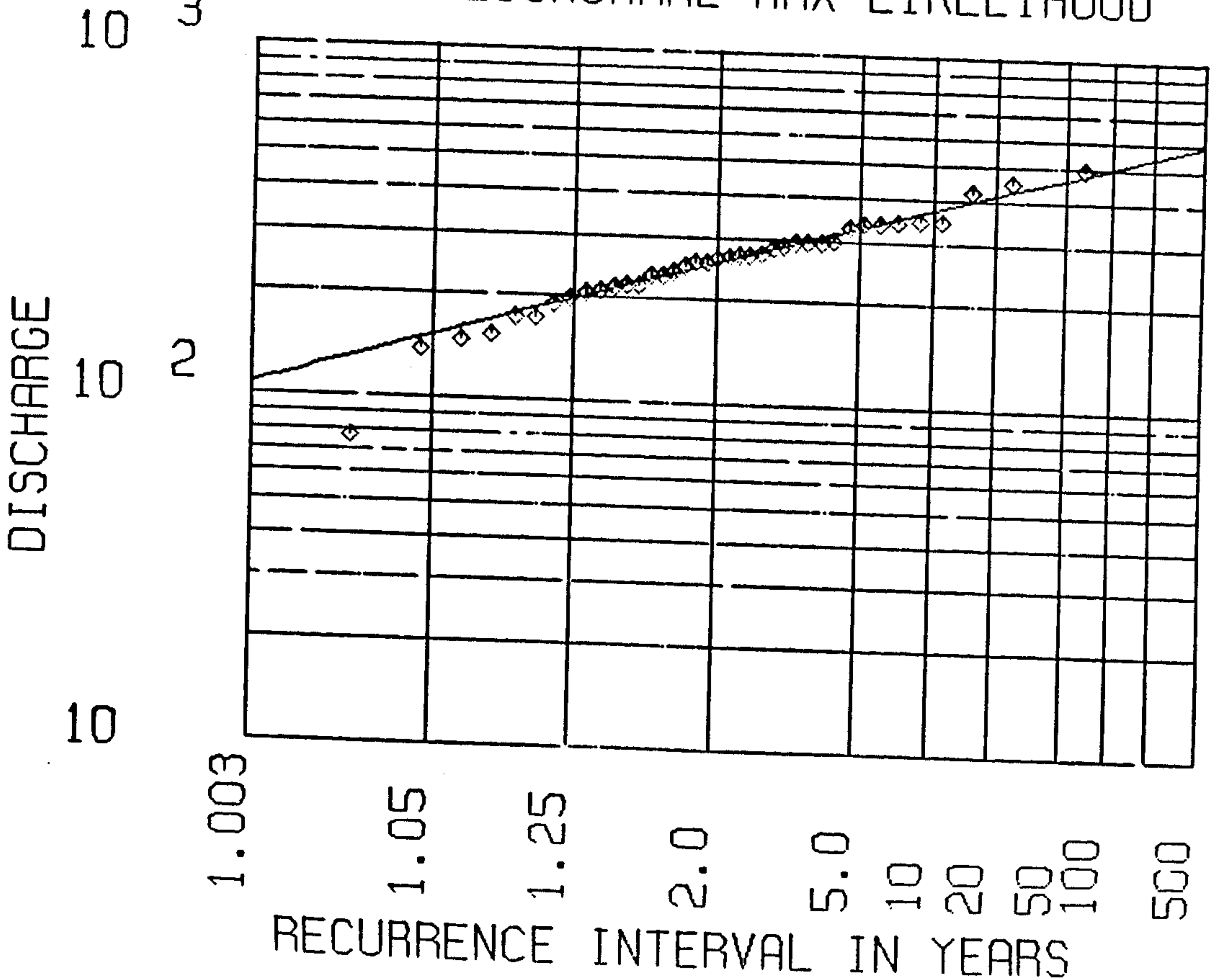
FREQUENCY ANALYSIS - 021.A012

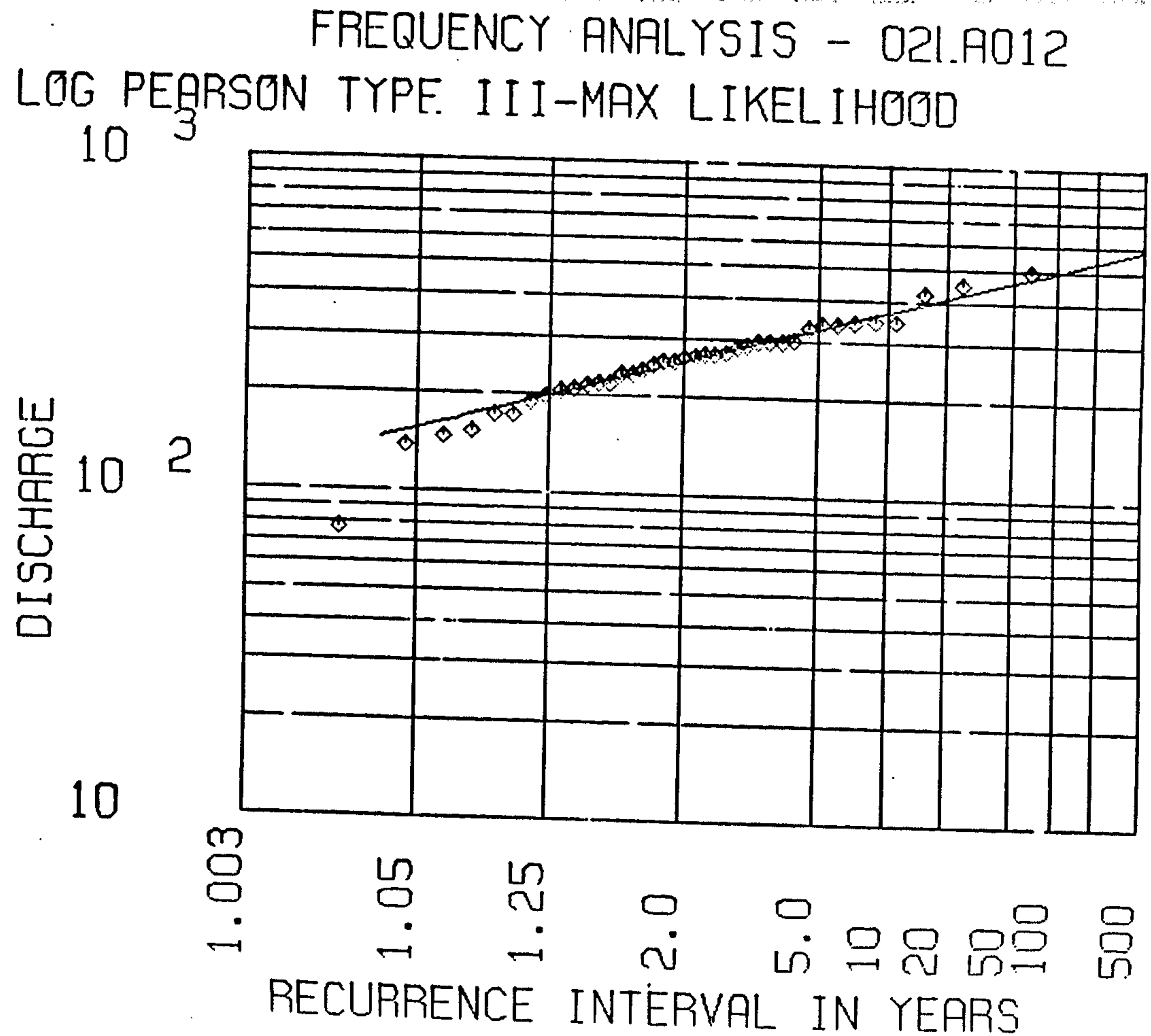
GENERALIZED EXTREME VALUE-MAX LIKELIHOOD



FREQUENCY ANALYSIS - 02LA012

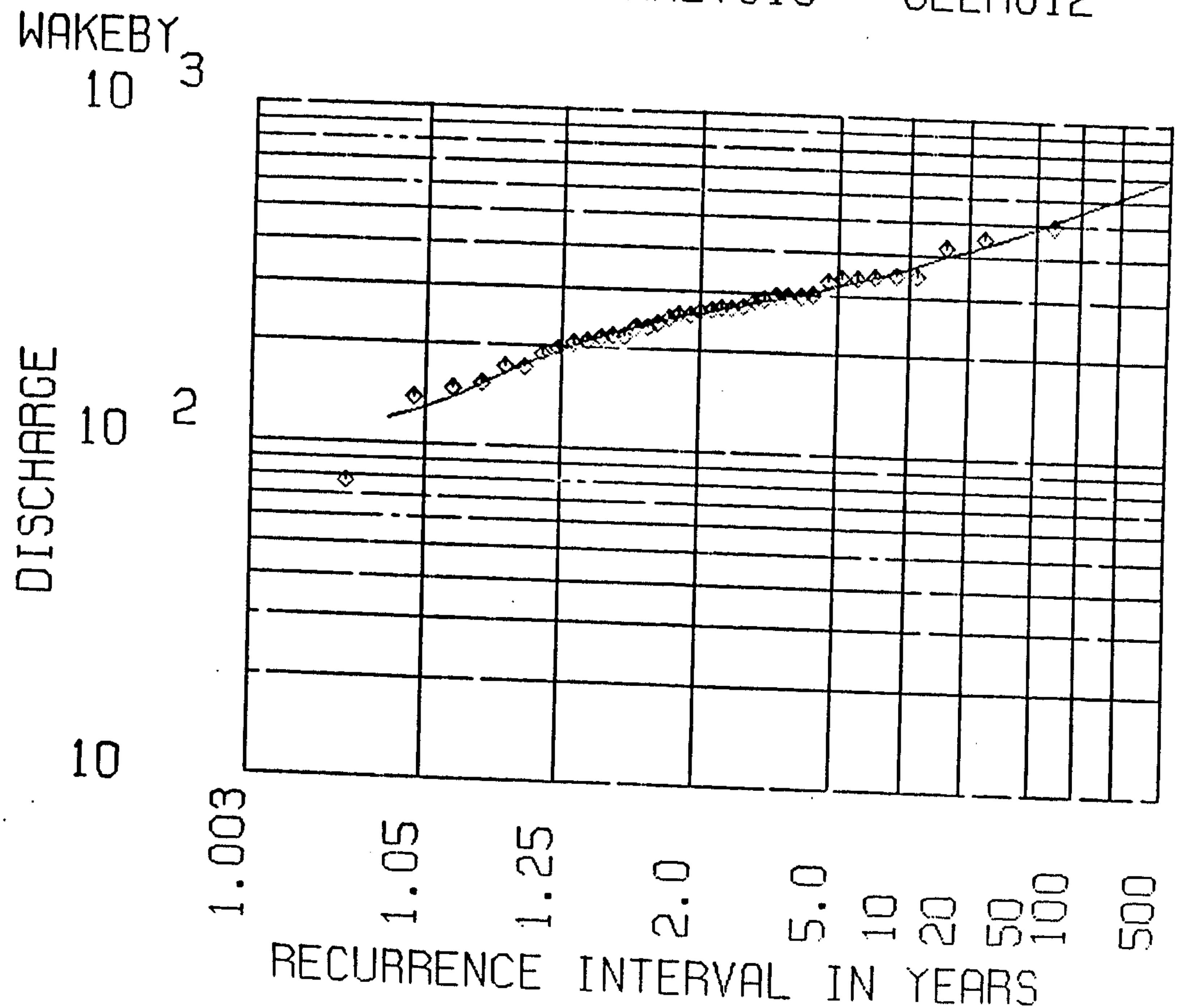
THREE PARAMETER LOGNORMAL-MAX LIKELIHOOD



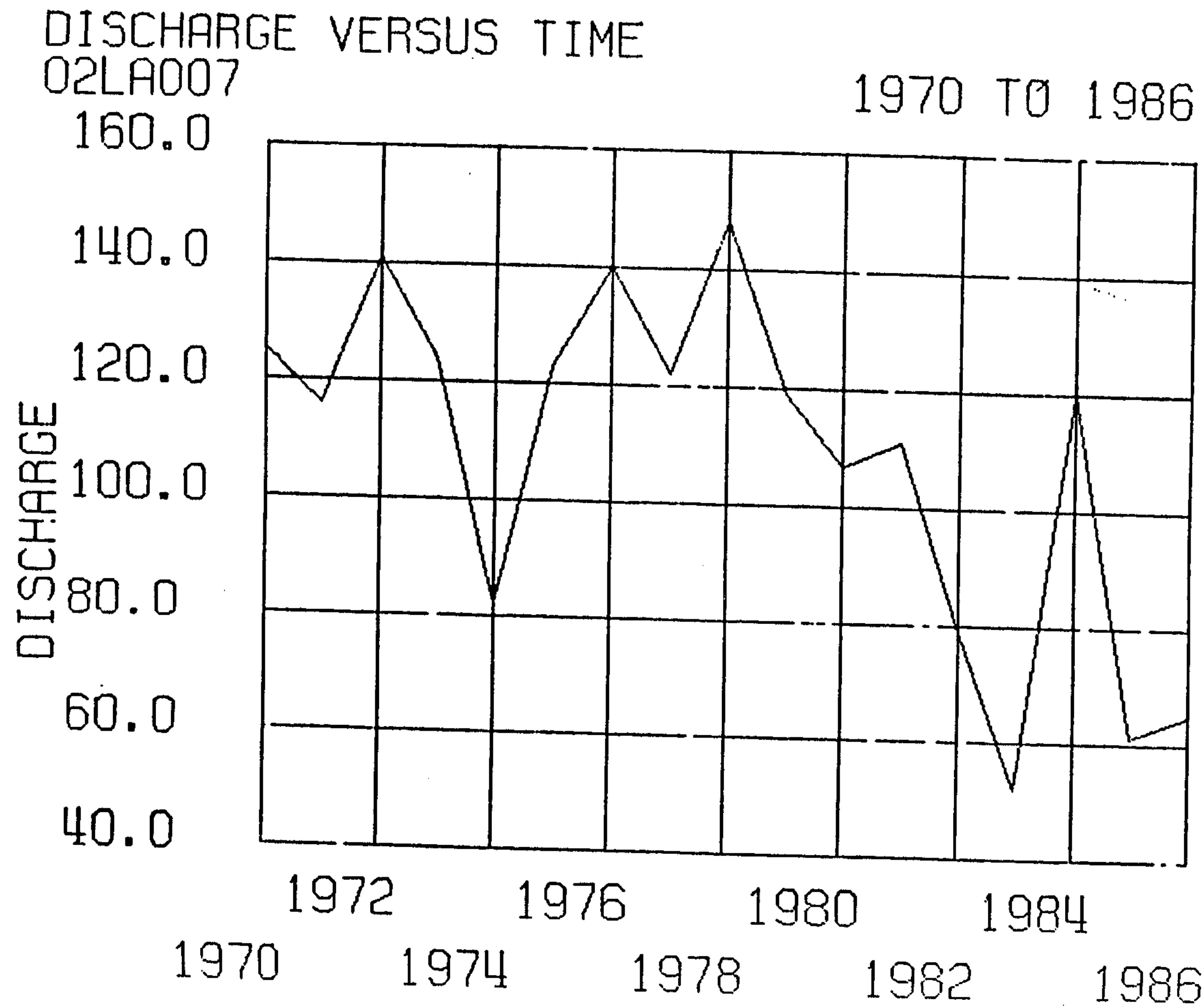


FREQUENCY ANALYSIS - 021.A012
LOG PEARSON TYPE III-MAX LIKELIHOOD
10 3
10 2
10
DISCHARGE
1.003
1.05
1.25
2.0
5.0
10
20
50
100
500
RECURRENCE INTERVAL IN YEARS

FREQUENCY ANALYSIS - 02LA012

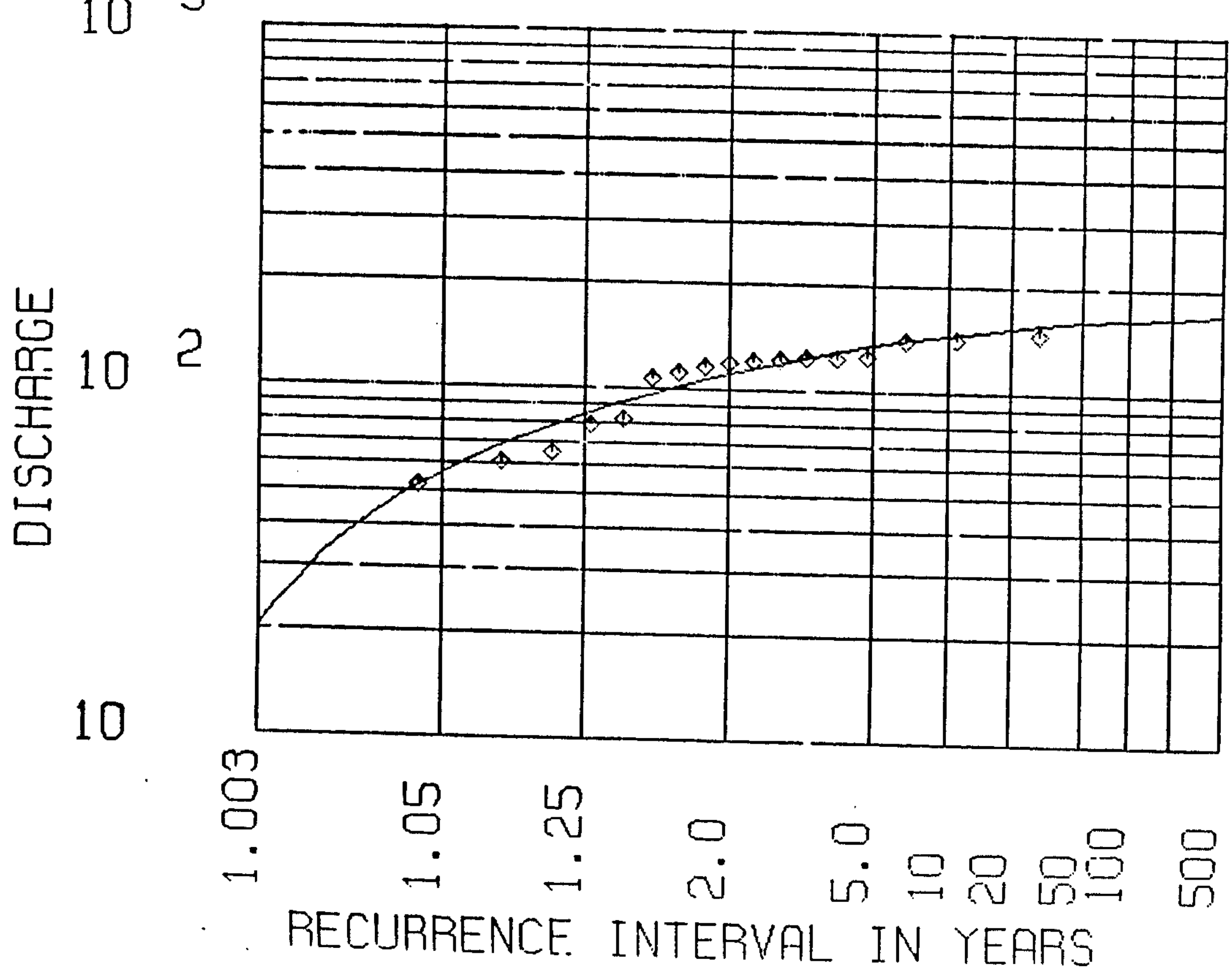


APPENDIX C1
CFA88 COMPUTER PLOTS
JOCK RIVER NEAR RICHMOND
STATION 02LA007


DISCHARGE VERSUS TIME
02LA007
1970 TØ 1986
DISCHARGE
160.0
140.0
120.0
100.0
80.0
60.0
40.0
1970
1972
1974
1976
1978
1980
1982
1984
1986

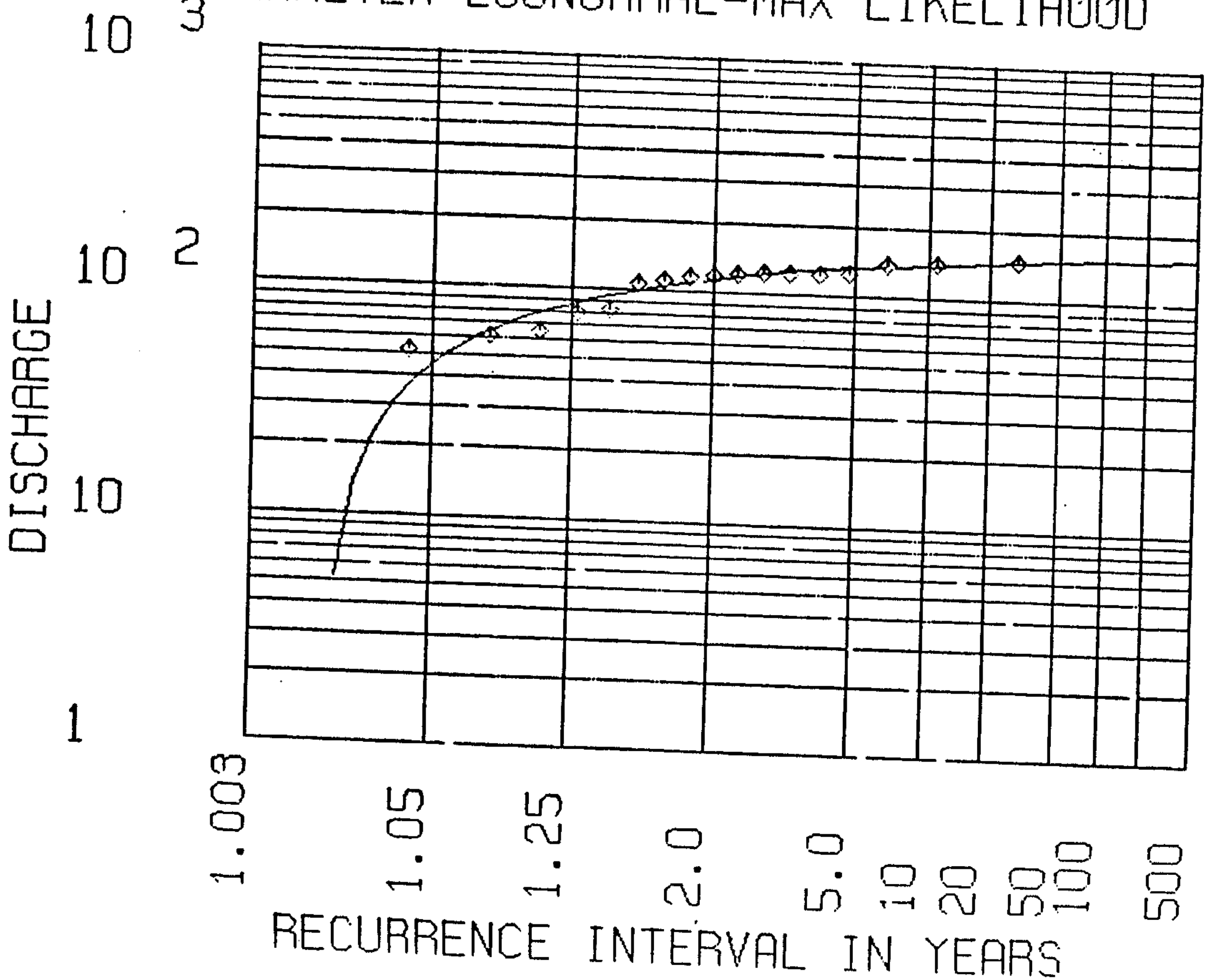
FREQUENCY ANALYSIS - 02LA007

GENERALIZED EXTREME VALUE-MAX LIKELIHOOD



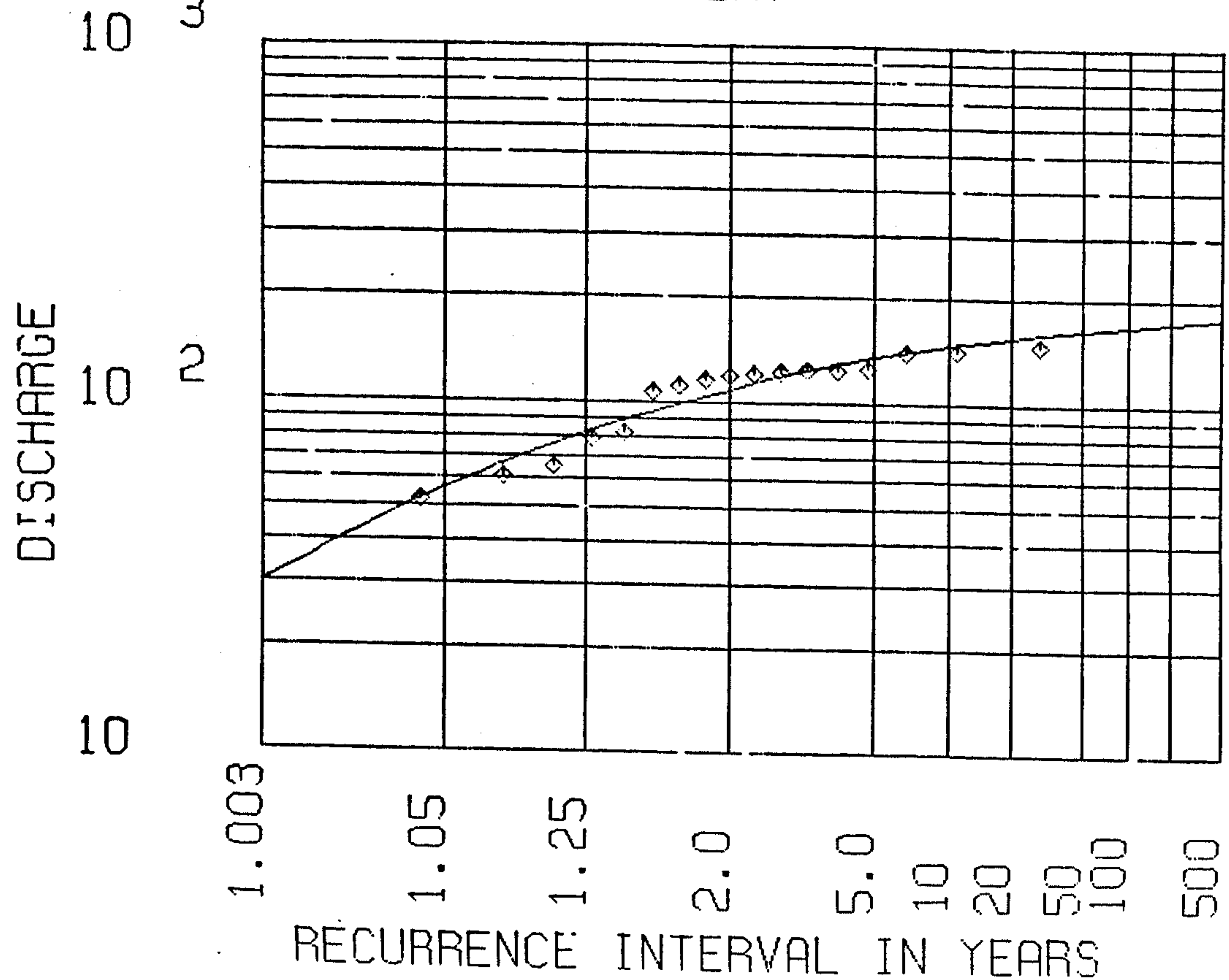
FREQUENCY ANALYSIS - 02LA007

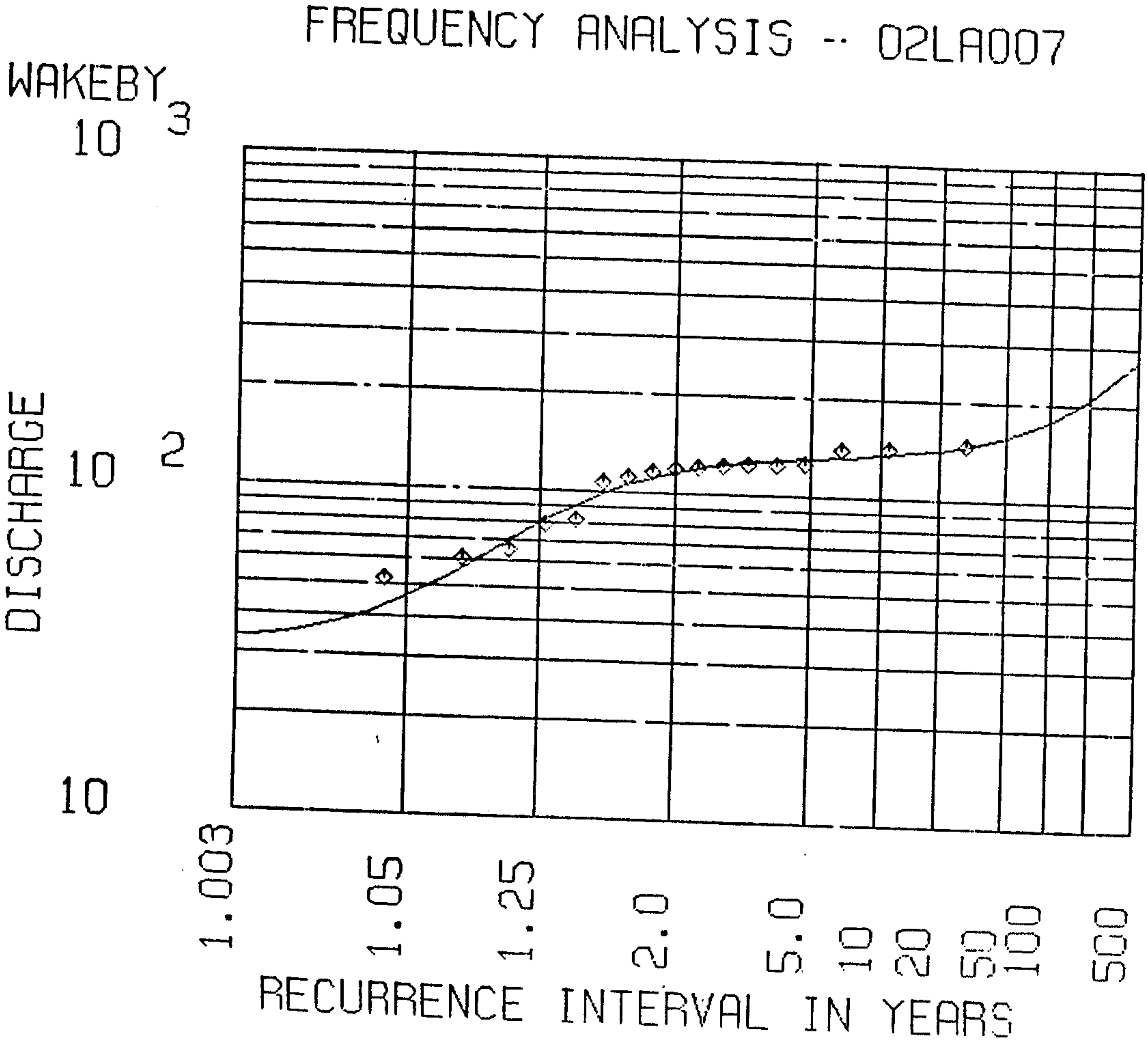
THREE PARAMETER LØGNØRMAL-MAX LIKELIHØØD



FREQUENCY ANALYSIS - 02LA007

LOG PEARSON TYPE III-MOMENT





FREQUENCY ANALYSIS -- 02LA007
WAKEBY
10 3
10 2
10
DISCHARGE
1.003
1.05
1.25
2.0
5.0
10
20
50
100
500
RECURRENCE INTERVAL IN YEARS

APPENDIX C2
JOCK RIVER
REGIONAL FREQUENCY ANALYSIS
MNR REGRESSION EQUATION METHOD

JOCK RIVER NEAR OTTAWA

MNR REGRESSION EQUATIONS

EASTERN REGION: PARAMETERS USED IN REGRESSION EQUATION

RETURN PERIOD	a0	a1	a2	a3	a4	a5
2	0.49113	0.8721	-0.5888	0.0000	0.0000	0.0000
20	0.41229	0.7792	-0.6526	0.0000	-0.1632	0.0000
100	0.62500	0.7392	-0.6712	0.0000	-0.1648	0.0000
PEAKING FACTOR	523.528	-0.3606	-2.0567	0.0000	0.3633	-0.6326

WATERSHED PARAMETERS

DRAINAGE AREA; DA = 559 km2
INDEX FOR LAKES AND SWAMPS; ACLS = 1.15
EQUIVALENT SLOPE; EQSLP = .0046 m/m
MEAN ANNUAL RUNOFF; MAR = 325 mm
MEAN ANNUAL SNOWFALL; MAS = 200 cm

CALCULATED PEAK FLOWS

RETURN PERIOD	MAX DAILY		MAX INSTANTANEOUS	
(YEARS)	(m3/s)	(cfs)	(m3/s)	(cfs)
2.0	112.6	3975.9	135.0	4766.7
5.0	118.9	4198.9	142.5	5034.1
10.0	122.3	4320.5	146.7	5179.8
20.0	125.3	4423.5	150.2	5303.4
50.0	138.6	4895.6	166.2	5869.3
100.0	148.3	5237.9	177.8	6279.7

PEAKING FACTOR = 1.199

APPENDIX D
HYDRAULIC BRIDGE TABLE
DATA

BRIDGE DATA

WATERCOURSE Rideau River

MAP SHEET NO. 1

LOCATION Section No. 0.000

U.T.M. GRID REFERENCE

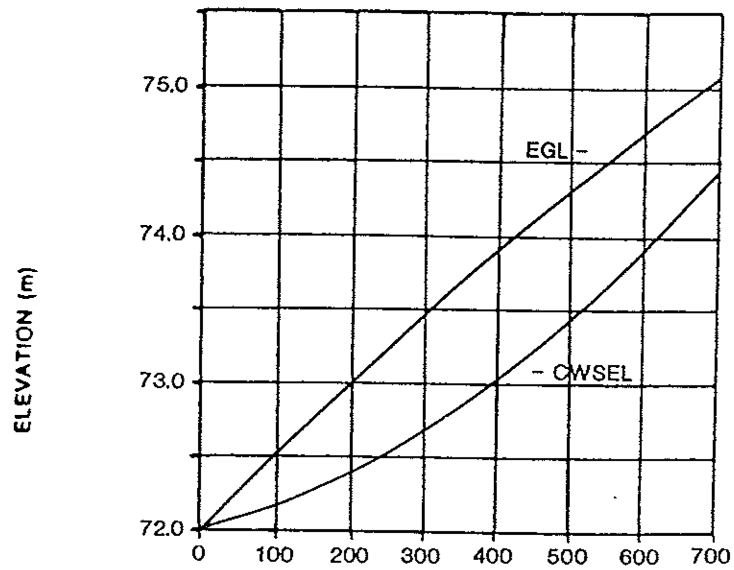
STRUCTURE Hog's Back Dam

A. SPECIFICATIONS

Span	67.7	m
Length of Structure	70.0	m
Top of Road Elevation	77.0	m
Low Chord (Soffit) Elevation	76.5	m
Upstream Invert Elevation	69.6	m
Effective Flow Area	311.2	m^2

8 - Bay concrete structure
6 - Bays with wooden stop logs
2 - Bays with steel control gate

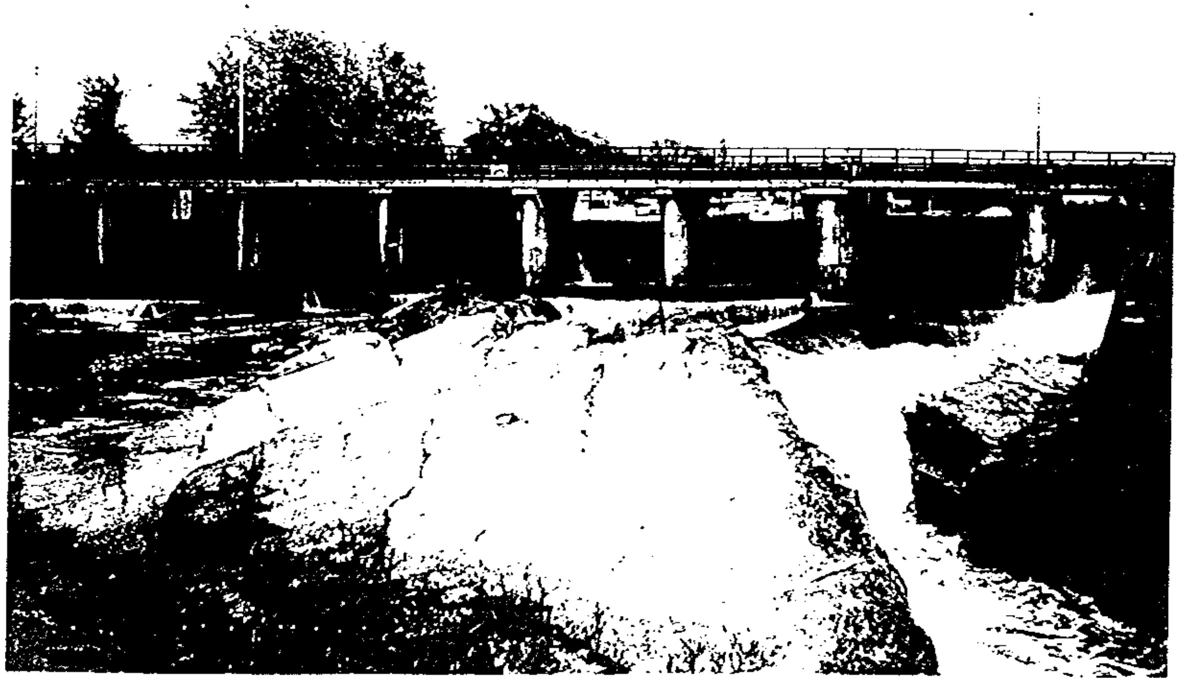
B. STAGE DISCHARGE CURVE @ Sec. 0.050



ALL STOP LOGS OUT AND GATES FULLY OPEN

DISCHARGE (m^3/S)

C. PHOTOGRAPHIC PRESENTATION: LOOKING UPSTREAM



BRIDGE DATA

WATERCOURSE Rideau River

MAP SHEET NO. 1

LOCATION Section No. 0.000

U.T.M. GRID REFERENCE

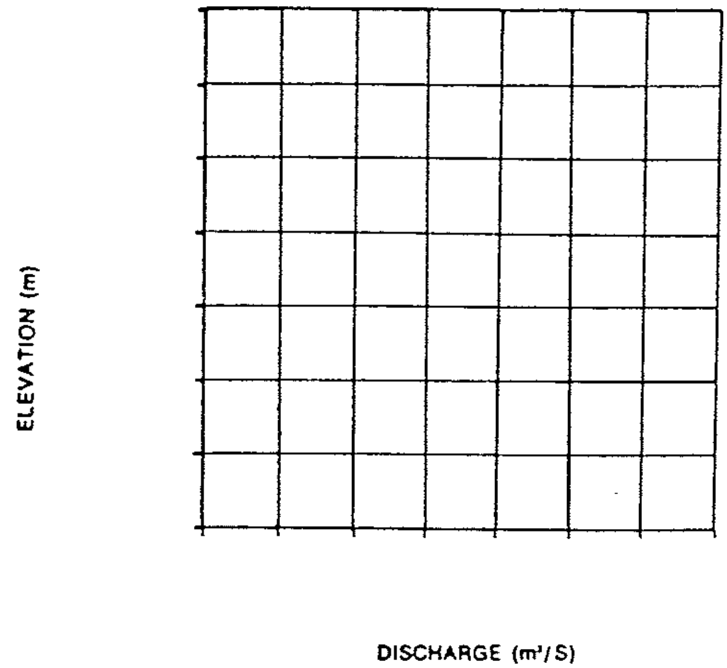
STRUCTURE Hog's Back Road Bridge

A. SPECIFICATIONS

Span	67.7	m
Length of Structure	12	m
Top of Road Elevation	78.4	m
Low Chord (Soffit) Elevation	76.5	m
Upstream Invert Elevation	69.4	m
Effective Flow Area	311	m^2

2 - Piers

B. STAGE DISCHARGE CURVE



C. PHOTOGRAPHIC PRESENTATION: LOOKING DOWNSTREAM

BRIDGE DATA

WATERCOURSE Rideau River

MAP SHEET NO 3

LOCATION Section No. 3.630

U.T.M. GRID REFERENCE

STRUCTURE Canadian National Rai¹way Bridge

A. SPECIFICATIONS

Span 138 m

Length of Structure 10 m

Top of Road Elevation 89.4 m

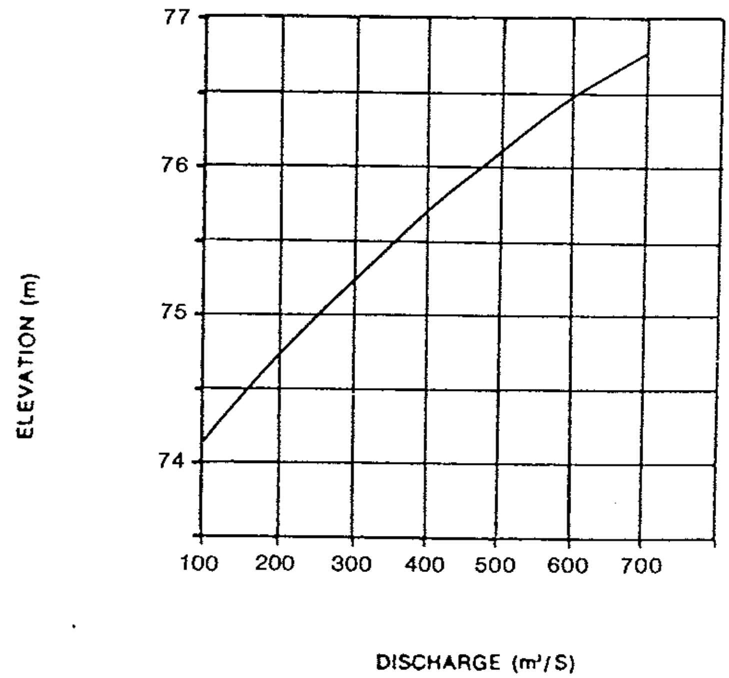
Low Chord (Soffit) Elevation 85.7 m

Upstream Invert Elevation 72.0 m

Effective Flow Area 375 m^2
@ 100-year level

4 - Piers

B. STAGE DISCHARGE CURVE @ Sec. 3.640



C. PHOTOGRAPHIC PRESENTATION : LOOKING DOWNSTREAM

BRIDGE DATA

WATERCOURSE Rideau River

MAP SHEET NO. 3

LOCATION Section No. 5.075

U.T.M. GRID REFERENCE

STRUCTURE Hunt Club Bridge

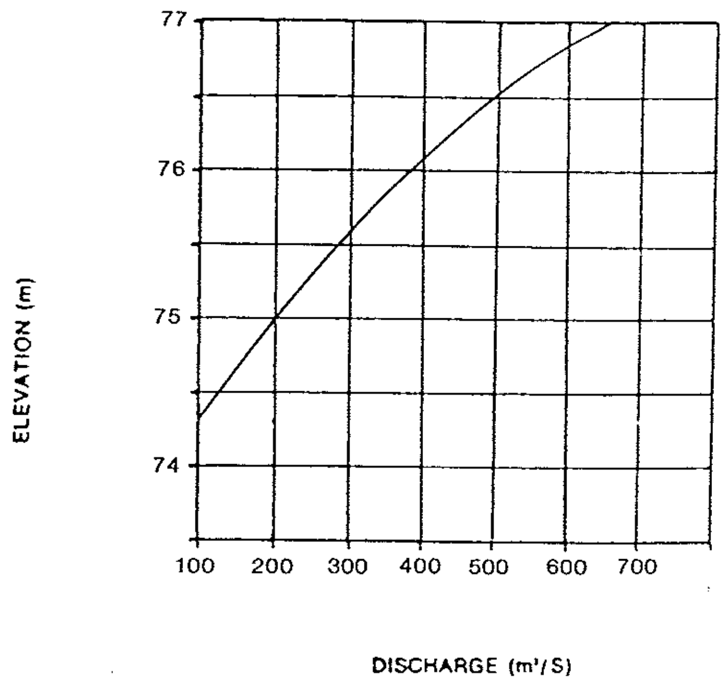
A. SPECIFICATIONS

Span	81.5 m
Length of Structure	25 m
Top of Road Elevation	92.7 m
Low Chord (Soffit) Elevation	96.6 m
Upstream Invert Elevation	67.7 m
Effective Flow Area	66.4 m^2

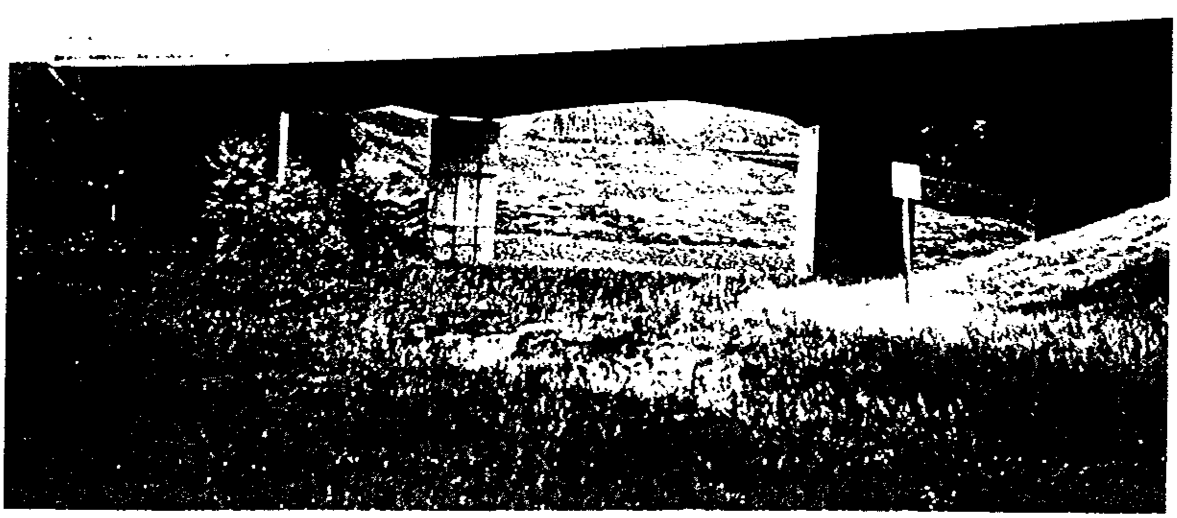
@ 100-year Level

4 - Piers

B. STAGE DISCHARGE CURVE @ Sec. 5.100



C. PHOTOGRAPHIC PRESENTATION: LOOKING UPSTREAM



BRIDGE DATA

WATERCOURSE Rideau River

MAP SHEET NO. 4

LOCATION Section No. 6.650

U.T.M. GRID REFERENCE

STRUCTURE Black Rapids Dam

A. SPECIFICATIONS

Span 166 m

Length of Structure – m

Top of Road Elevation 77.12 m

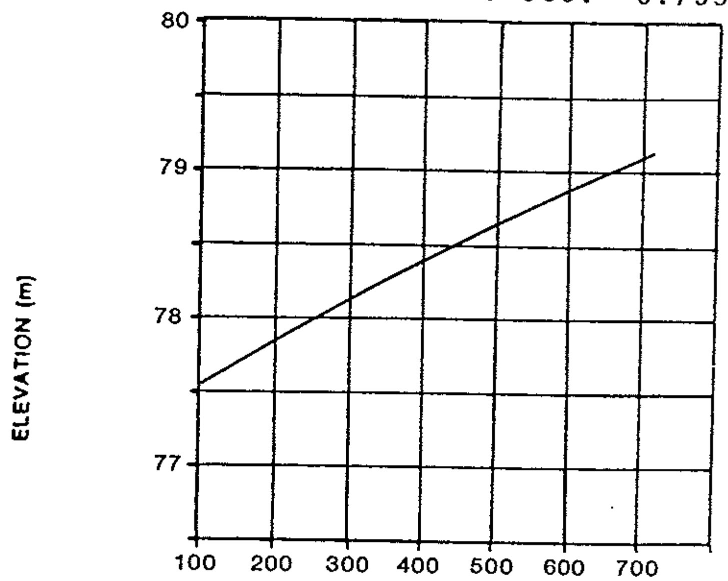
Low Chord (Soffit) Elevation – m

Upstream Invert Elevation 73.5 m

Effective Flow Area 185 m^2 @ 100-year Level

. Ogee Spillway
. 2-Piers
. 2-Weirs

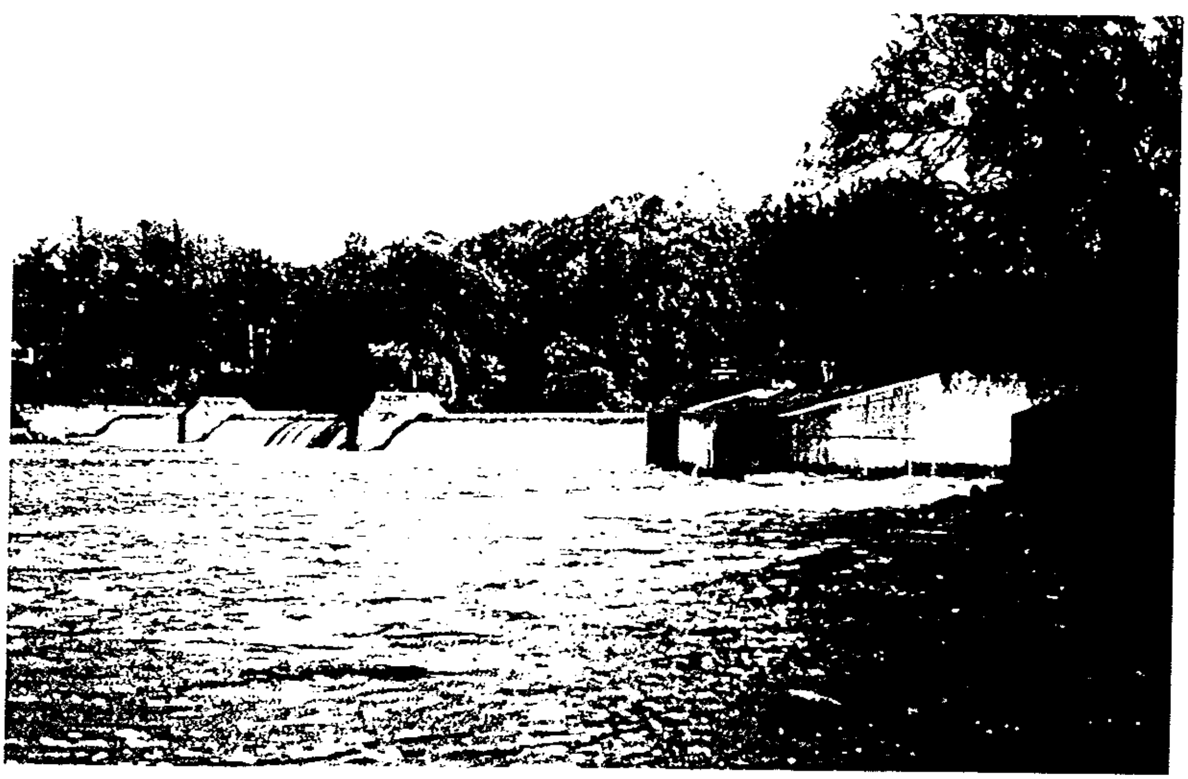
B. STAGE DISCHARGE CURVE @ Sec. 6.755



ALL STOP LOGS OUT

DISCHARGE (m^3/S)

C. PHOTOGRAPHIC PRESENTATION : LOOKING UPSTREAM



BRIDGE DATA

WATERCOURSE Rideau River

MAP SHEET NO. 10

LOCATION Section No. 15.425

U.T.M. GRID REFERENCE

STRUCTURE Long Island Dam - East Branch

A. SPECIFICATIONS

Span 38.5 m

Length of Structure 10 m

Top of Road Elevation 86.8 m

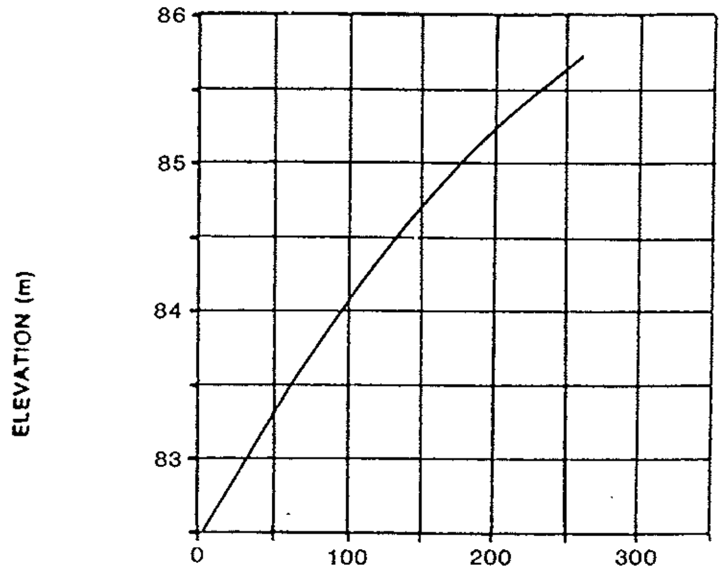
Low Chord (Soffit) Elevation 86.2 m

Upstream Invert Elevation 82.3 m

Effective Flow Area 119 m^2

5 - Bay concrete structures
3 - Bays with wooden stop logs
2 - Bays with steel control gate

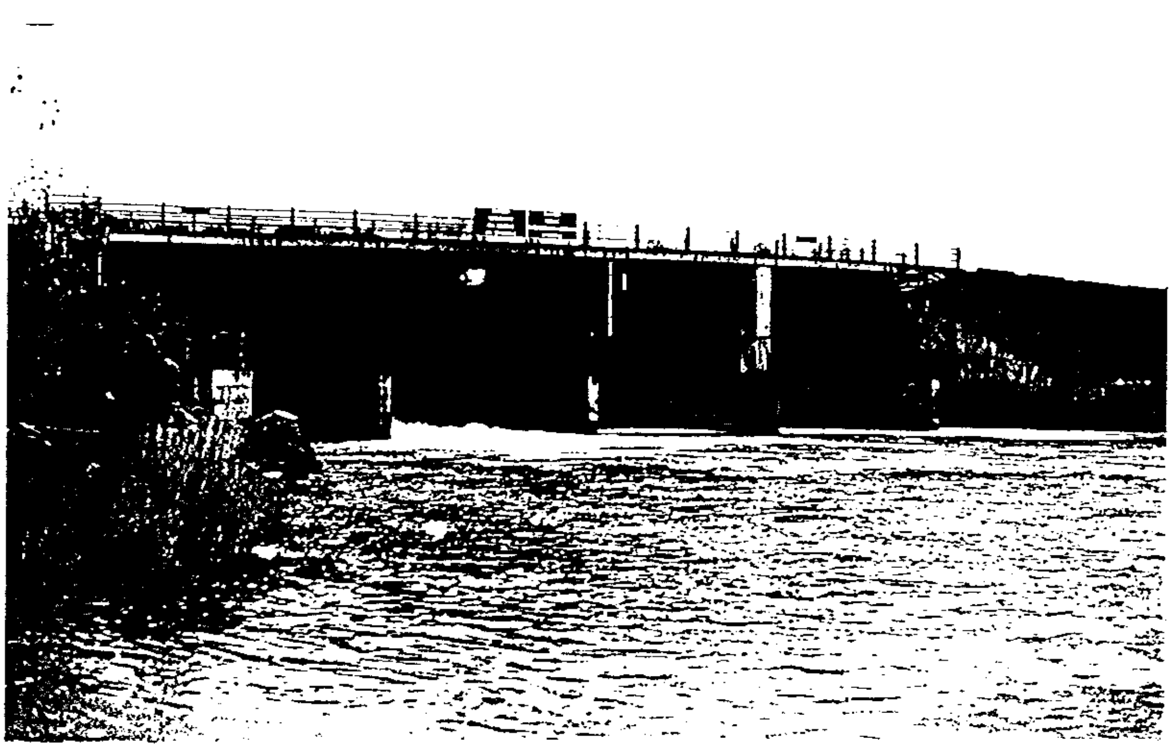
B. STAGE DISCHARGE CURVE @ Sec. 15.570



ALL STOP LOGS OUT AND GATES FULLY OPEN

DISCHARGE (m^3/S)

C. PHOTOGRAPHIC PRESENTATION : LOOKING UPSTREAM



BRIDGE DATA

WATERCOURSE Rideau River

MAP SHEET NO. 12

LOCATION Section No. 17.920

U.T.M. GRID REFERENCE

STRUCTURE Old Whitehorse Dam - East Branch

A. SPECIFICATIONS

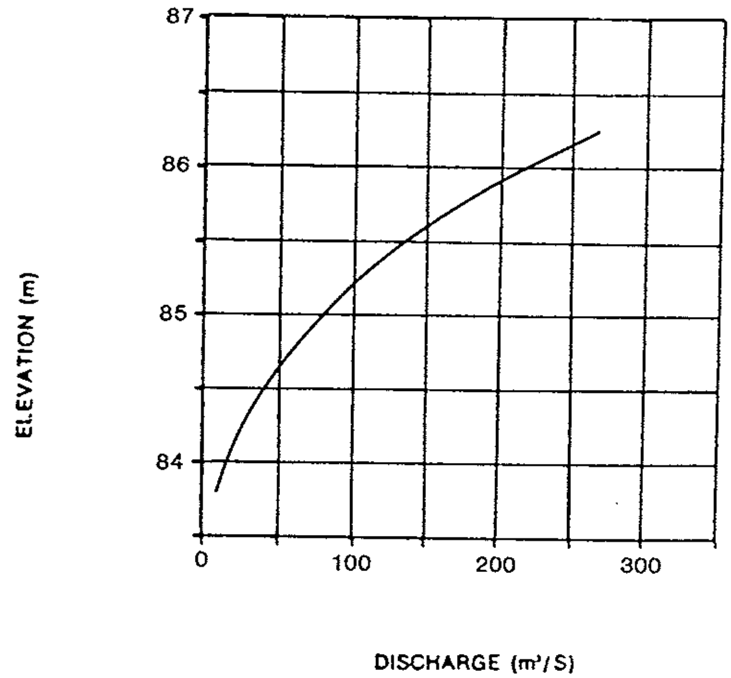
Span	73	m
Length of Structure	5	m
Top of Road Elevation	–	m
Low Chord (Soffit) Elevation	–	m
Upstream Invert Elevation	82.3	m
Effective Flow Area	96	m^2

@ 100-year Level

4 - Piers

Concrete Sill Below Waterline

B. STAGE DISCHARGE CURVE @ Sec. 18.135



C. PHOTOGRAPHIC PRESENTATION : LOOKING DOWNSTREAM

BRIDGE DATA

WATERCOURSE: Rideau River

MAP SHEET NO. 12

LOCATION: Section No. 18.185

U.T.M. GRID REFERENCE:

STRUCTURE: Manotick Bridge - East Branch

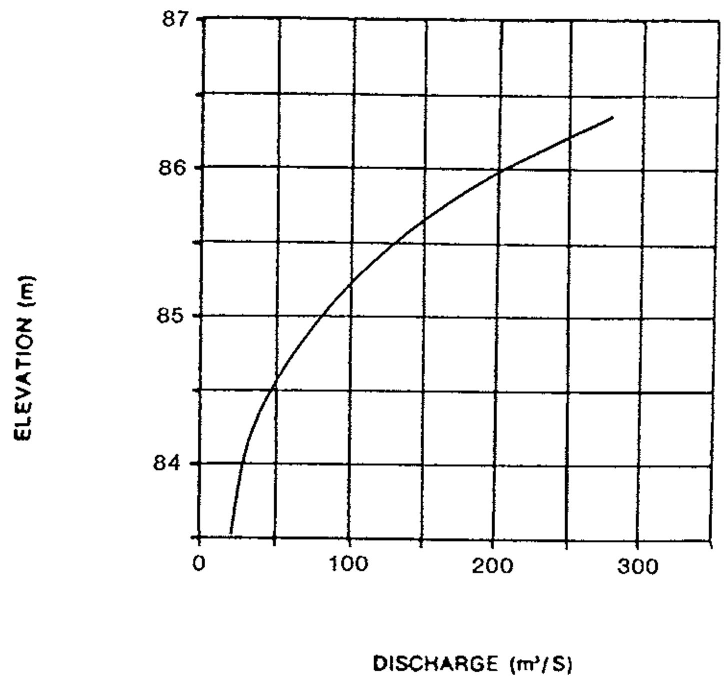
A. SPECIFICATIONS

Span	63.8	m
Length of Structure	11	m
Top of Road Elevation	93.0	m
Low Chord (Soffit) Elevation	92.2	m
Upstream Invert Elevation	81.4	m
Effective Flow Area	247	m^2

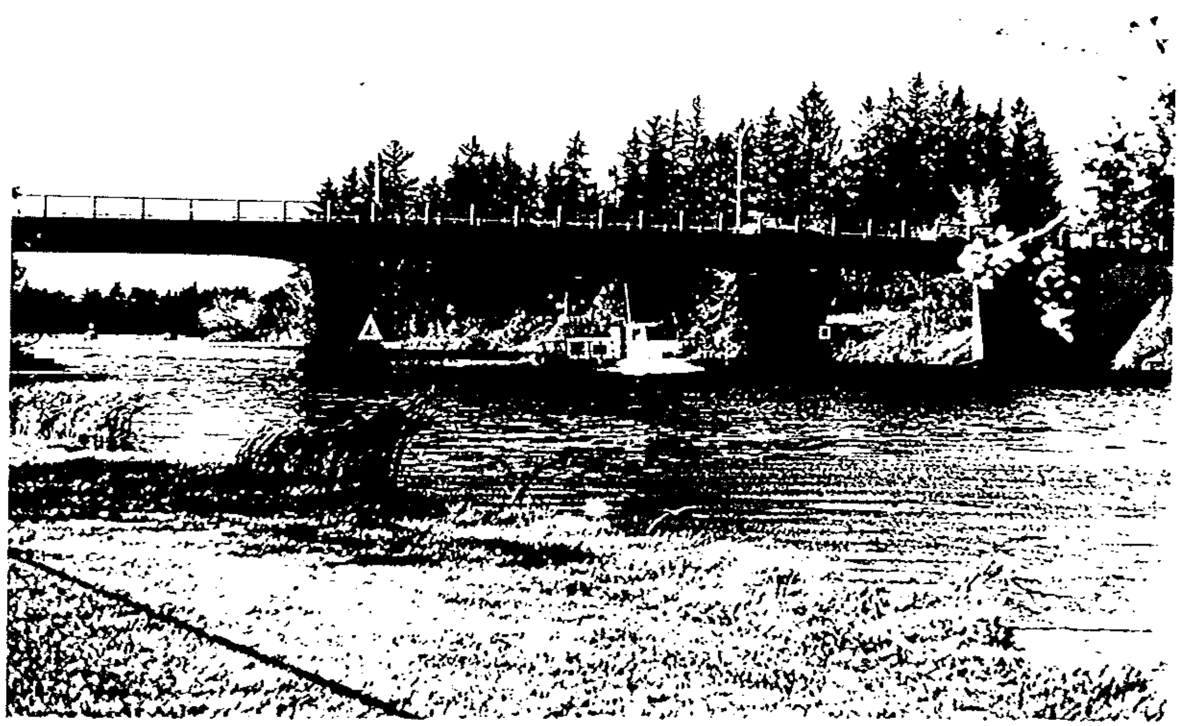
@ 100-year Level

2 - Piers

B. STAGE DISCHARGE CURVE @ Sec. 18.196



C. PHOTOGRAPHIC PRESENTATION: LOOKING DOWNSTREAM



BRIDGE DATA

WATERCOURSE Rideau River

MAP SHEET NO. 10

LOCATION Section No. 15.850

U.T.M. GRID REFERENCE

STRUCTURE Barnsdale Drive Bridge - West Branch

A. SPECIFICATIONS

Span 44.3 m

Length of Structure 12 m

Top of Road Elevation 86.5 m

Low Chord (Soffit) Elevation 85.0 m

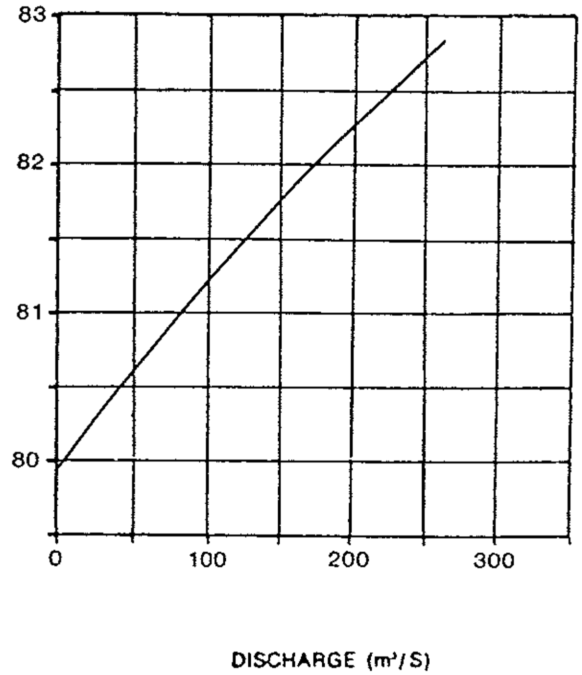
Upstream Invert Elevation 79.1 m

Effective Flow Area 166 m^2

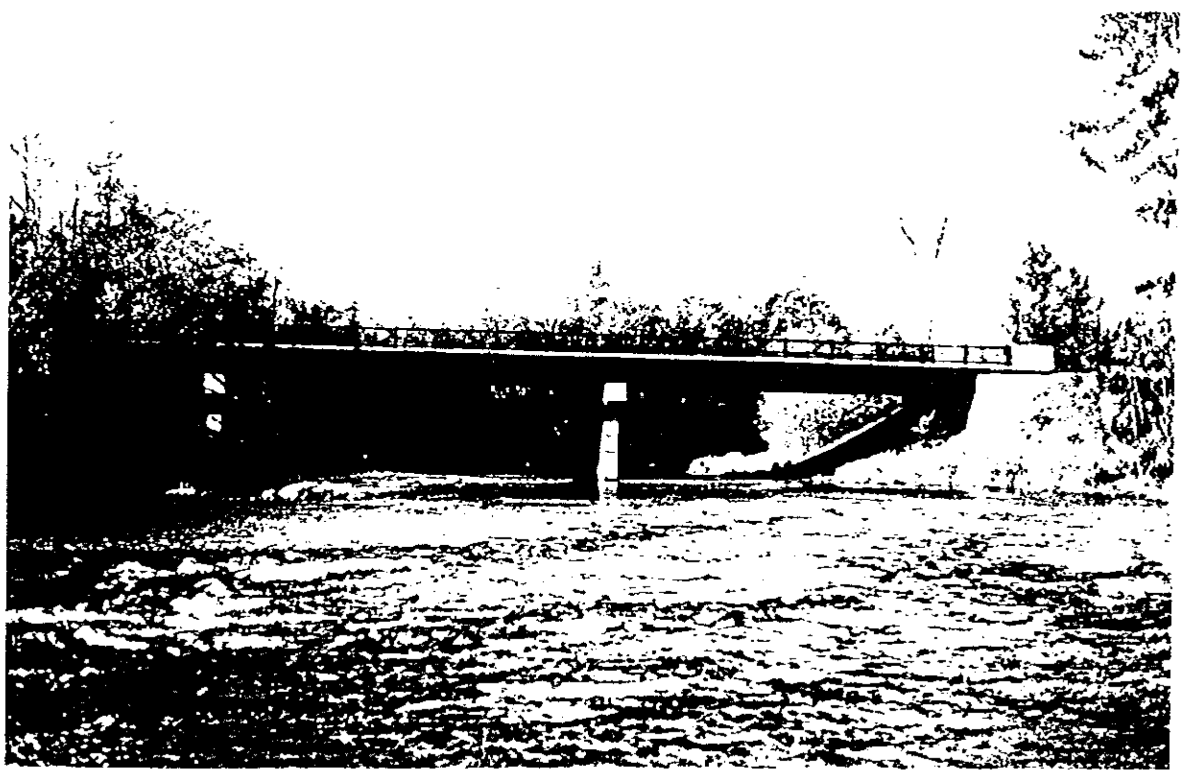
@ 100-year Level

2 - Piers

B. STAGE DISCHARGE CURVE @ Sec. 15.862



C. PHOTOGRAPHIC PRESENTATION: LOOKING DOWNSTREAM



BRIDGE DATA

WATERCOURSE Rideau River

MAP SHEET NO 12

LOCATION Section No. 18.620

U.T.M. GRID REFERENCE

STRUCTURE Manotick Bridge - West Branch

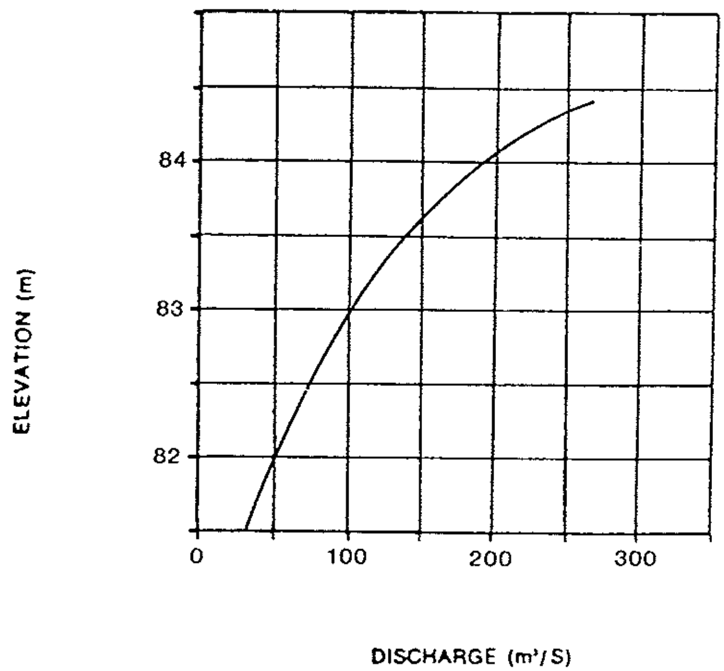
A. SPECIFICATIONS

Span	56.5	m
Length of Structure	14	m
Top of Road Elevation	88.5	m
Low Chord (Soffit) Elevation	87.8	m
Upstream Invert Elevation	80.0	m
Effective Flow Area	166	m^2

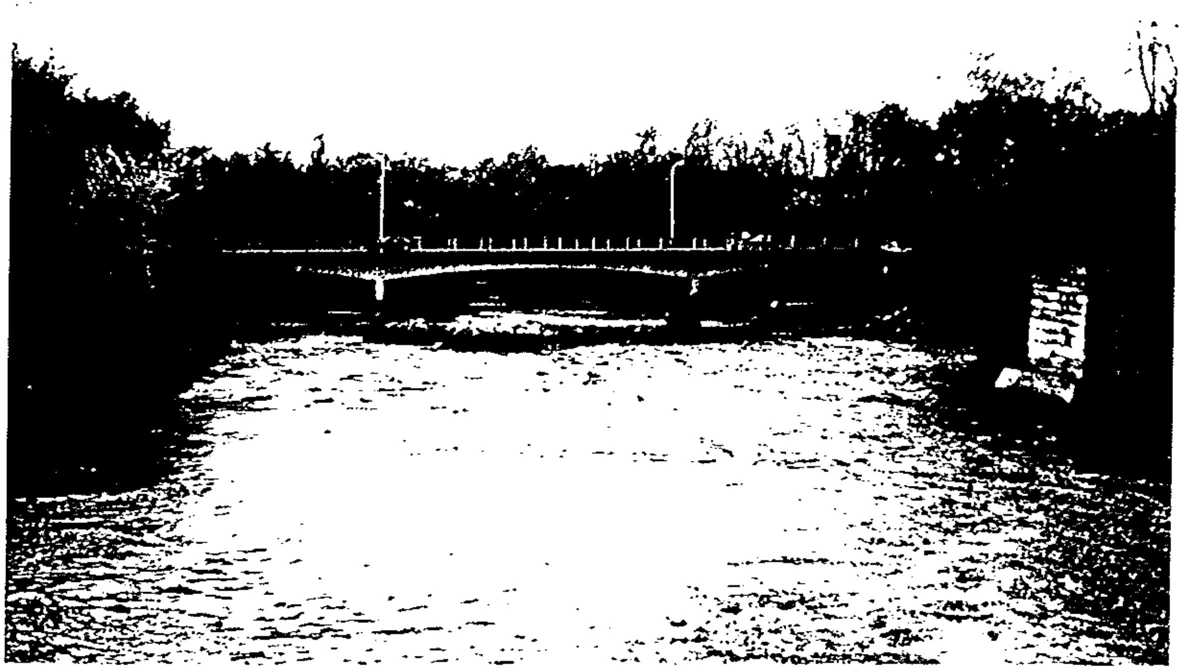
@ 100-year Level

2 - Piers

B. STAGE DISCHARGE CURVE @ Sec. 18.634



C. PHOTOGRAPHIC PRESENTATION : LOOKING DOWNSTREAM



BRIDGE DATA

WATERCOURSE: Rideau River

MAP SHEET NO.: 12

LOCATION: Section No. 18.789

U.T.M. GRID REFERENCE:

STRUCTURE: Manotick Dam - West Branch

A. SPECIFICATIONS

Span: 60 m

Length of Structure: 6 m

Top of Road Elevation: 87.8 m

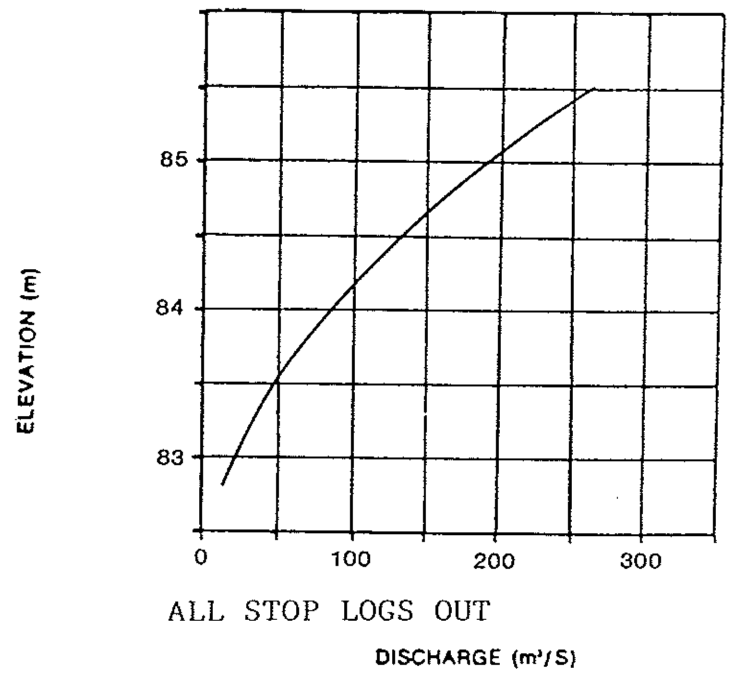
Low Chord (Soffit) Elevation: 87.4 m

Upstream Invert Elevation: 82.3 m

Effective Flow Area: m²

8 - Bay control structures
6 - Operational Bays with stop logs

B. STAGE DISCHARGE CURVE @ Sec. 18.885



C. PHOTOGRAPHIC PRESENTATION: LOOKING UPSTREAM

BRIDGE DATA

WATERCOURSE Rideau River

MAP SHEET NO. 18

LOCATION Section No. 28.715

U.T.M. GRID REFERENCE

STRUCTURE Regional Road 6 Bridge

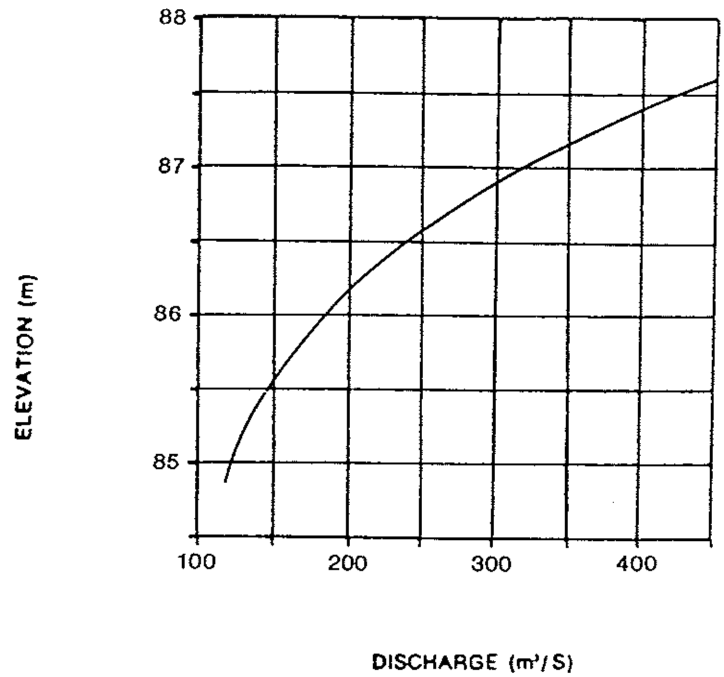
A. SPECIFICATIONS

Span	143	m
Length of Structure	12	m
Top of Road Elevation (Middle)	94.5	m
Low Chord (Soffit) Elevation	92.6	m
Upstream Invert Elevation	77.3	m
Effective Flow Area	846	m^2

@ 100-year Level

4 - Piers

B. STAGE DISCHARGE CURVE @ Sec. 28.727



C. PHOTOGRAPHIC PRESENTATION: LOOKING UPSTREAM