REPORT ON
FLOOD RISK MAPPING OF RIDEAU RIVER
FROM MOONEYS BAY TO RIDEAU FALLS

PREPARED FOR

RIDEAU VALLEY CONSERVATION AUTHORITY

PREPARED BY

A.J. ROBINSON & ASSOCIATES INC.
CONSULTING ENGINEERS

March 1984

A.J. Robinson & Associates Inc.

CONSULTING ENGINEERS
P.O. Box 13130 Kanata, Ontario K2K 1X3
Telephone (613) 592-6060

March 30, 1984

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

Attention: Mr. O. Stirajs, Resources Manager

Re: Flood Risk Mapping - Rideau River
From Mooneys Bay to Rideau Falls

Dear Sir:

We are pleased to submit our report of the above-noted study. It includes chapters on hydrology, hydraulics and engineering assessment of measures to alleviate flooding along the Rideau River through the Cities of Ottawa and Vanier.

Two sets of flood risk maps accompany this report: one set gives the location of the 1:5, 1:10, 1:25, 1:50 and 1:100 year flood lines as well as the fill line location; the other set gives the locations of the 1:100-year flood lines and fill lines.

The mapping related tasks were carried out by Northway-Gestalt Corporation and the field surveys and engineering were undertaken by A.J. Robinson & Associates Inc. with technical advise on ice-related concerns provided by Dr. B. Michel of Laval University.

We thank the Authority for the opportunity to undertake this assignment.

Yours very truly,

A.J. ROBINSON & ASSOCIATES INC.

J.P. Jolly

J.P. Jolly, P.Eng.

/jk

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ACKNOWLEDGEMENTS

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1. SUMMARY

A flood-risk mapping study of the Rideau River from Rideau Falls to Mooneys Bay has been completed for the Rideau Valley Conservation Authority. The flood risk maps provided with this report, which were produced from 1982 aerial photography, replace those prepared for the Rideau Valley Conservation Authority in 1972 by M.M. Dillon Limited and reflect the numerous changes in the watercourse and its flood plains which have occurred over the past ten years. The extent of flooding along the study reaches attributable to the Regional-Flood (100-year return period) has changed due to additional urban development and local differences in the topographic mapping and flood elevations resulting from the two studies. The frequency analyses of maximum annual floods give lower return period flows than the 1972 Dillon Report, but the hydraulic calibration and analyses show higher flood elevations for a given return period flood in some areas and lower water elevations in others. Compared with the 1972-determined topographic mapping and floodline elevations the following general statements are made about the current flood-risk mapping.

The area inundated under the 1:100-year flood in the New Edinburgh area is approximately the same with slight local variations due to differences in the contour elevations between the 1972 and 1982 mappings. In the Kingsview Park area the construction of the NCC bicycle pathway and the Vanier Parkway Extension, in addition to higher than previously surveyed roadway elevations between Wayling Avenue and Tudor Place, cause the flood line to run closer to the river shore in the more recent mapping.

The structural adequacies of the embankments between the Rideau River and the New Edinburgh and Kingsview areas are, however, in question and the areas are designated as "areas of reduced flood risk". Until such time as the embankments can be shown to be stable and impervious under conditions of the Regional Flood (1:100-year return period) the areas should be so designated.

There are fewer homes in the Brantwood Park area affected by the regional flood conditions than shown in the previous study. Higher ground elevations and slightly lower flood elevations in the area bring the flood line closer to the river. In the Rideau Garden Drive area the flood line does not inundate homes as was previously reported in the 1972 study. Field surveys confirm that flooding is contained closer to the river. Slightly higher flood elevations upstream of Billings Bridge result in the flood line extending farther up the banks in the Warrington Drive area with water crossing over Bank Street and inundating the area behind the Windsor Park Dike-Stage I. The flow width over the roadway is not extensive. The current pumping system recently installed with Stage I may be sufficient to drain the flood waters entering behind the dike, but the area must still remain as an area of reduced flood risk. Higher flood elevations in the Warrington Drive area leads to slightly more area inundated under the Regional Flood than in the 1972 flood mapping.

Although a roadway around the Carleton University Sports Centre has been built above the Regional Flood elevation, the area between the roadway and Bronson Avenue still remains below this elevation. The resulting flooded area extends passed the Sports Centre over Bronson Avenue and behind the Brewer Park Dike.

Those areas presently protected by dikes but below the 1:100-year return period flood elevation have been noted on the mapping as a "Reduced Flood Risk Areas", and the 1:100-year elevation plotted according to the policies of the Conservation Authorities and Water Management Branch, Ontario Ministry of Natural Resources. The affected Municipalities should adopt appropriate Official Plan Policies and implement Zoning By-Laws which will clearly define the permitted land uses in these reduced risk areas and the type of constraints which are to apply to developments.

The current mapping in the area of the Brewer Park Dike does not agree with the previously proposed top of dike elevations. The Conservation Authority should complete field surveys in order to establish the existing dike elevations. Similarily, in the area of the dike over the sewer on the west side of Bronson Avenue at Carleton University the mapping does not have spot elevations matching the proposed dike construction elevation given in Dillon's Construction drawings. It is recommended that the Authority resurvey this area.

The most feasible and economical solution to preventing open water flooding with the Regional Flood event is to construct dikes at several locations along the river. The areas considered are: Carleton University, Warrington Drive, Windsor Park - Stage II, Brantwood Park and New Edinburgh.

Ice jams are a major factor contributing to flooding in the Rideau River. Further improvements to the ice jam prevention techniques have been suggested. Additional proposed studies should be completed to determine the feasibility of these techniques.

Control of frazil ice formation at Strathcona Rapids can be accomplished by constructing weirs at the upstream end of Cummings Island to drown out the rapids. Reducing the occurrence of frazil ice formation and subsequent ice thickening could reduce the current ice clearing operation costs and flooding due to ice jamming in combination with higher flows.

2. INTRODUCTION

Flooding and ice jam control on the Rideau River have previously been studied. The preliminary engineering study of the river from the Ottawa River to Kars prepared by M.M. Dillon in July, 1972 [1] recommended dike construction to prevent inundation of valuable residential and commercial lands. Two of the diking schemes proposed have been implemented by the Rideau Valley Conservation Authority. The Brewer Park dike and dike extension have been built for some time and the Windsor Park Dike-Stage I has been recently completed.

The Rideau Valley Conservation Authority requested that a new study be completed to determine the magnitude of the 1:5, 1:10, 1:25, 1:50 and 1:100-year return maximum flows in the reaches of the Rideau River from Rideau Falls to Mooneys Bay (see Figure 1). The study also included computation of water surface elevations in the study reaches associated with the established design flows. Moreover, digitally produced contour mapping at a scale of 1:2,000 with 1.0 metre contours and 0.5 metre auxiliary contours was required with the 1:100-year flood and fill lines plotted on two sets of maps, and the 1:5, 1:10, 1:25, 1:50, 1:100-year flood and fill lines plotted on another set.

An assessment of the morphology of the Rideau River with respect to the formation of anchor ice and ice jams that identify sections of the river which may be particularly susceptible was to be completed. This assessment was to address, in a preliminary manner, the feasibility of carrying out modifications to the river channel to reduce the likelihood of the formation of ice jams. Any proposed solutions were to include preliminary designs and cost estimates.

Many floods large enough to inundate residential properties have been reported since the early 1860s in the study reaches. The majority were due to a combination of high snowmelt runoff flows and ice jammings at channel restrictions. Successful ice cutting and blasting operations dating back to 1887 have been completed to prevent ice jam formation. Unpredictable weather conditions during some spring freshet periods have resulted in the ice sheet not being removed early enough to prevent flooding above the normal open water levels.

The following chapters cover hydrology, hydraulics and a preliminary engineering assessment of reducing the extent of flooding and ice jam formations.

3. HYDROLOGY

Frequency analyses were carried out to determine the specified return period annual maximum flows for the reaches of the Rideau River between Mooneys Bay and the confluence with the Ottawa River at Rideau Falls, where the drainage area is 3,860 square kilometres. Water level readings have been recorded on the Rideau River at Ottawa since 1916, although year-round measurements were not taken until 1947. From 1947 to 1982, inclusive, there are records of annual maximum mean daily flows for the spring flood period with the exception of 1948. Hence there are 35 years of data for use in frequency analyses of annual maximum mean daily flows. Maximum instantaneous flows have been recorded since 1971 - except for 1974 - providing ten years of instantaneous maximum flow data.

The annual maximum flow series is given in the Historical Streamflow Summary [2] provided by the Water Survey of Canada (WSC), Environment Canada. Analyses were undertaken on these flows, which result from natural occurrences, modified by either the past or present rule curves applied to the Rideau River at the Poonamalie hydraulic control structure.

Environment Canada, Flood Damage Reduction Section has published Schedule B - Hydrologic and Hydraulic Procedures for Flood Plain Delineation [3] which recognizes that instantaneous peak flows should be used when analysing an annual maximum flow series. The observed annual maximum mean daily flows and maximum instantaneous flows are given in Table I. Comparing maximum instantaneous with maximum mean daily flows for the Rideau River at Carleton University (Station No. 02LA004) reveals that the ratio of these two quantities varies from 1.013 to 1.096 with an average of 1.053. Both linear and non-linear regression analyses were carried out between the ratio of the two statistics and the annual maximum mean daily flow; however, no meaningful relation was found. Thus the arithmetic mean is considered to be the best estimate. The recorded maximum mean daily flood flows before 1971 and the 1974 value were multiplied by this ratio to obtain a maximum instantaneous flow series for the 35-year record. These flow values are also listed in Table I.

The major flood storage reservoir on the Rideau River is in and upstream of the Big Rideau-Lower Rideau Lakes where the water levels are controlled by a hydraulic structure at Poonamalie, upstream of which is one third the total basin drainage area. This reservoir is lowered in the autumn and winter annually and is replenished during the spring runoff period; hence the flows at Ottawa during the spring runoff periods are not a direct function of the runoff from the total basin. The influence of the stored water in the Big Rideau-Lower Rideau Lakes reduces the flow magnitudes at Ottawa during the winter and spring runoff periods, which is when the maximum annual floods occur.

In 1977 the rule curve applicable to the hydraulic control structure at Poonamlie was changed [4]. Previous to 1977 the reservoir levels were controlled almost exclusively to enhance water conservation for summer operation of the Rideau Canal System. The policy was subsequently changed to provide as much control as possible in the high runoff periods so that the downstream flood magnitudes would be reduced.

The Poonamalie flow records for the period before 1972 are available from the Rideau Canal Authority; however, the flows had been estimated from stop-log settings as well as head measurements and are not considered accurate for the purposes of this analysis and therefore not considered further.

The annual maximum flows at Poonamalie and those passing through Ottawa from 1972 to the present (excluding 1974) are listed in Table II. Travel times between the two locations were not considered in comparing the two peaks. Although the times of peak occurrences at Poonamalie are known, the times of the corresponding maxima of the Poonamalie flood waves when they arrive at Ottawa are not known because of the large volumes of lateral inflow during flood events between the two locations from a drainage area twice the size of that above Poonamalie. From Table II it can be seen that for the period of common record before 1977, the ratio of annual maximum mean daily flow at Poonamalie to annual maximum mean daily flow at Ottawa varies from 0.069 to 0.139 with an average of 0.096. After 1976 the outflow from Poonamalie was restricted during spring freshets. This resulted in maximum flow ratios (Table II) ranging from 0.0 to 0.081 with an average of 0.028.

The maximum instantaneous flows of the Rideau River at Ottawa, which are listed in Table I, were altered in order to reflect the present or the past rule curve applied at Poonamalie. The first adjustment considered was to modify the before 1977 flows to reflect the new operational procedure. Instantaneous maximum flows at Ottawa (Table I) should then be reduced by an average of seven percent (Table III). The second condition is to increase maximum instantaneous flows at Ottawa (Table I) from 1977 to 1982 by seven percent (Table III). The latter adjustment was applied to determine the design flood flows in the study reaches, since it produces higher flood magnitudes and there is no guarantee that the current Poonamalie rule curve will continue to be applied. "Schedule B", cited above, gives requirements in a section on flood frequency analysis for converting regulated streamflow to naturally occurring ones. Accurate, continuous records of measured stages and corresponding flows for the period of record are required to complete such an analysis. Accurate water level records for most reservoirs in the Rideau River System are only available from 1978 to the present. Since the records being studied are for the 1947; 1949-1982 years, this type of analysis cannot be undertaken.

Frequency analyses were carried out on the annual maximum instantaneous flow data listed in Tables I and III using the computer program: Flood Damage Reduction Program Flood Frequency Analysis, Water Planning and Management Branch, Inland Waters Directorate, Environment Canada [5]. Computer printouts accompanying this report give statistics of these analyses.

Figure 2 is a histogram of annual maximum mean daily flows of the Rideau River at Ottawa for the 1947; 1949-82 years. The largest recorded spring flood flow is 583 m^3/sec and the lowest is 109 m^3/sec. The frequency plot of maximum instantaneous flows given in Table III is shown in Figure 3. It may be seen from the figure that the fit is not good. The analysis shows that the coefficients of skewness for the data are very negative and there is no distribution which closely fits the data. Upon review of the data of the maximum mean daily flows for the years 1957, 1961, 1964, 1965 and 1966 it was noted that these flows were very much below normal, all less than 225 m^3/sec.

Low flood flows were recorded in other basins in eastern Ontario during these same years, mainly in the South Nation River at Plantagenet Springs and the Mississippi River at Appleton. The annual maximum flow data for the same years for the two neighbouring basins were examined, which have drainage areas of 2900 and 3810 square kilometers, respectively, and are approximately the same size as the Rideau River basin. The histograms of annual maximum daily flows are given in Figures 4 and 5 for these two streams.

From Table IV it may be seen that the five lowest flood years on the Rideau River were also the same five lowest flood years on the South Nation River, although their serial ranking is not the same. Four of the five lowest flood years on the Mississippi River are the same as on the Rideau, although again they are not in the same order. Records of flow kept by the Rideau Canal Office of Parks Canada in Smiths Falls indicate that during the five lowest flood years either problems were encountered in filling the Rideau River System reservoirs or flows out of the reservoirs were severely restricted in order to try to fill the reservoirs for summer regulation.

Statistical tests based on the normal probability distribution were performed on the Rideau River annual maximum mean daily flows for the period of record considered to determine the existence of flow outliers (Appendix A) which were tested at the 5, 2.5 and 1.0% significance levels. There are no high outliers; however, the maximum flows of the five lowest flood years were found to be low outliers at the 2.5 % significance level. In other words, one can be 97.5 % confident that flows lower than 244 m^3/sec. - the cut off point for outliers - will occur with a probability of less than 3.3 % in any year. The same outlier tests were carried out on the South Nation and Mississippi River flood data, and the maximum mean daily flows of the five lowest flood years of the same period of record were also found to be outliers of the 5 % significance level but not at the 2.5% significance level (See Appendix B). The differences in the critical significance levels between the Rideau River flood series and those of the other two streams most likely represent the higher degree of storage on the Rideau compared with the other streams.

The annual maximum flood data recorded at the Hurdman Bridge hydrometric gauge (Station No. 02LA002) from 1947 to 1966 and at the Carleton University hydrometric gauge (Station No. 02LA004) since 1967 were tested for consistency of gauge data. This was completed by the Dalrymple method [6]. The data sets are homogeneous, hence the data collected at the two stations are consistent.

The recorded annual maximum instantaneous flows increased by seven percent are given in Table II and plotted in Figure 6 along with the three-parameter lognormal curve which fits this data. The fit of the distribution is very good, only the thirty-year data set was used, ie., the lowest five years of data were not included. A copy of the computer printout accompanies this report.

On the instructions of the Conservation Authority's staff two- and three-parameter Weibull probability distributions [7] were applied to the data. The two-parameter distribution gave a poor fit; the three-parameter distribution fitted the observed data better. Compared with the three-parameter log normal distribution, which was determined by excluding the five lowest floods, the three-parameter Weibull better fits the observed data at the lower end of the flood range. A weakness of applying the latter distribution is that the lower bound is a negative

460 m^3/sec which is impossible physically. On the other hand, it is difficult to accept that there could be five low flood outliers in thirty five years of record. Hence a hydrologist is faced with the difficult task of choosing the better of the two distributions, which agree relatively well with the observed data in the higher flood range, but not at the lower. The three-parameter Weibull distribution gives a 1:100-year flood magnitude of 623 m^3/sec; the three parameter log-normal distribution gives 654 m^3/sec.

In order to decide which curve fits the flow data better the standard errors of estimate were calculated. The standard error of estimate obtained by the fit of the three-parameter log normal (exluding the five lowest floods) to the observed data is 12.4 m^3/sec; the same statistic obtained for three-parameter Weibull for the same range of flood values is 10.6 m^3/sec. The difference in these values were tested with the F-statistics test, and they were found not to be significantly different. Thus the two probability distributions fit the observed data equally well in the range of the design return periods. The difference in maximum flood magnitudes in the return period range investigated increases with the return period. The difference is zero percent at the 1:5-year and five percent at the 1:100-year flood.

To be on the conservative side the values of the larger set of flood magnitudes (three-parameter log normal) will be used in the calculations of the various prescribed water surface profiles. The flow magnitudes for the specified return periods are given below.

Average Return Period (Years)	Three-Parameter Log Normal Distribution Flows (m^3/s)
5	513
10	552
25	598
50	626
100	654

MEAN ANNUAL FLOOD BASED ON RECORDED MAX-INSTANTANEOUS FLOOD AS RECORDED @ STATION [illegible], 1913-82

The 1972 flood risk mapping study gave a 1:100-year flood flow of 737 m^3/sec which was determined with a Gumbel probability distribution applied to 1916 - 1972 mean daily data collected only during open-water conditions. Moxley [8] questioned the applicability of fitting the Gumbel distribution to the flood data and recommended another curve which more closely fits the fifty-four year data set. His 1:100-year flood estimate is 609 m^3/sec based on a probability distribution with an upper limit which is not acceptable on geophysical grounds.

The three-parameter log normal distribution is used in the subsequent analysis because it was carried out on maximum instantaneous flow statistics, which were collected during the complete spring runoff periods, and because it was developed from the most recently available data (to 1982) that excludes low flood outliers and the distribution has a low coefficient of skewness.

The study reach is approximately eleven kilometres long and the local lateral runoff is drainage from a 33 square kilometre area - less than one percent of the total drainage area. Flood flows are measured with precisions varying between ninety and ninety-five percent. Thus any lateral increases in flow to be considered along the study reaches are within the bound of precision that the maximum flood flows are known. Hence it is not technically sound with the available data to increase the flows along the study reaches for use in the backwater computations.

4. HYDRAULICS

4.1 General

A flood profile was established by M.M Dillon Limited[1] in 1972 for the section of the Rideau River between the Ottawa River and Kars - a distance of 39 kilometres. The floodplain mapping produced was registered under Ontario Regulation No. 875/76 (as amended by Ontario Regulation 52/80) for flood elevations based on the Regional Flood (100-year return period) flow magnitude of 736 m3/sec. This flow was obtained through flood frequency analysis of published and unpublished flow and stage data from the previous hydrometric gauging station at the Canadian Pacific Railway Bridge upstream of Hurdman Bridge and the current Ottawa hydrometric station adjacent to Carleton University.

A subsequent study by Dillon in 1977 [9] carried out several statistical fitting techniques on the recorded flood flows. The analysis essentially confirmed the previously reported [1] maximum flood flow magnitudes and corresponding water surface elevations.

Stage measurements and corresponding discharges have been available for the Rideau River at Hurdman Bridge since 1916 with continuous recordings since 1946. Stage recording at this location was discontinued in 1966. The established rating curves of stage-discharge at the present Carleton University hydrometric gauge (No. 02LA004) and the former gauge site at Hurdman Bridge (02LA002) have provided a source of observed water level information for ice-free conditions.

4.2 Methodology and Assumptions

Cross sections of the river below the 1982 summer water surface elevations were obtained at representative reaches to check and supplement the bathymetry completed by the Canadian Hydrographic Service for the NCC in 1970. The cross-section elevations were tied into Geodetic Datum. Existing structures, such as bridges, dikes, etc., which affect the floodline were photographed and surveyed. Field data obtained included a survey of dimensions and elevations. Bridge data collected by M.M. Dillon Limited in 1972 was verified and used in this study. The photographs of the bridges as well as dikes and the bridge data sheets are given in Appendix E.

Topographic mapping at a scale of 1:2,000 along the Rideau River from the Ottawa River to the Hogs Back Road was completed by Northway-Gestalt Corporation. The mapping strip width varies along the study reaches to include sufficient contour information to delineate the floodlines and associated fill lines. Cross-sections of the river and floodplain above the water surface were taken from the mapping and supplemented by field surveys.

The starting water surface elevations at the Rideau Falls East and West Dams, used in backwater computations, were the critical depths over the two dam sills, since the latters are close enough to the falls to act as broad-crested weirs. When flood conditions prevail all the stop logs from both dams are removed. Since the five-year return period flood is at bank-full stage or higher, these assumptions of critical depths and complete removal of stop logs are valid.

The east and west channels upstream of the Rideau Falls were considered separately to determine each channel effects on flood profiles. The flow split of 28% in the west channel and 72% in the east channel was determined through successive computer runs.

The water surface profiles for the specified return period floods were computed for the compiled Rideau River model using the current version of HEC-2, the backwater analysis program developed by the Hydrologic Engineering Center, Corps of Engineers, U.S. Army, Davis California [10]. Collection and processing of data, computational procedures and analysis of computed profiles meet the criteria and guidelines published by the Hydrologic Engineering Center User's Manual and Training Documents [10].

Along the study reach Manning's roughness values ("n" values) were selected for the main channel and left as well as right overbank areas from field observations and reference [11 & 12] and used as initial estimates for all return period flood flows. Adjustments to the "n" values were made in order to calibrate with the stage-discharge information for the previous Hurdman Bridge and the present Carleton University hydrometric gauges. Coefficients for bridge hydraulic loss computations were selected from the HEC-2 User's Manual [10] for flow contraction, flow expansion, pier shape and coefficient of discharge.

The HEC-2 river model at bridge sections was coded with the necessary information for the Normal Bridge or Special Bridge Routines. Initial HEC-2 computer runs indicated that the 1:100-year flood water level would reach the level of the low chords on several bridges, namely, Cummings Bridge and the Old Hurdman Bridge because of the arched bridge openings. The Cummings Bridge was simulated using the Normal Bridge Routine because of the arches. All other bridges were simulated using the Special Bridge Routine.

The existing Brewer Park Dike and the section of the recently finished Windsor Park, Stage I Dike were coded into the HEC-2 River model, and flood profile computations completed.

4.3 Calibration

The points of calibration for the HEC-2 river model are the known and extrapolated stage-discharge curves for the old Hurdman Bridge gauge and the Carleton University gauge. There have been four stage-discharge curves developed by WSC for the Hurdmans Bridge hydrometric gauge throughout its period of operation - 1916 to 1966. The reason for the different curves is because of changes in the river regime and hydraulic control points, which occur in every river especially those that traverse urban areas. The latest developed WSC stage-discharge rating was developed from current metering data collected in the 1960 to 1966 period. Since it is based on the most recently available data, it is used for hydraulic calibration because it should closely represent the existing river regime.

The stage versus discharge points for the designated return periods flows are listed for each gauge on the following page.

Rideau River
at Ottawa
1947; 1947-1982
Station No. 02LA004
Histogram of Annual Maximum
Daily Flows

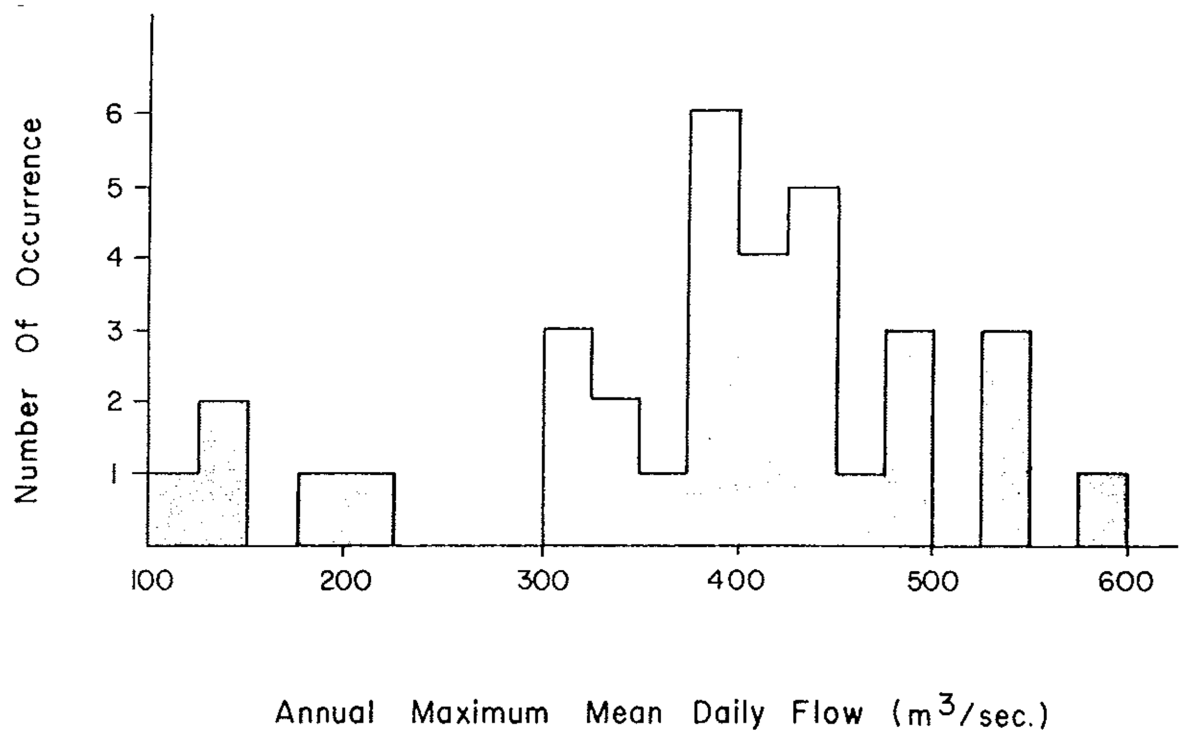

Figure 2

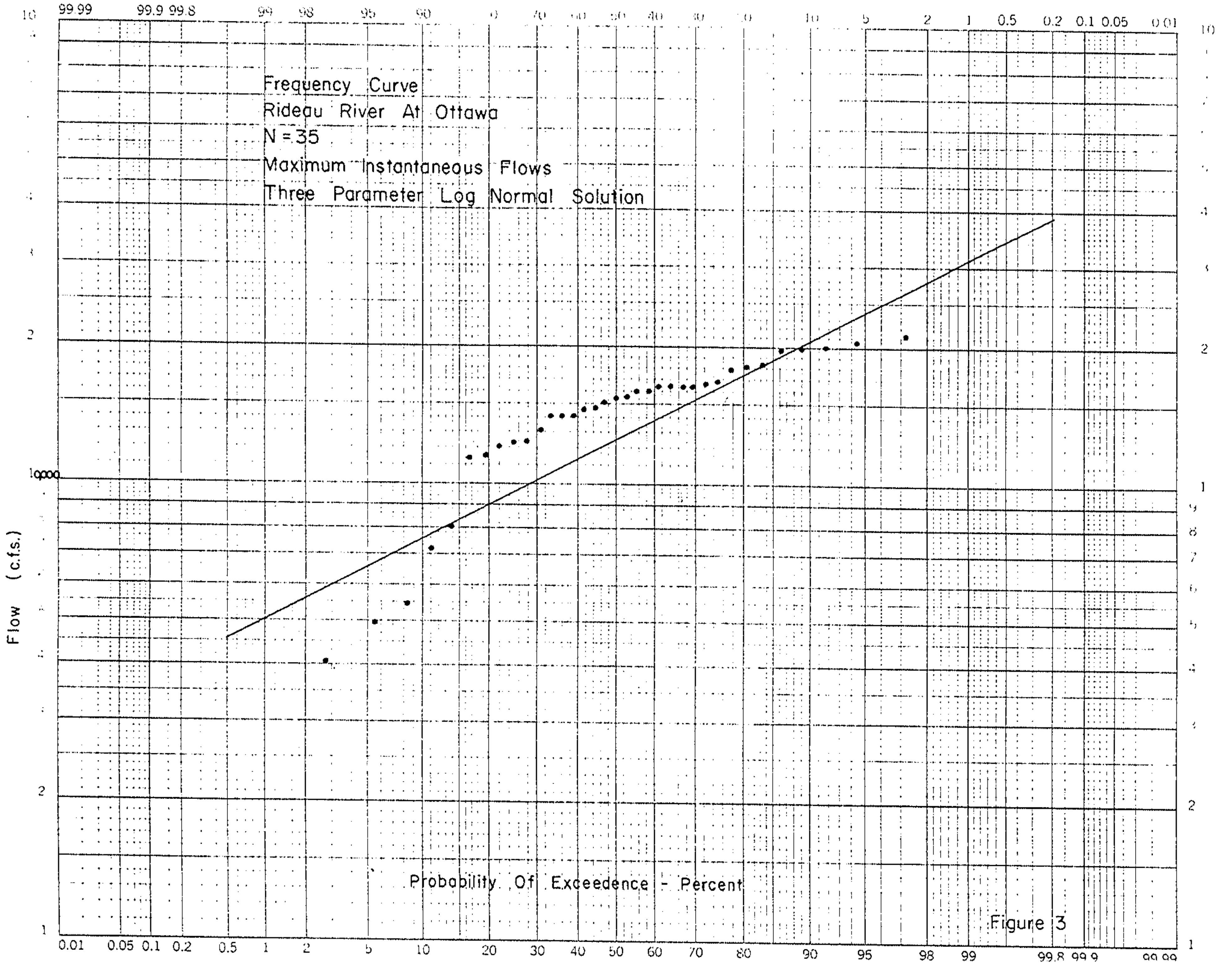
Frequency Curve
Rideau River At Ottawa
N = 35
Maximum Instantaneous Flows
Three Parameter Log Normal Solution
Flow (c.f.s.)
10000
Probability Of Exceedence - Percent

Figure 3

SOUTH NATION RIVER

NEAR PLANTAGNET SPRINGS

1947;1949-1982

Station No. 02LB005

Histogram of Annual Maximum

Daily Flows

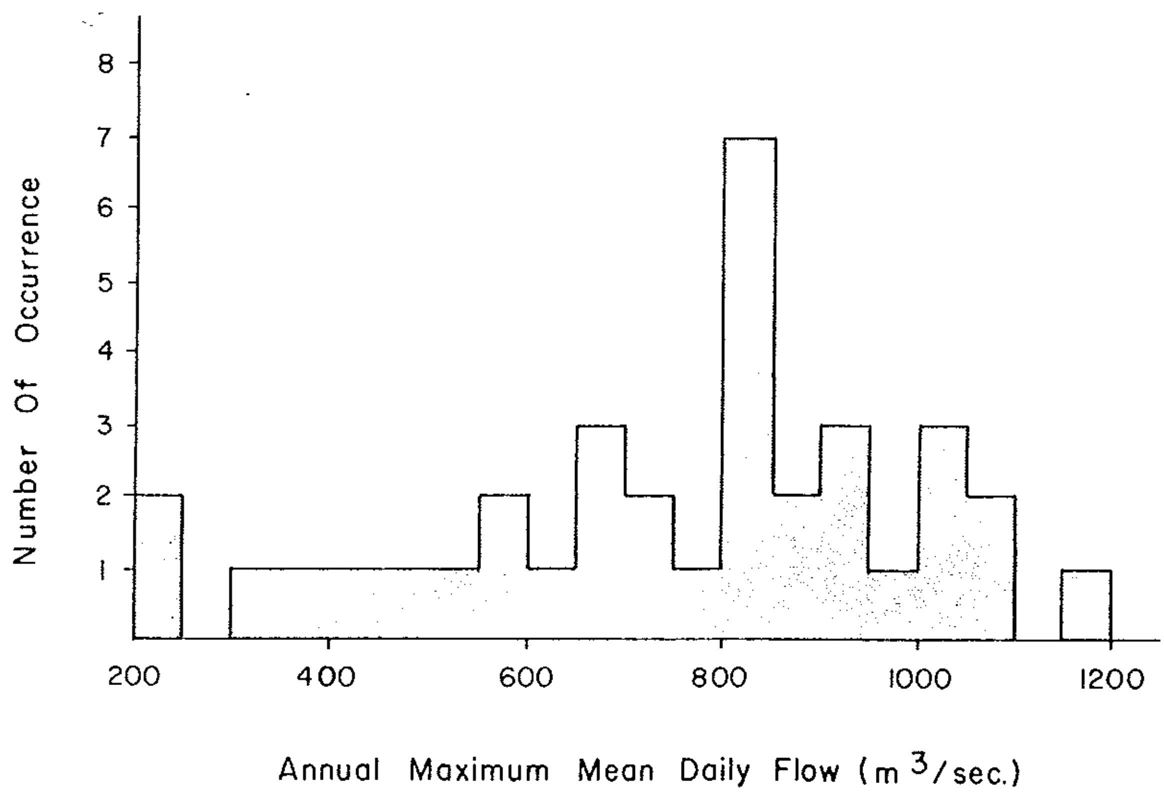

Figure 4

Mississippi River
at Appleton
1947;1949-1982
Station No. 02KF006
Histogram of Annual Maximum
Daily Flows

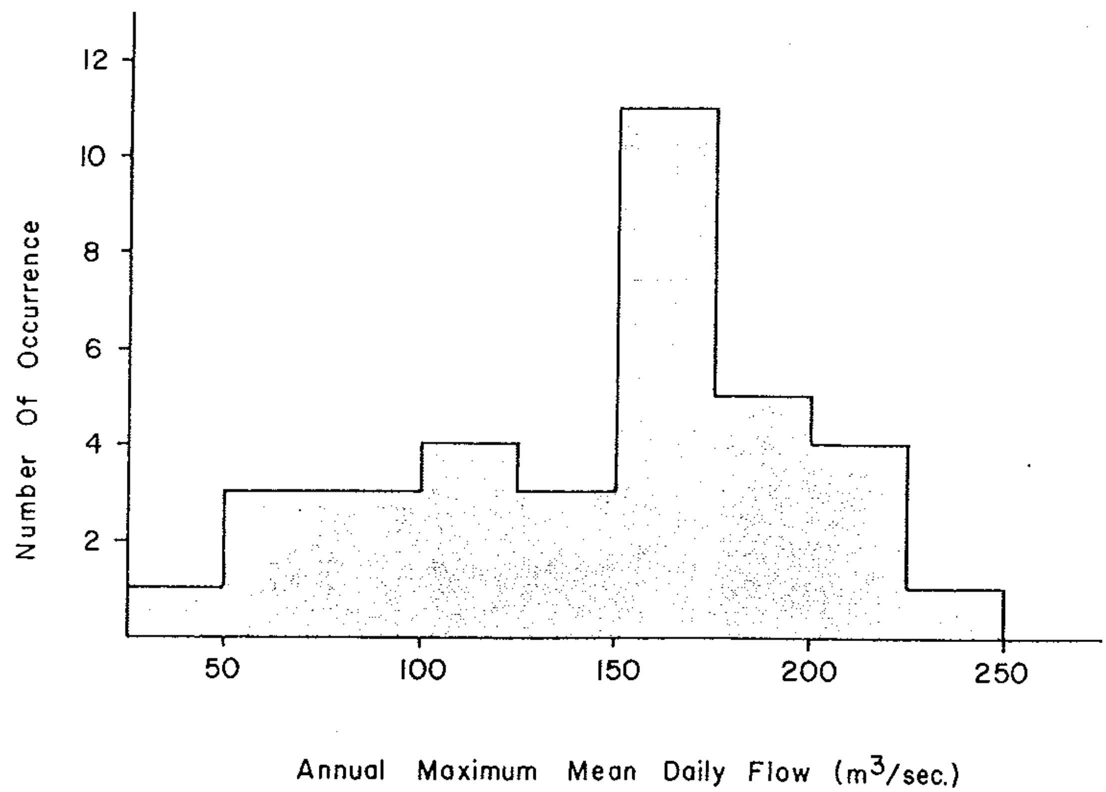

Figure 5

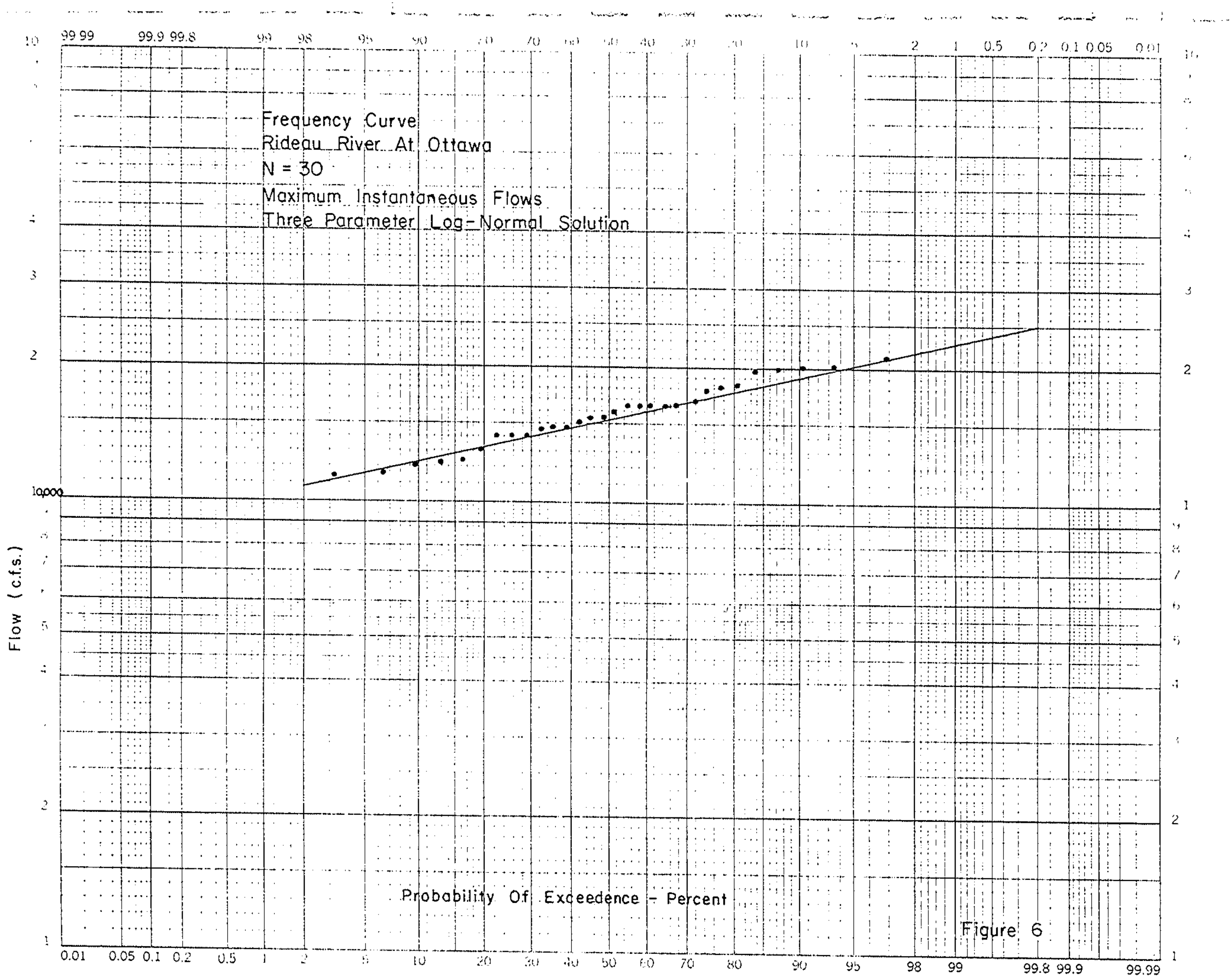
Frequency Curve
Rideau River At Ottawa
N = 30
Maximum Instantaneous Flows
Three Parameter Log-Normal Solution
Probability Of Exceedence - Percent
Flow (c.f.s.)
10,000

Figure 6

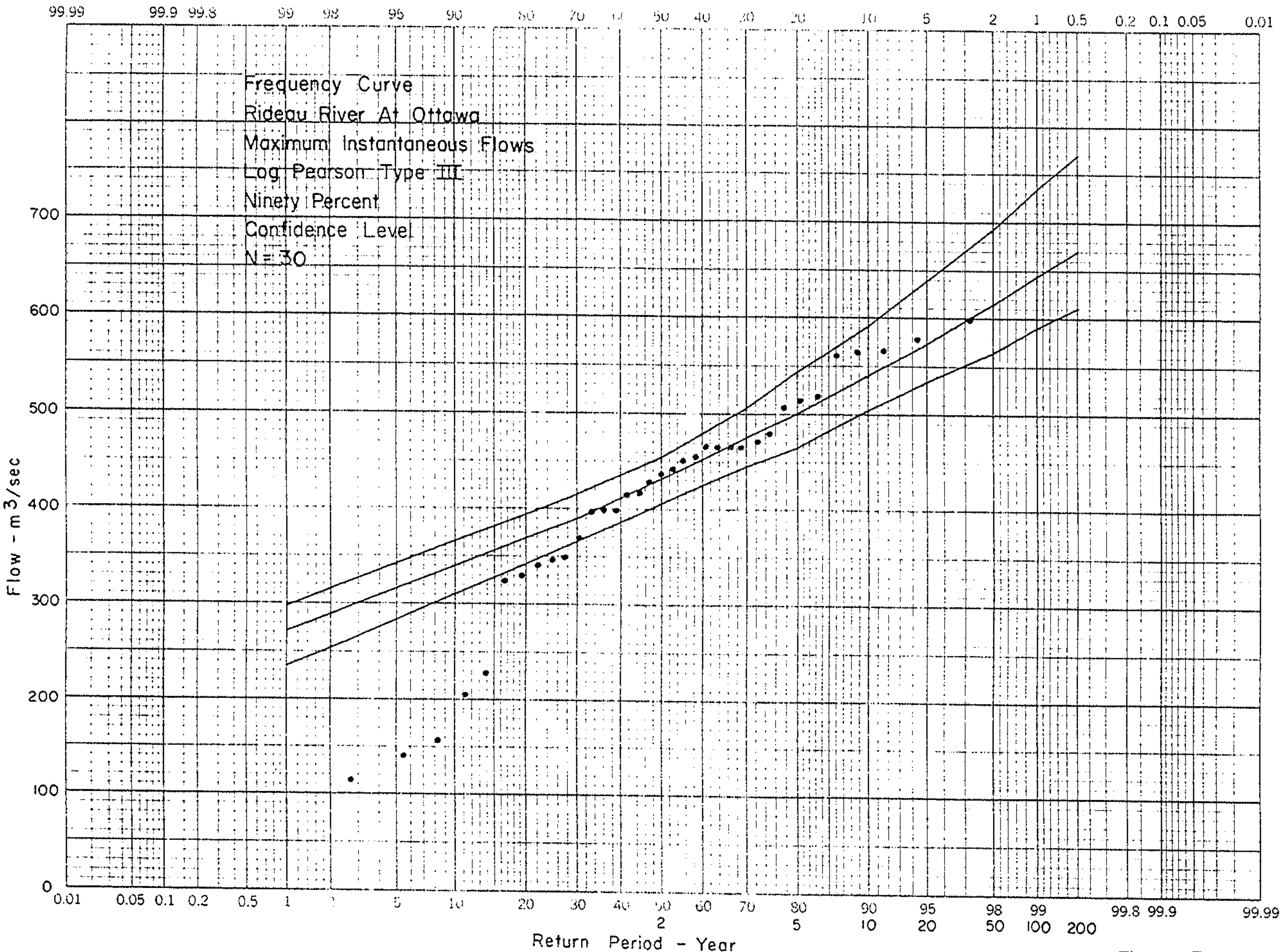
Frequency Curve
Rideau River At Ottawa
Maximum Instantaneous Flows
Log Pearson Type III
Ninety Percent
Confidence Level
N=30
Flow - m3/sec
Return Period - Year

Figure 7

Return Period (yr)	Maximum Flow (m3/sec)	Hurdman Bridge Gauge GSC Elevation Rating Curve (m)	Hurdman Bridge Gauge GSC Elevation Model Calibration (m)	Carleton University GSC Elevation Rating Curve (m)	Carleton University GSC Elevation Model Calibration (m)
5	513	57.45	57.44	60.08	60.06
10	552	57.56	57.56	60.20	60.20
25	598	57.70	57.70	60.34	60.35
50	626	57.79	57.78	60.42	60.41
100	654	57.87	57.85	60.50	60.49

The initial estimates of the Mannings roughness "n" values were too low, and adjustments were made to the "n" values in the main channel and some overbank areas. The Special Bridge Routine was used in computing bridge losses in order to account for the presence of piers at most bridges. The Normal Bridge Routine was used for the Cummings Bridge because of the sloping bridge deck and the multiple-arch openings between piers, since they cannot be approximated by a representative trapezoid for use with the Yarnell equation for low flow in the Special Bridge Routine.

4.4 Design Flood Profiles

The design return period flows were used to compute water surface elevations at various points along the river. The final flow split in the east and west channels around Green Island upstream of Rideau Falls was 78 and 28 percent, respectively. The final adjusted Manning's roughness ("n") values provided a good calibration of the HEC-2 river model. The model accuracy at the old Hurdman Bridge gauge was +-0.01 m, and +-0.02 m at the Carleton University gauge for all return period flood profiles. The water surface elevations at various points along the river for the design flows are listed in Table VI.

Two sets of flood risk maps accompany this report. One set has all the prescribed return period flood and fill lines plotted; the other has the 1:100-year flood and fill lines. The fill lines, which restrict the placement of fill adjacent to the flood lines, have been set at approximately 15 metres back from the flood lines or of the top of slope if a flood line crosses an embankment. As much as possible the fill lines have been set along property or house lines.

4.5 Changes in Flood Line Elevations

The Regional Flood (1:100-year return period) line has been plotted on the 1:2,000 scale, digitally produced topographic mapping completed by Northway-Gestalt Corporation (Sheets Nos. 1 through 16) which accompany this report. The new floodline corresponds to the floodline established in 1972 in most areas through the study reaches.

Differences in water elevations are due to the different Hurdman Bridge rating curves used in the calibrations and the lower maximum flows for various return period flows used in the current study. In other reaches the water surface elevations for various return periods are higher in the current study because of the additional calibration carried out with the Carleton University gauge data.

In the New Edinburgh area slightly higher Regional Flood elevations result in more lands being inundated. Several more houses are within the floodplain than were shown on the 1972 mapping.

The previous mapping showed a rather large flooded area in Kingsview Park. Due to the recent construction of the bicycle path by the NCC and the Vanier Parkway extending along the original River Road alignment in this area as well as across St. Patrick Street, the formerly designated flooded area in the City of Vanier around Landry and Charlevoix Streets is no longer inundated with the occurrences of the 1:100-year flood. The structural adequacy of the embankments between the Rideau River and this area is, however, in question and the area is designated as an area of reduced flood risk. Until such time as the embankment is found to be stable under conditions of the Regional Flood (1:100-year return period) the area should be so designated.

Field surveys were completed in the area of Tudor Place and Wayling Avenue in the Kingsview area. The ground elevations obtained in the field show that the Regional Flood line is contained close to the right river bank.

In the area of the Rideau Tennis Club, extensive gabion walls have been constructed to prevent damage from ice since the previous mapping was undertaken. The new 1:2,000 mapping shows that the Regional Flood line does not extend as far up the river bank as was shown on the 1972 mapping in this area. The computed flood elevations are lower in magnitude resulting in a substantially smaller flooded area.

In the Brantwood Park area the slightly higher flood elevations and the higher topographic elevations lead to smaller flooded areas included within the floodplain for the Regional Flood. One slight difference is that ground elevations in the area of the intersection of Centennial Blvd and Bullock Ave are higher on the new mapping, which exclude a number of houses from the flood plain. The designated flood plain in the area of Rideau Garden Drive is not as extensive as formerly due to higher ground elevations on the new topographic mapping. Several houses are no longer situated within the Regional Flood line.

In the area of Rideau River Drive and Smythe Road, the extension of the roadway and filling of the area adjacent to Smythe Road means higher land elevations prevents some flooding.

A portion of the 1972 proposed Windsor Park Dike has been recently constructed to prevent flooding in the area of Windsor Ave and Riverdale Ave. The topographic mapping and construction drawings for Bank Street at Riverdale Ave show a low point in the road which is below the Regional Flood elevation. The area behind the Windsor Park Dike - Stage II will still be subject to minor effects of the Regional Flood. The current flood risk mapping in the area of Warrington Drive and Harvard Ave is more extensive than on the 1972 mapping. The present computations give a flood elevation approximately 0.15 m higher than the old floodplain mapping.

The Brewer Park Dikes were built high enough to prevent the Regional Flood from flowing over the dikes, however, at present the flood waters will flow over Bronson Avenue from the Carleton University area and inundate areas behind the dikes. The designated flooded area around Carleton University shown within the floodplain on the 1972 mapping is slightly larger in the current mapping.

5. PRELIMINARY ENGINEERING ASSESSMENT

5.1 Introduction

The Rideau River downstream of Hogs Back Road has been subjected to serious flooding during snowmelt runoff periods since the former community of Bytown was settled. The City of Ottawa has carried out blasting operations to eliminate ice jams, and hence alleviate flooding, since the 1880s. Each year throughout the 1900s the City of Ottawa has normally dynamited the ice cover before the main flood occurs in the spring. These blasting operations start at Rideau Falls and progress upstream to Billings Bridge. Where there are electric conduits and water mains ,etc., buried beneath the stream bed, keys are cut beforehand traversely across the stream so that the conduits will not be damaged. When the ice sheets between the keys are subsequently blasted the broken ice pans flow downstream until they enter the Ottawa River.

The details of these ice clearing operations will not be described here; however, these operations are dangerous as well as costly, and one of the tasks of this study is to consider in a preliminary way means of improving these operations and recommending feasible alternatives to prevent flooding in the Rideau River through Ottawa and Vanier.

The two main categories of remedial measures of eliminating or reducing flooding are: 1) improving ice clearing operations, and 2), change the flow regime of some river reaches so that water surface elevations will be lower for a particular flood magnitude in either open-water or ice-cover conditions.

The water surface slopes in the downstream half of the eleven kilometre long study area are in the order of 1:10,000 m/m and in the upstream half about 1:1,000 m/m. There are two major rapids in the study reaches, Strathcona Rapids and those adjacent to as well as upstream of Carleton University. There are also small rapids in the reach from Strathcona Rapids to Hurdman Bridge and some smaller ones occur upstream of Billings Bridge. The smaller ones are submerged during high flows; however, the larger rapids at Carleton University and Strathcona Park are at locations where frazil and anchor ice can readily form during relatively high December flows (approximately 100 m3/sec). The high velocities at the rapids and the river and atmospheric thermal regimes during freeze up periods, cause frazil ice and hanging ice dams to form downstream of the rapids. These ice formations raise the water levels during subsequent spring runoff periods. The normal ice sheet thickness can be one metre or more, and the occurrence of frazil and hanging ice dams cause greater ice thicknesses and ice jamming in some locations.

The various alternatives to eliminate flooding or reduce flood levels considered in this study are:

1. Controlling the upstream flow entering the reaches during runoff conditions;
2. Divert flow through the Rideau Canal downstream of Hogs Back Road during high flow occurrences;
3. Flow control during ice sheet formation;
4. Modifying the current ice clearing operations that are carried out to eliminate ice jam formation;
5. Raising the water level at Strathcona Rapids to drown out the rapids during periods when frazil ice forms;
6. Excavating the channel at one or more hydraulic control

points to reduce the flood levels; and

7. Building dikes along the reaches where there are flood-prone areas.

These alternatives will be considered in greater detail in the following paragraphs.

5.2 Control of Upstream Flood Releases

The Rideau River drainage basin is 3860 square kilometres of which 1273 square kilometres are upstream of the Poonamalie control structure. The outflow from Big Rideau-Lower Rideau Lake has been reduced to about 10% of the corresponding flow at Ottawa during the passage of flood peaks in Ottawa since 1976. One additional means of reducing the flood peaks, and in turn the flood levels in the study reaches, is to store water during the passage of the main stem flood in an additional reservoir, or reservoirs, constructed on the Jock River or Kemptville Creek or both.

The drainage basin of the Jock River is 588 square kilometres, 549 of which are upstream of the hydrometric gauge near Moodie Drive. This basin represents about 21 % of the 2560 square kilometres of uncontrolled drainage area downstream of Poonamalie. The Jock and Rideau Rivers usually peak concurrently during spring runoff, and the average ratio of the Jock River to Rideau River recorded maximum flood flows is 25%. Because of these characteristics there would be a flow magnitude reduction in the study reaches if a control structure were built at Richmond. Acres Report [13] states that such a structure would reduce the peak outflow by 23 m^3/sec for the 1:10-year flood and no substantial reduction would be realized with with floods of 15 years or longer recurrence intervals. The storage volume available is 41 million cubic metres [MCM]. Acres' report stated that the 1:10 year flood elevations would be reduced by 0.08 metres in the study reaches.

There may be a storage site on the reaches downstream of Richmond on the Jock River; however, the drainage area between Richmond and the confluence with the Rideau River is relatively small so that there would be no advantage of having a storage reservoir downstream of the one considered by Acres at Richmond, which would provide flood protection at Richmond as well as some in the Ottawa reaches. Hence a reservoir on the Jock River would not markedly reduce the flood peak magnitudes nor flood damage in Ottawa.

The drainage area of Kemptville Creek is 466 square kilometres or 18 percent of the uncontrolled basin downstream of Poonamalie. The stream gradients are mild, and the only significant storage areas are in the headwaters of its two tributaries - the South and North Kemptville Creeks. On the latter stream there is 3.1 MCM storage available at Cranberry Lake, and 2.8 MCM available at Oxford Mills on the South Branch [14 and 15]. These storages are much smaller than the proposed one at Richmond on the Jock River and their drainage areas are much less, hence their possible contributions to flood attenuations at Ottawa are small. Because of low stream gradients there are no suitable, larger reservoir locations on the downstream reaches of Kemptville Creek. Hence the possible influence of storage reservoirs on the Jock River and Kemptville Creek on flood level reduction in the study reaches is not sufficient to warrant further study.

A possible reduction in the flood hazard may be realized along the study reaches by storing more water in Big Rideau-Lower Rideau Lake upstream of Poonamalie during freshet, ie., do not allow any discharge from the lake. This is not considered to be feasible for two reasons: Parks Canada has attempted to control outflows since 1976 - normally less than three m^3/sec - during peak flood occurrences; and the Poonamalie control structures only control 33% of the drainage area of the Rideau River basin. Although there is a little more storage at each of the downstream locks and upstream of the control structures, their total storage is not large enough to appreciably attenuate the flood peak in the study reaches. Thus the Poonamalie control structure cannot be used more effectively than it is at present in attenuating the peak flood flow rate of the Rideau River through the study reaches.

5.3 Flood Releases Through The Rideau Canal

One possibililty of alleviating some flood damages in the Rideau River through Ottawa during large floods is to allow some of the flood flow through the portion of the Rideau Canal System from Mooneys Bay to the Ottawa River, thus reducing the flow magnitude that is conveyed through the Rideau River. Calculations indicated that approximately 28 m^3/sec could flow through the canal system without damaging the locks or eroding the channel banks. A consideration, which has not been conceptually resolved, is how to free the canal of its ice cover in a reasonable duration before the flood peaks without causing structural damages and without interfering with traditional recreational use of the ice covered canal. In any case the flow rate of 28 m^3/sec, which is the largest that the canal can feasibly carry, when subtracted from the maximum flow rate in the Rideau River would decrease the water surface elevation upstream of Cummings Bridge by 0.08 and 0.11 metres for the 1:100- and 1:50-year floods, respectively. Hence using the Rideau Canal System through the study reaches is not considered feasible because of its small effect on Rideau River flood levels and the expense of adapting the Rideau Canal for reliable hydraulic conveyance in early spring.

5.4 Upstream Flow Control During Ice Formation

Frazil ice, which causes some of the thickening of ice sheet as well as anchor ice and hanging ice dam formations, results from supercooling of water particles in rapid flow locations. Thus, if the flow variation, during the periods of the year when the air temperatures are below freezing and an ice cover is forming, is minimized there will be less frazil ice produced. Hence one possible means of lessening the ice jamming potential is to have the flow rates in the lower river system fluctuate as little as possible during the formation period each year. One result would be that ice formation and subsequent thickening of ice sheet immediately downstream of the Strathcona Rapids would be minimized.

With a reservoir built on the Jock River and the freedom to control Poonamalie releases, about 48% of the basin runoff could be controlled. These measures alone are not adequate to stop rapidly rising and falling water stages during winter rainstorm conditions especially when rain on frozen ground runoff occurs which is a common event in the Rideau River basin in December and January.

Although there would be some benefits from enacting upstream controls, its feasibility cannot be assessed until a comprehensive study on the study reaches winter regime and Rideau River winter hydrology as recommended by Dr. Michel -given in Appendix C- has been completed.

5.5 Ice Sheet Removal

Meetings were held with Mr. W. Freitag of the City of Ottawa to discuss the city's ice clearing operations. Also Dr. B. Michel, a river ice expert, inspected the river reaches and the ice clearing operations and later met with City of Ottawa and the Conservation Authority officials. He submitted a letter report on his findings and conclusions, which is given in Appendix C.

Basically he concluded

> "The present ice control operations are the best to prevent jamming and they are fine-tuned to the river hydraulic and glaciological regime. However, because of the high annual cost of the ice clearing operations I recommend that studies be undertaken to either improve them or to design permanent works that would reduce costs."

In order to assess the feasibility of these alternatives he recommended two studies be commissioned

A) " A comprehensive study on the winter regime, on the evolution of ice cover and on ice control operations as well as their effectiveness in preventing flooding. This report should also include the hydrology of the upstream basin and its influences on winter flash floods and the pre-breakup use of water to improve the ice clearing operations."

B) "A study on possible improvements to current ice control operations such as full mechanization of key cutting, blasting alternatives, ice dusting and floating ice breaking devices."

Most of the information required to undertake the first study is available from the City of Ottawa, the Rideau Valley Conservation Authority and Parks Canada files. There is extensive technical literature on most phases of the second study.

5.6 Drowning Strathcona Rapids

The average summer flow in the Rideau River at Ottawa is approximately 20 m^3/sec (non-storm flow); the average December storm flow is about 100 m^3/sec and the average annual flood, which is due to snowmelt runoff, is approximately 450 m^3/sec.

The Strathcona Rapids either do not freeze over or their location is the last one along the study reaches to freeze over each year. In the reach between Cummings and Hurdman Bridges the adjacent lands are subject to flooding during the high return period floods [Figure 8]. Some of this flooding results from the frazil ice and anchor ice formations in the reaches downstream of Strathcona Rapids caused by restricted hydraulic conveyance during the spring runoff period.

One means considered to reduce or eliminate frazil and anchor ice formations in this location is to reduce the velocities of flow at and in the vicinity of the Strathcona Rapids when the ice cover is being formed. This could be accomplished by building two broad-crested weirs across the river upstream of Cummings Bridge in the two channels adjacent to Cummings Island [See Figure 9]. One weir could be approximately 55 metres long and would extend from the east shore (Vanier) to the island and the other would be 95 metres long and would extend from the west shore (Ottawa) of the river bank to the island. The simulated weirs have 1.75 m crest lengths and rounded upstream edges.

Various weir cross-sectional shapes as well as invert elevations were considered in order that the combined hydraulic conveyance rating through the two weirs would have such characteristics that the upstream water surface elevations would be higher than they are under current conditions at flow rates around 100 m^3/sec and are not above, or at most only a little above normal, when the high annual floods occur in the spring.

Various combinations such as weir type, triangular and rectangular -in each channel around the island, weir invert elevations between 52.5 and 54 metres as well as the triangular notch angle between 140 and 160 degrees were simulated. Both weirs were considered located at the upstream end of Cummings Island (See Figure 9). The rating curve of the combination of two weirs, with various physical geometries, were compared with rating curves of the flow with the present stream geometry in order to determine rating curves of the two weirs in place that would have higher water surface elevations at low flows and lower than current conditions at high flows (in the neighbourhood of annual maximum flows). It was found that there were two combinations that provided the desired results. Both have a rectangular weir in the 55-metre wide east channel and a triangular weir in the wider west channel.

Both cases had the invert of the triangular weir set at 52.5 metres and the rectangular at 54 metres. In one case the notch angle was set at 140 degrees in the other 160 degrees. The initial calculations showed that both combinations would give an increase in water surface elevation at the island of approximately 0.6 metres above existing conditions when the flow is 100 m^3/sec and less than existing with flows above 400 m^3/sec. These hydraulic conditions do not consider the influence of the backwater due to the downstream hydraulic control.

In order to take into account the backwater effects the weir combinations simulated at the upstream end of Cummings Island were tested with the HEC2 model, and the water surface elevations determined in the river reach between station 2490 and 3909, which are the cross sections immediately downstream and upstream of Strathcona Rapids. Because of the backwater from Cummings Bridge, the increase in water elevations for the considered flows were not as high as when the hydraulics of the weirs was considered alone. For the 160 degree notch weir case it is about 0.08 metre; and for the 140 degree notch weir case it is about 0.17 metres.

The average velocity at the Strathcona Rapid section with the existing channel section when the flow is 100 cms is 94 cm/sec. This is decreased to 82 cm/sec for the 160 degree notch weir case and to 77 cm/sec. for the 140 degree notch weir situation. These are not significant decreases in water velocities and would not significantly decrease the formation of frazil ice.

The two broad-crested weirs were simulated again with the invert at at elevation 54.5 metres which gives water surface elevations higher than 57.0 metres with flow rates higher than 350 m^3/sec and an increase in water elevation of one-half metre. The maximum spatial average water velocity in the rapid reach is 64 cm/sec when the flow rate is 100 m^3/sec. Frazil ice will form when water is flowing at velocities greater than 25 cm/sec when the water particles in the turbulent flow are supercooled. Thus frazil ice still will form at the Strathcona Rapids if broad-crested weirs are placed at Cummings Island.

With the weirs in place the rapids will be drowned out. Although the velocities will be reduced only slightly the turbulence most likely will decrease by a greater extent. The weirs under these flow conditions would not be drowned out by the downstream constriction at Cummings Bridge, hence there will be rapid but smooth flow immediately downstream of the weir. The weirs could be effective if the turbulence immediately downstream of the weirs is less than it is over the rapids with the current conditions.

In order to assess the effectiveness of this measure physical hydraulic modelling or sophisticated hydrodynamic numerical modelling would have to be carried out on the river reach from downstream of Cummings Bridge to upstream of Strathcona Rapids. These are beyond the scope of the present study and should only be considered after the other alternatives are reviewed.

No doubt if weirs were placed with invert elevations higher than 55 metres the Strathcona Rapids would be drowned out when the flow is approximately 100 m3/sec (during the formation of an ice cover); however, the weirs would cause such high water surface elevations during spring flooding periods that there would be overbank flooding in the reaches between Cummings and Hurdman Bridges. Flood elevations would be higher than those that occur under existing natural conditions. This would outweigh the advantages of the weirs eliminating or hampering formation of ice jams. Thus this alternative will not be considered further.

Mathematically modelling weirs at other locations along the river was not considered because the only other area where there are rapids is upstream of Billings Bridge. There the water stream gradients are so high - 10 times higher than in the lower reaches - that the weirs would not be effective. Moreover, these areas are not the ones where ice jamming and serious flooding occur.

5.7 Channel Improvements

5.7.1 General

The location and elevation of bedrock in the Rideau River are not precisely known with the exceptions at Strathcona Rapids and at the geological fault that crosses the Rideau River at Carleton University. A 1:50,000-scale map labelled Bedrock Topography of Ottawa-Hull gives the bedrock elevations with 25-foot contours [16]. The map shows bedrock elevations varying between 45.7 and 53.3 metres above Geodetic datum in the study reaches; however, the map is not precise enough to obtain bedrock elevations where channel improvements are considered. All bedrock excavations considered to lower the flood stage or move the hydraulic controls are at high points on the riverbed profile [see Figure 8]. It was assumed that the riverbed at these locations consists of rock only, since if there was overburden the action of annual flood velocities and ice movements would have eroded it deeper during the 10,000 years that the Rideau River has been flowing in its present course. At Hogs Back Road the rock is limestone of the Ottawa Formation, downstream there is black shale of the Billings Formation and in the lower reaches there are the dark grey limestones with shales of the Eastview Formation. The cost of excavating this rock is based on building and removing berms, blasting the rock and removing it.

A number of channel improvements have been considered which would lower the floodwater elevation in some reaches and help alleviate ice jammings and subsequent backwaters. Unit material costs which were obtained from various sources, including the Rideau Valley Conservation Authority, are given in Table VII.

5.7.2 Strathcona Rapids

Excavating the bedrock at Strathcona Rapids from the present maximum rock invert elevation of 54.6 metres to a lower elevation at a location where the streambed is approximately 125 metres wide was considered (See Figure 8). One possibility is to excavate to elevation 53.95 metres. This involves blasting and removing 7200 cubic metres of rock at a cost of $576,000. This would lower the water surface elevations immediately upstream by nine centimetres and by five centimetres at Hurdman Bridge during a 1:100-year flood.

The other possibility at Strathcona Rapids is to excavate to elevation 53.80 metres. This would involve blasting and removing 10,640 cubic metres of rock at a cost of $ 851,000.00. This would lower the water surface elevation immediately upstream by eleven centimetres and by five centimetres at Hurdman Bridge during the Regional Flood.

5.7.3 Cummings Bridge and Island

The 1:100-year open water level through the Cummings Bridge is below the crowns of the arches, nevertheless a small backwater occurs. The major problem at this location is when ice is flowing with large spring floods. Ice jams have occurred at the bridge during large annual maximum flows causing serious flooding in the upstream reaches such as that which occurred in February 1981. The river turns to the west just downstream of the bridge hence the water surface will be super-elevated on the east side during high flows making it more susceptible to ice jamming, since there is a higher probability of the eastern bridge span openings being blocked with ice.

The total length of the arched bridge is 170 metres with five, 26 metre spans and a series of smaller arches. An effective way of eliminating the susceptibility to ice jamming is to replace the bridge with a one- or two-span bridge. The cost would be approximately $ 4,000,000. including removing the existing bridge. Since the bridge is old and discussions have taken place at the Regional Municipality level to modify it, this possibility should be considered further.

Improved hydraulic conveyance of the bridge would be somewhat negated if the Cummings Island located immediately upstream of the bridge retards the large magnitude flows which would cause higher backwaters and reduces the hydraulic effectiveness of building a new one- or two-span bridge. The combined cross sectional area of flow in both channels around Cummings Island is larger than that of the bridge at flood stages, hence the island is not presently constricting open water flow.

5.7.4 Porter Island

The next possibility is excavating the channels on both sides of Porter Island to elevation 53.2 metres and completing rock excavation in the channel to the St. Patrick Street Bridge to the same elevation. This would involve excavating 13,100 cubic metres of rock at a cost of $1,048,000. The 1:100-year water surface elevations would be lowered by five centimetres at the northern Porter Island Bridge, ten centimetres at St. Patrick Street Bridge and eight centimetres at the downstream edge of Cummings Bridge.

5.7.5 St. Patrick Street Bridge

This bridge was built where the streambed is high - elevation 53.65 metres. The excavation width considered is 155 metres. Two possibilities were investigated: one is to excavate the bedrock to 53.35 metres; the other is to excavate it to 53.2 metres. The first conditions would involve excavating 720 cubic metres of rock at a cost of $58,000. This would lower the water surface elevation by one centimetre for the 1:100-year flood. The second possibility would involve excavating 2400 cubic metres at a cost of $192,000. This would lower the 1:100-year flood water surface elevation by two centimetres immediately upstream.

The stretch of the river near St. Patrick Street Bridge is under the backwater influence of a downstream hydraulic control. Hence it is not feasible to consider excavating to any depth at this location without also excavating at other locations.

Another possibility is to excavate the channel bed from St. Patrick Street bridge to the southern end of Porter Island to elevation 53.2 metres. This lowers the 1:100-year water surface by another one centimetre. The total rock excavation is 6000 cubic metres and it would cost approximately $480,000.

5.8 Diking in Flood Prone Areas

5.8.1 General

The measures considered were dikes placed along the shore adjacent to areas where the ground elevations are below the Regional Flood elevation. The proposed dikes top widths would be two metres and side slopes four horizontal to one vertical. The height will be up to two metres with 0.6 metre freeboard above the 1:100-year flood line to account for enbankment settlement and wave action. The cross-sections will be trapezoidal and will be constructed of fill material. The in-situ overburden will be excavated to the bottom of organic material along the route over the width of the dike in order to key the dike into its foundation. Topsoil, 80 mm thick, is to be placed on the dike side slopes and then seeded in order to develop stable slopes that are erosion-resistant.

Since most of the dikes are either in or close to parklands the tops could be paved with asphalt (40 mm thick) with a 100 mm layer of gravel base underneath to form a bicycle path. The proposed dikes have been located so that they will follow as much as possible the borders of parks and property lines. The dikes are situated inshore so that the park land will still be part of the 1:100-year and smaller recurrence interval floodplains. The total number of residential properties that would be protected by the dikes is approximately 200.

5.8.2 Carleton University

This dike would be built to elevation 60.0 metres between the west embankment of Bronson Avenue to the Carleton University

Sports Center roadway, a distance of 25 metres. The estimated total cost of construction, engineering and contingencies is $12,000. The extent of the diking is shown in Figure 10.

5.8.3 Warrington Drive

The dike would extend on the north side of the river from upstream of Billings Bridge to the retaining wall at the east of Osborne Avenue at the townhouses. The concrete dike length would be 480 metres and would be built to an elevation varying between 59.6 and 59.9 metres. Provisions for storm water pumping are required. The estimated total construction, engineering and contingencies cost is $260,000. The extent of the diking is shown in Figure 11.

5.8.4 Windsor Park - Stage II

This dike would have a top elevation varying from 59.5 to 59.6 metres and extend along the west shore adjacent to the junction of Rideau River Drive and Brighton Street to the Windsor Park-Stage I Dike - a distance of 435 metres. The estimated total construction cost of the dike plus engineering and contingencies is approximately $210,000. The extent of the diking is shown in Figure 12.

5.8.5 Brantwood Park

The proposed dike which would be placed along the shoreside of Onslow Avenue would be approximately 300 metres long extending from Burnham Road to Brantwood Drive. The dike top elevation will vary from 59.20 to 59.35 metres. The estimated total construction cost including pumping, engineering and contingencies is $100,000. The extent of the diking is shown in Figure 13.

5.8.6 New Edinburgh

This dike would be located on the east side of the Rideau River from Sussex Drive to Union Street and then south until it meets the 56.2 metre contour adjacent to the old railway bridge embankment in New Edinburgh Park. The total length of dike would be 620 metres, 280 metres north of Union Street and 340 metres south of that street. The estimated total construction cost including storm water pumping, engineering and contingencies is $65,000. The extent of the diking is shown in Figure 14.

5.8.7 Local Drainage

The spring runoff from the local drainage area upslope of each dike must be transported to the river by means other than the existing storm sewers. It is proposed to place a one-way valve in each storm sewer flowing to the Rideau River that drains areas protected by the proposed dikes and pump the local runoff over the dike from a sump located at the lowest point adjacent and upslope of the dike. This would occur every year.

The most feasible means of draining the areas contributing to runoff behind the dikes is to build a permanent sump one metre deep at the lowest point in a protected area next to the dike and purchase four portable sump pumps (one for each area)that would be stored in a warehouse and transported to the sumps at the onset of high flows. These pumps would be used for a maximum duration of two weeks in the high spring floods and would be stored in a warehouse for the remainder to the year. At the onset of a large flood each pump would be moved to the sump, set up and operated as reguired until the flood subsides. An allowance has been made to have an operator on duty at each site for up to a two-week duration in the economic analysis.

5.9 Economic Analysis of Diking

5.9.1 Objective

Along the Rideau River downstream of Hogs Back Road there are residential areas that are subjected to flooding during the spring runoff, if not every year then during higher than average spring runoff years. These include the Warrington Drive area, Windsor Park - Stage II and Brantwood Park, all within the City of Ottawa on the left shore. The Kingsview Park area in the City of Vanier and the New Edinburg area in the City of Ottawa, both on the right shore, could flood with the occurrence of high return period floods. Most of the buildings in the latter two areas would be flooded if existing berms (the structural and hydraulic integrities of which are presently unknown) failed. The expected annual flood damage costs in these areas were calculated and compared with the cost of preventing flooding by the construction as well as maintenance of dikes and pumping the local runoff during high river stages.

Outlined in the following paragraphs are the methodology and results of an economical analysis to determine whether or not diking is an economically viable means of protecting these residential properties from flood damage whose probabilities of occurrence are equal to or greater than 0.01 in any year. The planning horizon used is twenty five years hence, if diking is built in the recommended areas the risk that the dikes will be overtopped and residential areas flooded once in the next twenty-five years is 22 percent. The proposed dikes will have 0.6 metre of freeboard above the Regional Floodlines; these dikes will contain floods larger than the 1:1000 year return period flood with 0.3 metres of freeboard. The risk of such a flood occurring at least once in a twenty-five-year period is 2.7%.

5.9.2 Methodology

The number of buildings within the prescribed flood lines were counted. The Listings of the Real Estate Board of Ottawa-Carleton [17] for the second half of 1983 were consulted to determine the average selling price of houses in each flood-prone area. In most cases because of the small number of building sales in the flood-prone areas, houses outside a flood-prone area but yet in the immediate vicinity were considered. In addition, some 1984 house selling transactions were investigated to determine the selling prices. From this an average house value was calculated for each area.

The flood damage to a house was estimated by judging from the ground elevation near the houses and the flood line elevation whether the water level would be below or above the elevation of the first floor. Average values of flood damages versus flood water level given in Flood Damage Study [18] were used to approximate the flood damage as a percentage of property value. When the water level is above the first floor elevation of a building the structural damage was taken as 4% of the property value. When it does not come up to the level of the first floor, the structural damage was taken as 3% of property value. The house content damages were taken as 7.5% if the flood level comes above the first floor and 6% if the water level does not come up to the first floor. The number of buildings within each prescribed floodline were multiplied by the average property value and by the appropriate combined percentage of structural and content damages to obtain the total flood damage for various probabilities of exceedence. These damage amounts were plotted against probability of exceedence and the areas under the curves were integrated to obtain the expected average annual flood damage.

5.9.3 Economic Analysis

The average value of properties for the various flood prone areas are given below:

Area	No. of 1983 Sales Considered	Average Price
Warrington Drive	2	$149,000.
Windsor Park	8	$130,000.
Brantwood Park	1	$130,000.
Kingsview		
- East of Vanier Parkway	15	$ 88,000.
- West of Vanier Parkway	2	$130,000.
New Edinburgh	2	$178,000.

The sampling in Brantwood Park is too small to obtain an average; hence the Windsor Park average was used in this area. The rationale being that the two areas are close and the houses are of similar style and age. All property values have been increased by three percent to express them in 1984 dollars.

The expected annual flood damage to properties as well as the expected total flood damage over twenty five years located within the 1:100-year flood lines are given in Table VIII.

A design horizon of 25 years is considered, mainly because of the expected life of pump machinery and continual changing stream morphology.

The construction costs of the dykes are:

Carleton University	$ 12,000.
Warrington Drive Area	$260,000.
Windsor Park, Phase II	$160,000.
Brantwood Park	$ 80,000.
New Edinburgh	$ 50,000.

The cost of pumping and auxilary equiment has been estimated to range between $5000 and $20,000 -depending on capacity and in turn location.

The future costs and benefits were discounted at 7% per year to obtain their 1984 value as recommended in the publication: Benefit-Cost Guidelines for Conservation Authority Flood and Erosion Control Projects[19]. In addition, discount rates of 5 % and 10% were also applied in order to reflect the sensitivity of varying opportunity costs of money during the 25-year planning horizon. The diking costs and benefit-cost ratios are given in Table lX.

At Kingsview Park in Vanier and in New Edinburgh there are two ground depressions where houses and other buildings have been built. The basements and the first floor levels in most of these buildings are below the Regional Flood elevation. At present these areas are protected from flooding by the roadway along Stanley Street in the New Edinburgh area and by the River Road and Vanier Parkway in Kingsview Park area. The stability and hydraulic imperviousness of these roadways are unknown, and the Rideau Valley Conservation Authority requested that the damages resulting from structural or hydraulic failures on either roadway be assessed. It has been assumed that the roadways will act as an effective barrier to flooding for flood magnitudes of 1:5-year return period or less. The damages to buildings in these areas are also given in Table VIII for various discount rates. The roadways have so far provided flood protection in the reduced flood risk areas. Before consideration is given to any additional protection it is recommended that geotechnical field investigations and analyses be undertaken to determine their stability and effectiveness in preventing excessive seepage and piping.

The results of this economic analysis are approximations only because of the method used to determine property values and the probable flood damges to the buildings. To obtain more precise results would require extensive field surveys and property assessments, which are beyond the scope of a preliminary engineering assessment. The benefit-cost ratios are so high (all over two); however, that it may be concluded that dike construction should be completed in the Warrington Drive, Windsor Park - Stage II and Brantwood Park flood-prone areas.

6. RECOMMENDATIONS AND CONCLUSIONS

Flood flows for the specified return periods determined in the flood frequency analyses are the best estimates for design purposes along the study reaches. The three-parameter lognormal distribution gives the best statistical fit to the flood data and these results have been used in determining return period flood elevations.

The flood elevations and extent of flooding are similar to those reported in the 1972 study. Exceptions are those lands previously noted as flooded in the Kingsview Park area. Higher than previously shown road elevations between Tudor Place and Wayling Avenue contain the Region Flood closer to the river. Along Rideau River Drive (near Billings Bridge) there are higher than previously shown road and ground elevations. The current analysis and new topographic mapping show that the flooding is contained along and close to the stream bank in these areas.

The completion of the considered dike schemes will substantially reduce flooding at the Warrington Drive, Windsor Park - Stage II and Brantwood Park areas. We recommend that these works be undertaken. Also the hydraulic and structural integrities of Stanley Street roadway in New Edinburgh and of River Road and the Vanier Parkway in Kingsview Park be investigated and if found to be inadequate then remedial measures be enacted.

Conveying flow through the Rideau Canal at Hogs Back Road and channel excavations are not economical alternatives. Providing additional control of upstream flows during the spring runoff periods may be a viable means of reducing the severity of flooding; however, in order for this to be determined a hydrological analysis as recommended in Chapter V must be completed.

Replacing Cummings Bridge is not economically feasible for ice jam elimination alone; however, if the bridge is to be replaced because of other considerations, it is recommended that the design of the spans and obverts take into account ice passage considerations.

Before the dikes are designed the feasibility of building them higher to protect flood-prone areas from larger floods than 1:100-year should be investigated.

The placement of broad-crested weirs at Cummings Island to reduce frazil ice formation at Strathcona Rapids could be feasible, but physical hydraulic model testing or sophisticated hydrodynamic numerical modelling would have to be conducted in order to establish this.

A study should be commissioned to determine means and economic feasibility of improving the ice clearing operations. Also, another study should be undertaken on the evaluation of the influence of ice cover thickness and variability on ice control operations as well as the latter costs and effectiveness in preventing flooding. This study should also include the hydrology of the upstream basin for controlling flow variations during ice cover formation and to provide pre-flushing water at the optimum time for ice clearing operations.

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TABLES

TABLE I

FLOOD FLOWS FOR STATION NO. 02LA004, RIDEAU RIVER AT OTTAWA

Year	Maximum Instantaneous m^3/sec	(cfs)	Maximum mean Daily m^3/sec	(cfs)	Maximum Instantaneous (Maximum mean Daily x 1.053) m^3/sec	(cfs)
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)	417	(14730)
1973	464	(16390)	447	(15790)	464*	(16390)
1972	578	(20410)	535	(18890)	578*	(20410)
1971	513	(18210)	496	(17520)	513*	(18120)
1970			442	(15610)	465*	(16440)
1969			328	(11580)	345	(12200)
1968			377	(13310	397	(14020)
1967			311	(10980)	327	(11570)
1966			215	(7590)	226	(8000)
1965			146	(5160)	154	(5430)
1964			109	(3850)	115	(4050)
1963			442	(15610)	465	(16440)
1962			323	(11410)	340	(12010)
1961			193	(6820)	203	(7180)
1960			532	(18790)	560	(19780)
1959			413	(14590)	435	(15360)
1958			306	(10810)	322	(11380)
1957			133	(4700)	140	(4950)
1956			351	(12400)	370	(13050)
1955			493	(17410)	519	(18330)
1954			405	(14300)	426	(15060)
1953			331	(11690)	349	(12310)
1952			379	(13380)	399	(14090)
1951			419	(14800)	441	(15580)
1950			447	(15790)	471	(16620)
1949			379	(13380)	399	(14090)
1947			538	(19000)	567	(20010)

* recorded values

TABLE II

ANNUAL MAXIMUM MEAN DAILY FLOWS AT POONAMALIE AND OTTAWA AND RATIO OF TWO IN PERCENT

	Date	Poonamalie		Ottawa		
		m^3/sec	(cfs)	m^3/sec	(cfs)	Ratio%
1982	April 1, 2	11.7	(410)	397	(14020)	2.95
1981	Feb. 24	35.1	(1240)	435	(15360)	8.09
1980	Mar. 22	2.3	(80)	385	(15600)	0.52
1979	Mar. 25	3.4	(120)	403	(14230)	0.84
1978	Apr. 14	21.2	(750)	487	(17200)	4.37
1977	Mar. 15	0.0	(0)	467	(16490)	0.00
1976	Mar. 28	55.8	(1970)	583	(20590)	9.57
1975	Apr. 20	31.4	(1110)	394	(13910)	7.98
1974	Apr. 6	55.2	(1950)	396	(13990)	13.94
1973	Mar. 18	30.9	(1090)	447	(15790)	6.90
1972	Apr. 21	51.8	(1830)	535	(18890)	9.69

TABLE III

MAXIMUM INSTANTANEOUS FLOW - STATION NO. 02LA004

RIDEAU RIVER AT OTTAWA

Year	Decrease Flows Prior to 1977 by 7 percent m^3/sec (cfs)	Increase Flows After 1977 by 7 percent m^3/sec (cfs)
1982	435* (15360)	465 (16440)
1981	446* (15750)	477 (16850)
1980	421* (14870)	450 (15910)
1979	423* (14940)	453 (15980)
1978	527* (18610)	564 (19910)
1977	473* (16700)	506 (17870)
1976	555 (19600)	597 (21080)
1975	384 (13560)	413 (14590)
1974	388 (13700)	417 (14730)
1973	464 (15240)	464 (16390)
1972	538 (18980)	578 (20410)
1971	477 (16850)	513 (18120)
1970	432 (15270)	465 (16440)
1969	321 (11330)	345 (12200)
1968	369 (13040)	397 (14020)
1967	304 (10740)	327 (11570)
1966	210 (7420)	226 (8000)
1965	143 (5060)	154 (5430)
1964	107 (3780)	115 (4050)
1963	432 (15270)	465 (16440)
1962	316 (11170)	340 (12010)
1961	189 (6670)	203 (7180)
1960	521 (18390)	560 (19780)
1959	405 (14290)	435 (15360)
1958	299 (10580)	322 (11380)
1957	130 (4600)	140 (4950)
1956	344 (12150)	370 (13050)
1955	483 (17050)	519 (18330)
1954	396 (13990)	426 (15060)
1953	325 (11460)	349 (12310)
1952	371 (13100)	399 (14090)
1951	410 (14480)	441 (15580)
1950	438 (15470)	471 (16620)
1949	371 (13100)	399 (14090)
1947	527 (18610)	567 (20010)

* Unchanged from Table II

TABLE IV

SOME FLOW STATISTICS OF MISSISSIPPI, SOUTH NATION AND RIDEAU RIVERS

	Mississippi	South Nation	Rideau
Year of lowest flood	1957	1957	1964
Year of second lowest flood	1961	1965	1957
Year of third lowest flood	1964	1964	1965
Year of fourth lowest flood	1966	1961	1961
Year of fifth lowest flood	1962	1966	1966

TABLE V

RETURN PERIOD FLOOD ESTIMATES

Maximum Instantaneous Flows
After 1976 Increased by 7%

Return Period Flows

m^3/s

Distribution	Return Period					Skew Coef.
	5	10	25	50	100	
LN	513	552	606	632	663	-.1076
3PLN	513	552	598	626	654	-.0389
LP-3-M	513	552	600	626	654	-.2652
LP-3-L	513	549	581	615	637	-.2652

Note:
- LN - means lognormal distribution
- 3PLN - means three parameter lognormal distribution
- LP-3-M - means log Pearson type III distribution using moments
- LP-3-L - means log Pearson type III distribution using maximum likelihood

REVISED - TABLE VI

SUMMARY OF RETURN PERIOD FLOOD ELEVATION

Location	Chainage (m) from Rideau Falls	100	50	25	10	5
		Return Period (yr) Metres above Geodetic datum				
East Channel						
Rideau Falls	41	53.57	53.51	53.46	53.37	53.31
East Dam	49	54.22	54.14	54.07	53.93	53.81
	86	54.52	54.44	54.35	54.22	54.11
Sussex Drive	114	54.85	54.78	54.70	54.58	54.47
Minto Bridges	387	55.36	55.29	55.21	55.08	54.97
	396	55.37	55.29	55.21	55.09	54.97
	506	55.58	55.50	55.41	55.27	55.15
	599	55.58	55.50	55.41	55.27	55.15
West Channel						
Rideau Falls	30	53.47	53.43	53.38	53.31	53.24
West Dam	37	54.77	54.70	54.62	54.50	54.40
Sussex Drive	125	55.41	55.33	55.23	55.08	54.95
	145	55.41	55.33	55.24	55.09	54.95
Minto Bridges	385	55.51	55.43	55.36	55.23	55.11
	396	55.51	55.43	55.36	55.23	55.11
	506	55.60	55.52	55.44	55.30	55.19
	599	55.60	55.52	55.44	55.30	55.19
Main Channel						
	846	55.71	55.63	55.55	55.40	55.29
	1142	55.81	55.73	55.66	55.51	55.40
Porter Island	1351	55.97	55.90	55.83	55.70	55.60
Bridge	1399	56.04	55.97	55.90	55.77	55.66
Porter Island	1511	56.11	56.04	55.97	55.84	55.73
Foot Bridge	1515	56.12	56.04	55.97	55.84	55.73
St. Patrick St.	1690	56.25	56.17	56.10	55.97	55.86
Bridge	1720	56.26	56.18	56.11	55.97	55.86
Cummings Bridge	2474	56.49	56.41	56.33	56.19	56.07
	2490	56.52	56.44	56.36	56.21	56.09
Strathcona Rapids	3352	57.85	57.78	57.70	57.56	57.44
Old Hurdman	4399	57.62	57.54	57.47	57.33	57.22
Bridge	4406	57.63	57.56	57.48	57.35	57.24
Queensway	4434	57.70	57.63	57.55	57.41	57.30
Bridge	4464	57.71	57.64	57.56	57.42	57.31
CPR Railway	4521	57.81	57.74	57.66	57.52	57.41
Bridge	4524	57.85	57.78	57.70	57.56	57.44
Smyth Road	6899	58.63	58.55	58.47	58.32	58.21
	6920	58.65	58.57	58.49	58.34	58.23
Billings	8171	58.98	58.90	58.83	58.67	58.57
Bridge	8189	59.00	58.92	58.85	58.68	58.58
Dunbar	9493	59.33	59.25	59.17	59.00	58.89
Bridge	9513	59.34	59.25	59.17	59.00	58.89
CPR Bridge	10006	59.89	59.81	59.75	59.61	59.46
	10011	59.92	59.85	59.78	59.64	59.49
Carleton Gauge	10168	60.49	60.41	60.35	60.20	60.06
Heron Road	10834	61.65	61.57	61.52	61.38	61.29
	10878	61.70	61.63	61.57	61.44	61.38
	11507	72.03	71.94	71.85	71.69	71.56
Hog's Back Dam	11550	73.55	73.43	73.32	73.11	72.95

TABLE VII

DIKING AND EXCAVATION UNIT MATERIAL COSTS

Channel Rock Excavation	\$80.00/m^3
Earth Excavation	\$1.58/m^3
Asphalt (HL3) per lift	\$7.88/m^3
Gravel (granular A)	\$9.08/tonne
Topsoil	\$10.50/m^3
Grass seeding	\$0.50/m^2
Clay	\$12.00/m^3
Reinforced Concrete - Installed	\$400.00/m^3

TABLE VIII

DIKING BENEFIT - 1984 DOLLARS

FLOOD PRONE AREA	NUMBER OF BLDGS. SUBJECT TO 1:100 YR FLOOD	EXPECTED AVERAGE ANNUAL FLOOD DAMAGE	TOTAL FLOOD DAMAGE OVER TWENTY-FIVE YEARS DISCOUNT RATE		
			5%	7%	10%
Warrington Drive	34	$185,000.	$2,607,000.	$1,960,000.	$1,817,000.
Windsor Park Stage II	26	65,000.	916,000.	689,000.	638,000.
Brantwood Park	63	29,000.	408,000.	307,000.	285,000.
Kingsview:					
East of Vanier Parkway	46	157,000.	2,213,000.	1,663,000.	1,542,000.
West of Vanier Parkway	69	196,000.	2,762,000.	2,076,000.	1,925,000.
New Edinburgh (existing conditions)	41	28,000.	394,000.	326,000.	254,000.
New Edinburgh (if roadway fails)	41	130,000.	1,832,000.	1,515,000.	1,180,000.
TOTAL	279				
TOTAL (excluding areas behind existing dikes)		307,000.	4,325,000.	3,282,000.	2,994,000.
TOTAL (considering areas behind existing dikes)		762,000.	10,738,000.	8,210,000.	7,387,000.

TABLE IX

DIKING COST AND BENEFIT-COST RATIO

Location	Dike Construction Cost (in 1984 dollars)	Pump and Sump etc. Cost (in 1984 dollars)	Average Annual Pumping Cost ($)	Expected Annual Flood Damage ($)	Benefit-Cost Ratio, Discount Rate 5%	Benefit-Cost Ratio, Discount Rate 7%	Benefit-Cost Ratio, Discount Rate 10%
Carleton University	10,000.	5,000.	4,000.				
Warrington Drive	260,000.	10,000.	4,000.	185,000.	7.3	5.6	5.4
Windsor Park Stage II	160,000.	20,000.	4,000.	65,000.	3.4	2.5	2.4
Brantwood Park	80,000.	5,000.	4,000.	29,000.	2.5	2.0	2.0
Kingsview Park							
East of Vanier Pkwy.				157,000.			
West of Vanier Pkwy.				196,000.			
New Edinburgh	50,000.	5,000.	4,000.	29,000.	4.7	4.1	3.3
New Edinburgh (if roadway fails)				129,000.			

FIGURES

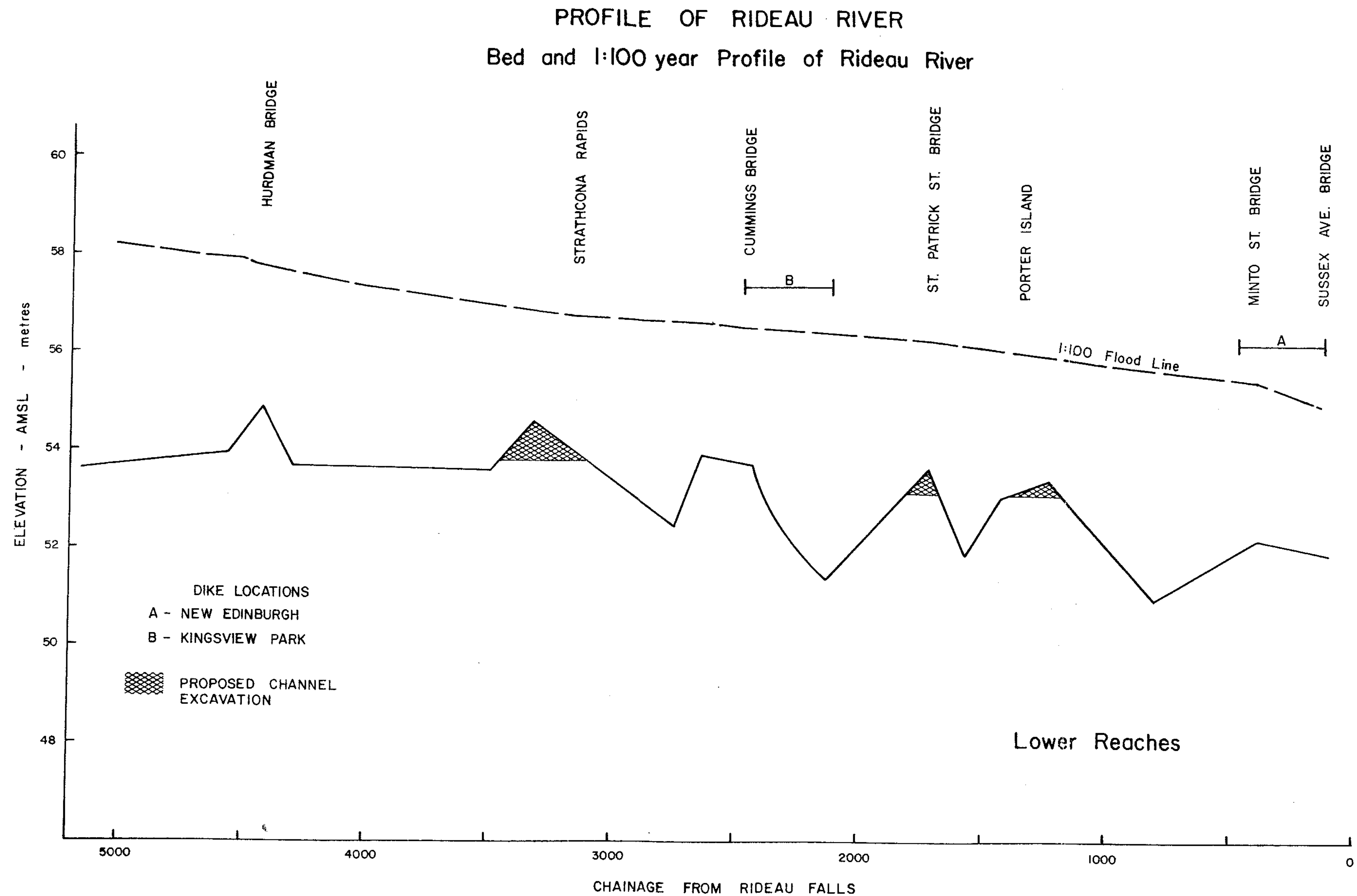

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Figure 8a

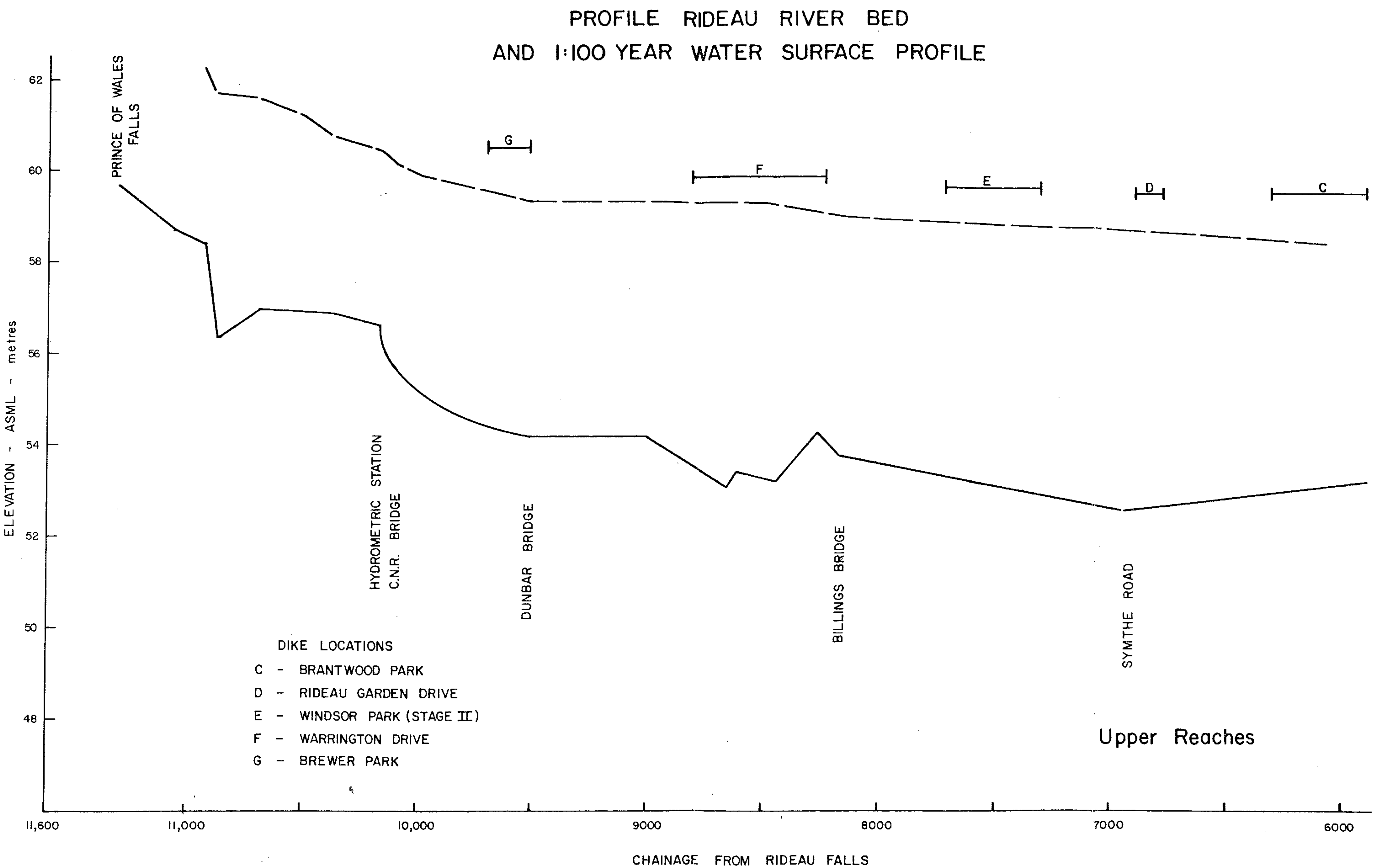

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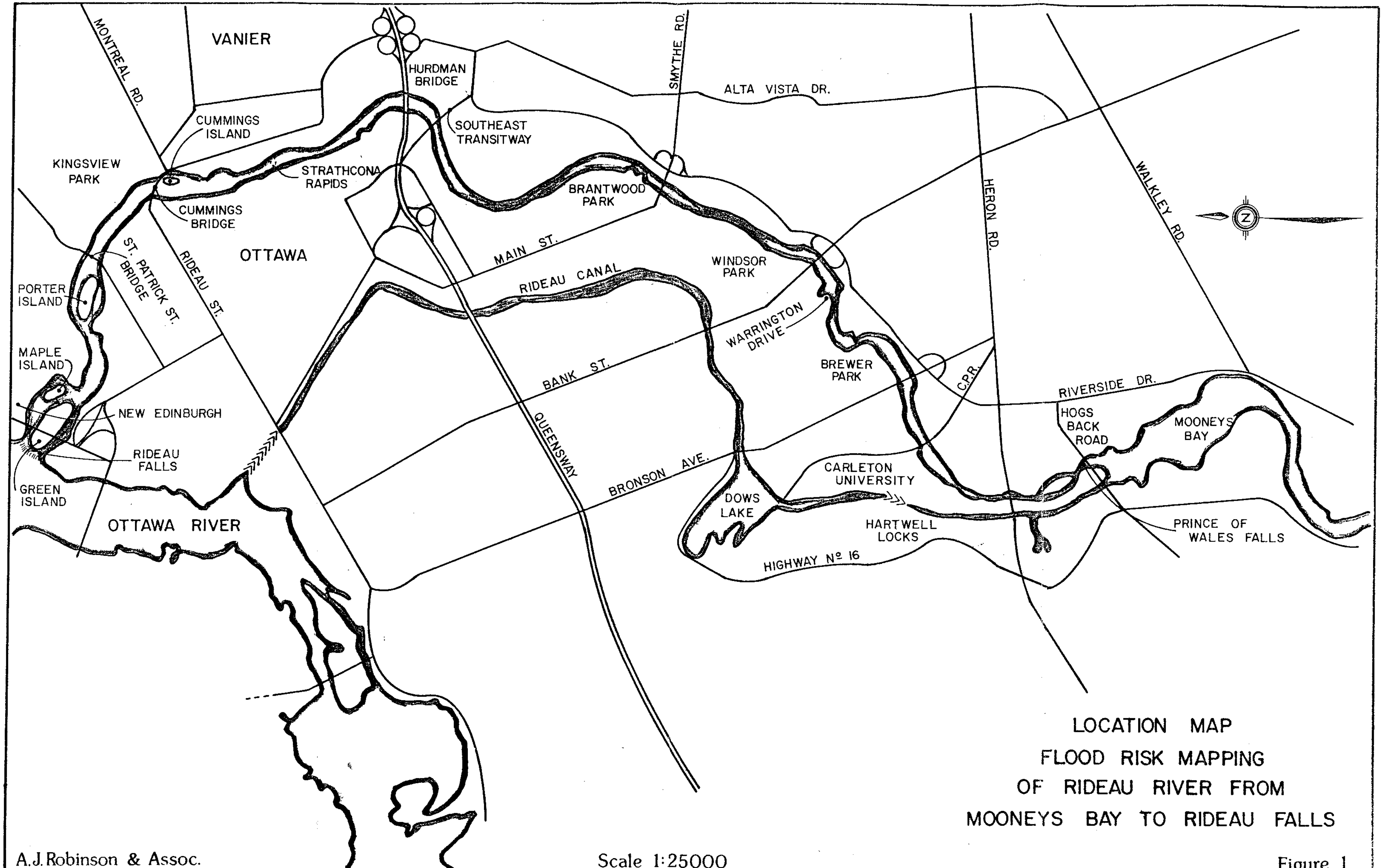
Figure 8b



LOCATION MAP
FLOOD RISK MAPPING
OF RIDEAU RIVER FROM
MOONEYS BAY TO RIDEAU FALLS

A.J. Robinson & Assoc.

Scale 1:25000

Figure 1

APPENDICES

APPENDIX A

OUTLIER TESTING RIDEAU RIVER

APPENDIX A

The initial data set consisting of 35 years of continuous flow data (Maximum Instantaneous Flows) was analysed to determine the occurence of outliers. The methodology used was taken from: Bulletin 17B of the Hydrology Committee, U.S. Water Resources Council, Revised September 1981, "Guidelines for Determining Flood Flow Frequency". These computations are as follows:

10% Significance Level

Data From Table II - Historic Data

1st attempt using natural logarithms			metric
mean of logarithms	X	9.50397	5.93951
standard deviation of logs	S	.39587	.39578
skew coefficient of logs	G	-1.70682	-1.70252
years of data		35	

Low outlier estimate cutoff

X = X - KN . S   where KN given in Appendix 4

KN for sample size of 35 = 2.628

XL = 9.50397 - 1.0403

= 8.4636

= 4740 cfs

therefore eliminate 4050 cfs - 1964

2nd attempt

mean of natural logarithm	9.53970
std. dev. of natural logarithm	.34183
skew coef of natural logarithm	-1.68568
years of data	34

test for low outliers XL= X - KNS = 9.53870 - 2.616(.34183)
= 8.64547 (5684 cfs)

therefore eliminate data 1957-(4950 cfs), 1965-(5430 cfs)

<u>3rd attempt</u>

mean	ln	9.60135
std. dev.	ln	.23993
skew	ln	-1.23855
Years		32

test for low outliers KN = 2.591

XL = X - KNS
= 8.97970
= 7,940 (cfs)

therefore eliminate 7180 cfs - 1961

<u>4th attempt</u>

mean	ln	9.62408
std. dev.	ln	.20373
skew	ln	- .86700
years		31

Kn = 2.577 XL = X - KNS
= 9.09908
= 8,947 cfs

therefore eliminate 1966 (8000 cfs) data

5th attempt

mean	ln	9.64531
std. dev.	ln	.16877
skew of	ln	- .00213

Kn = 2.563   XL = 9.21276
=10,024 cfs

No more to eliminate

Data from Table III - Flows after 1976 Increased by 7%

Attempt No. 1

mean	ln	9.51556
std. dev.	ln	.40196
skew	ln	-1.69276
years		35

XL = X - KNS   KN = 2.628
=
= 8.45920 (4718 cfs)
therefore eliminate 1964 - (4050 cfs) data

Attempt No. 2

mean	ln	9.55112
std. dev.	ln	.34766
skew	ln	-1.67332
years		34

XL = X - KNS   KN = 2.616
= 8.64164 (5663 cfs)
therefore eliminate 1957 (4950 cfs) and 1965 (5430 cfs) data

Attempt No. 3

mean	ln	9.61348
std. dev.	ln	.24555
skew	ln	-1.26005
years		32

XL = X - KNS KN = 2.591
= 8.97727 (7921 cfs)
therefore eliminate 1961 (7180 cfs) data

Attempt No. 4

mean	ln	9.63717
std. dev.	ln	.20915
skew	ln	- .93363
years		31

XL = X - KNS KN = 2.577
= 9.09819 (8939 cfs)
therefore eliminate 1966 (8000 cfs) data

Attempt No. 5

mean	ln	9.65883
std. dev.	ln	.17378
skew	ln	- .15723
years		30

L = X - KNS KN = 2.563
= 9.21344 (10,031 cfs)
No more points to eliminate.

APPENDIX B

OUTLIER TESTING

MISSISSIPPI RIVER AND SOUTH NATION RIVER

APPENDIX B

The last 35 years of continuous flow data for the South Nation River near Plantagenet Springs (Station No. 02LB005) and for the Mississippi River at Appleton (Station No. 02KF006) have been analysed to determine the occurence of outliers. The methodology was taken from: Bulletin No. 17B "Guidelines for Determining Flood Flow Frequency", U.S. Water Resources Council, Revised September 1981.

The streams were tested at the 5 percent significance level.

South Nation River

mean of logarithms	10.23470
standard deviation of logs	.24655
skew coefficient of logs	- .67747
years of data	31

XL = 9.55446   KN = 2.759

= 14,107 cfs (399 m^3/sec)

therefore only 4 outliers

mean of logarithms	10.25400
standard deviation of logs	.22569
skew coefficient of logs	- .56724
years of data	30

XL = 9.63448   KN = 2.745

= 15,283 cfs (433 m^3/sec)

therefore only 4 outliers

Mississippi River

mean of logarithms	8.61729
standard deviation of logs	.24157
skew coefficient of logs	- .35981
years of data	30

XL = 7.95418   KN = 2.745

= 2847 cfs (80.6 m^3/sec)

therefore 5 outliers

APPENDIX C

LETTER FROM DR. B. MICHEL RE ICE CLEARING OPERATIONS

F:8?:5.

B. C. MICHEL, DR. ING.
739, RUE DES VIGNES
STE-FOY, QUÉBEC G1V 2Y1
TÉL.: 418-653-3404

March 2, 1983

Dr. Peter Jolly
A.J. Robinson & Ass. Inc.
Cons. Eng.
P.O. Box 13130
Kanata, Ontario
K2K 1X3

Re: Anchor ice and ice jam in the Rideau river

Dear Peter:

Following my visit to Ottawa on March 1st, which included a tour of the river and of the ice control operations under the supervision of Mr. Wayne Fretag; completed by discussions with your engineers, those of the Rideau Valley Conservation Authority and those of the City of Ottawa, I have the following remarks to make concerning the study of ice jamming in this reach of the river.

My first observation is that the ice control operations on the Rideau river are about the best ones that can be carried to prevent jamming and, that they are fine tuned to the actual hydraulic and glaciological regime of the river. Very little amelioration can be suggested to improve on what is presently being done. The knowledge of Mr. Fretag on ice control on this reach of the river and its winter regime is outstanding.

However, because this program has a high annual cost I believe that studies should be done either to improve, if possible, the ice control operations or even try to design permanent works that would reduce their cost.

First of all, a comprehensive report is needed on the winter regime of this reach of the river, on the evolution of the ice cover from the time of formation to final breakup, on the ice control operations, their cost and effectiveness in preventing flooding. This report should also include an hydrological study of the effect of flash floods in winter on the thickening of the solid ice cover as well as on the pre-breakup use of water from the storage dams upstream to improve the ice clearing operations. Conclusions should be drawn relating to the best modes of water storage management.

.../2

A second report should deal with small possible improvements on the actual ice control operations. This should include full mechanization of the key cutting operations and study of alternatives to blasting, like the use of ACV vehicles, other floating ice-breaking devices, dusting or other means. This engineering report could also contain a preliminary study with cost analysis of more permanent works to reduce or eliminate the ice control operations. These works would essentially be levees and dikes and river bed corrections or excavations. Most important would be the construction of a weir to replace the Strathcona rapids, stabilizing the ice upstream of it and cutting the anchor ice production downstream.

I think that these are essentially the conclusions of my visit and discussions on this project and may I say that I enjoyed very much the hospitality combined with the wonderful weather we had for the tour.

Yours truly,

Bernard Michel

Bernard Michel, Dr. Eng.

BM/dd
Enclosure: invoice

APPENDIX D

REPORT ON REVIEW OF HYDROMETRIC SURVEY DATA TO 1966 FOR RIDEAU RIVER AT OTTAWA STATION NO. 2LA2

By D.K. Randall
Water Survey of Canada
Inland Waters Branch
Department of Energy Mines and Resources

February 26, 1968

Rideau River at Ottawa

Station No. 2 LA-2

Contents

DEPARTMENT OF ENERGY, MINES AND RESOURCES

INLAND WATERS BRANCH - WATER SURVEY OF CANADA

STATION HISTORY

for Rideau River at Ottawa Station No. 2 L

Period of Record:

Open water operation from May 1933 to November 1945; continuous operation from Apr, 1946 to November, 1947 and continuous operation from April 1948 to September 1966, when the station was discontinued.

Location:

Latitude 45° 24' 56", longitude 75° 39' 49", Ontario, at Canadian National Railways bridge, upstream from Hurdmans Bridge at Ottawa.

Discharge Measurements:

From bridge, boat and by wading.

Types of Gauges:

Reference point on bridge from April 1933.

Wire weight box gauge from May 19[illegible].

Elevation of Gauge Datums:

0.00 feet (D.P.W. 1905 datum) from May 1, 1933 to May 17, 1955.
[illegible].30 feet (G.S.C. datum 1949 adj.) from May 18, 1955 to September 30, 1955.
[illegible] feet (G.S.C. datum 1949 adj.) from [illegible] 1, 1955 to September 30, 1965.
[illegible] feet (G.S.C. datum 1964 adj.) from [illegible] 1, 1965 to September 30, 1966.

Water levels at this station were originally referred to a Department of Public Works datum, as from information extracted from a Department of Public Works daily gauge height and discharge record form for 1922-23, it is found that water levels were referred to Bench Mark No. [illegible], elevation 194.68 feet, by levels taken March 21, 1921. The bench mark could not be found in the two publications, Geodetic Survey of Canada Publication No. 57, and Geodetic Survey of Canada Publication No. 19, which were available in the District Office. By information received from Geodetic Survey of Canada, it was found that this bench mark was established by the Department of Public Works, in 1905 at an elevation of 194.71 feet, connected by Geodetic Survey of Canada in 1913 to give an elevation of 195.254 feet, and adjusted in 1929 to an elevation of 194.553 feet. It is not known why the Department of Public Works used an elevation of 194.68 feet, but for simplicity, the review staff will assume that water levels were referred to the original Department of Public Works, 1905 datum.

When the Branch assumed the operation of this station in April 1933 the Department of Public Works datum was retained until sometime probably in 1955.

SUMMARY OF REVISIONS

for Rideau River at Ottawa Station No. 2 LA-2

Comments: (cont'd)

1955

Mean discharges for June 29 was computed as 364 cfs. and should be 255 cfs., a +30% error in transferring gauge heights from observer's book to R79-B as above. No revision is necessary.

For	December 22	the discharge was	2510 cfs.	and should be	2430 cfs.,	+3%	error
"	July 25	" " "	251 cfs.	" " "	260 cfs.,	-4%	error
"	September 7	" " "	216 cfs.	" " "	255 cfs.,	-18%	error

On December 22, July 25 and September 7, discharge measurements were taken and the measured flow was used as the mean discharge for the day. The reason why this was done was not apparent, however the percent differences involved if the mean gauge height for the day were used are:

December 22	+ 3%
July 25	- 4%
September 7	-18%

Therefore original discharges for these days were accepted.

1958

Minimum daily discharge for June was published as 240 cfs. and should be 232 cfs.

NOTE:

Discharges for the 1946, 1947, 1948 and 1949 water years are published in the 1949-50 and 1950-51 publication - Water Resources Paper No. 107.

DEPARTMENT OF ENERGY, MINES [illegible] RESOURCES

INLAND WATERS BRANCH - WATER SURVEY OF CANADA

STATION HISTORY

for Rideau River at Ottawa Station No. 2 LA-

From information extracted from a Gauge Record Card, the datum was then corrected by +0.08 feet to a so called "Geodetic Survey of Canada datum". The review staff set out to find why the datum was corrected. There were no level notes available to indicate a change. Later, it was learned that the +0.08 feet adjustment was made because Geodetic Survey of Canada had made a general adjustment of levels in the area. From information received from Geodetic Survey of Canada datum, it was found that one bench mark, located near the gauging site, No. 503, was changed in elevation from 195.530 feet to 195.540 feet, by the 1949 Geodetic Survey of Canada level adjustment. The adjustment in elevation of Bench Mark No. 503 was approximately +0.[illegible] feet and this is believed the basis for the adjustment of levels at the gauge by +0.0[illegible] feet. It should be noted that the elevation of Bench Mark No. [illegible] was only adjusted +0.025 feet by the same 1949 Geodetic Survey of Canada adjustment, but this could be a reflection on the stability of the Geodetic Survey of Canada Bench Marks themselves.

This change in datum of +0.0[illegible] feet appears to have been applied on May 1[illegible], 1955 when a new [illegible] gauge was installed to replace the former reference point. This datum was used to the end of the 1955 water year.

On August 27, 1955, levels were run in order to tie-in water levels at the gauge with Geodetic Survey of Canada Bench Mark No. [illegible] which was established in 1904. From these levels the new datum at the gauge was found to be [illegible].15 feet lower than the former Geodetic Survey of Canada 1949 datum.

Bench Marks:

Bench mark on base of north rail on Canadian National Railways bridge at gauge, elevation [illegible] feet (G.S.C. datum, 19[illegible] adj.). This bench mark was first mentioned in level notes dated March [illegible], 19[illegible], [illegible] "[illegible]".

It is believed that this bench mark was the reference point used from 1933 to 1955 when it had an elevation of 19[illegible].5[illegible] feet (G.S.C. 19[illegible] datum). By the levels taken August 27, 1955, the elevation of this bench mark was found to be 198.47 feet (G.S.C. datum, 1949 adj.).

Remarks:

This station was operated by Department of Public Works, Ottawa, from 1[illegible] to 1933.

Prior to the 1958 water year records were published under the title "Rideau River at Hurdmans Bridge".

This station was discontinued in September 196[illegible]. For records after this date see Ottawa River at Ottawa, Station No. 2 LA-4.

DEPARTMENT OF NORTHERN AFFAIRS AND NATIONAL RESOURCES

WATER RESOURCES BRANCH – HYDROMETRIC SURVEY DATA REVIEW SECTION

Extremes of Discharge in cubic feet per second

of Rideau River at Ottawa Station No. 2 LA-2

Water Year	Daily Discharges in WRP No.	Maximum Instantaneous Discharge	Date	Maximum Daily Discharge	Date	Minimum Daily Discharge	Date
1933	z	n	--	r	--	-- q	--
1934	z	n	--	r	--	162	June 1,2,4-6
1935	z	n	--	r	--	138	June 3-7
1936	z	n	--	r	--	127	June 21
1937	z	n	--	r	--	89	Sept. 3
1938	z	n	--	r	--	150	July 5,6 &7
1939	z	n	--	r	--	138	Nov. 3
1940	z	n	--	r	--	208	Aug. 21,22
1941	z	n	--	r	--	150	May31, June 1
1942	z	n	--	r	--	191	Oct. 6,7
1943	z	n	--	r	--	127	June 10
1944	z	n	--	r	--	208	Various time
1945	z	n	--	11300	May 19	-- q	--
1946	107	n	--	~~15000~~ s	~~Mar. 9~~	250	Various ti
1947	107	n	--	19000	Apr. 12	208	Oct. 6
1948	107	n	--	~~14300~~ s	~~Mar. 20~~	250	May 1
1949	107	n	--	13400	Mar. 30,31	176	Various tin
1950	107	n	--	15800	Apr. 5	162	Oct. 26
1951	107	n	--	14800	Apr. 2	176	May 16
1952	111	n	--	13400	Apr. 6	250	July 7
1953	111	n	--	11700	Mar. 28	208	July3, Aug.3
1954	115	n	--	14300	Apr. 8	250	Nov. 3,4
1955	115	n	--	17400	Apr. 6,7	160	May 25
1956	119	n	--	12400	Apr. 5	138	Sept. 17
1957	119	n	--	4680	Mar. 15	117	May 13
1958	126	n	--	10800	Mar. 31	105	Dec. 19
1959	129	n	--	14600	Apr. 6	191	June 7
1960	133	n	--	13800	Apr. 14	150	July 5
1961	137	n	--	6810	Mar. 30	108 eb	Feb. 10 to 20
1962	140	n	--	11400 eb	Apr. 1	155 eb	Mar. 5
1963	143	n	--	15600	Mar. 31	159	July 4
1964	147	n	--	3850	Apr. 23	162	Sept. 30
1965	1965	n	--	5090	Apr. 13,17	123 eb	Feb. 1-11
1966	--	n	--	7650	Mar. 6	184	July 7,31

q – Not determined; minimum may have occurred during period when gauge was not operating.
eb – Estimated under ice conditions.
n – Not determined; insufficient readings to define peak.
z – Records were not published.
r – Records were not reliable from December 1 to April 30.
s – *Records were not reliable from December 1 to MARCH 31.*

Prepared by H.A. McGarvey

DEPARTMENT OF ENERGY, MINES AND RESOURCES

R 239
(9-67)

INLAND WATERS BRANCH-WATER SUR__ OF CANADA

SUMMARY OF REVISIONS

for Rideau River at Ottawa Station No. 2 LA-2

Period of Record Reviewed:

From May 1933 to September 1966.

Revisions:

1933 to 1945, 1946 and 1948.

Comments:

General:

Since the reference point used from 1933 to 1955 was the base of a rail and this reference point was acknowledged as the bench mark for the station, through to 1966 when the station was discontinued, then there is a good possibility of variations in elevations at the gauge throughout the years.

This station has been operated by the Branch since April 1, 1933 even though the publication has only acknowledged records beginning October 1, 1945. After reviewing these previously unpublished records from the 1933 to the 1945 water years, the review staff noted that ice corrections were never made during the winter months of these years. Therefore, only open water records should be published. The beginning and terminating dates of the ice period for each water year could not be determined. An attempt was made to determine whether or not the daily maximum discharges occurred during open water, but it was revealed that in some cases the maximum discharge has occurred during the ice period. Therefore, it was decided to only publish months which were always completely free of ice, namely October, November, and May to September. As a result, the maximum daily discharges will not be published for any of the years except for 1945, when the maximum is known to have occurred on May 19, 1945 when the maximum water level occurred during open water.

It should be noted that although the discharges for the ice periods are questionable, the water level data which were collected are reliable. Therefore, the maximum daily water level which occurred was 189.45 feet G.S.C. on March 24, 1938. Two other extreme water levels occurred on April 12, 1947 (189.42) and on April 14, 1960 (188.72).

1946 and 1948

The winter records for these two years were published (except for January and February 1948) but it was noted that corrections due to ice conditions were not made. Therefore, the winter records are unreliable and will not be published. The beginning of the ice periods could not be determined, but the hydrographs indicate that the spring freshets occurred in the middle of March. It was therefore decided that the open water period for which data would be published was from April 1 to November 30, for the 1946 and 1948 water years. The maximum daily discharges occurred during the spring freshets, so they will not be published.

SUMMARY OF REVISIONS

for Rideau River at Ottawa Station No. 2 LA-2

1951

Errors were made in transferring gauge heights from observer's book to R79-A for April 9 and 15. As a result of this the mean discharge for April 9 was computed as 8940 cfs. and should be 8800 cfs. a +2% error. The mean discharge for April 15 was computed as 7490 cfs. and should be 7540 cfs., a -1% error. Since both errors are less than the 50% review criterion, then no revisions are necessary.

Prepared by H. A. McGarvey
Date October 18, 1967

Approved by D. W. Kirk
Date June 16, 1971

Sheet of

SUMMARY OF REVISIONS

Rideau River at Ottawa 02LA002

1947

The original records were calculated with allowances being made for ice conditions in December, January and February. It is not known how these ice conditions were calculated since there were no winter measurements made on which to base the estimates.

A look at the temperature and precipitation charts (see hydrograph) would suggest an ice period extending into March, which disagrees with the original calculations. Another indication that additional backwater reductions should be made is the fact that the discharges for January, February and March were exceptionally high compared with other years.

However, the monthly precipitations were well above average throughout the winter and the monthly mean temperatures (except for February) were also above the average, which suggests an unusually high winter runoff. These high discharges were further supported when compared with other stations in the vicinity, e.g. Mississippi River at Appleton, Bonnechere River near Castleford and Petawawa River near Petawawa.

Revisions will not be made since they cannot be substantiated.

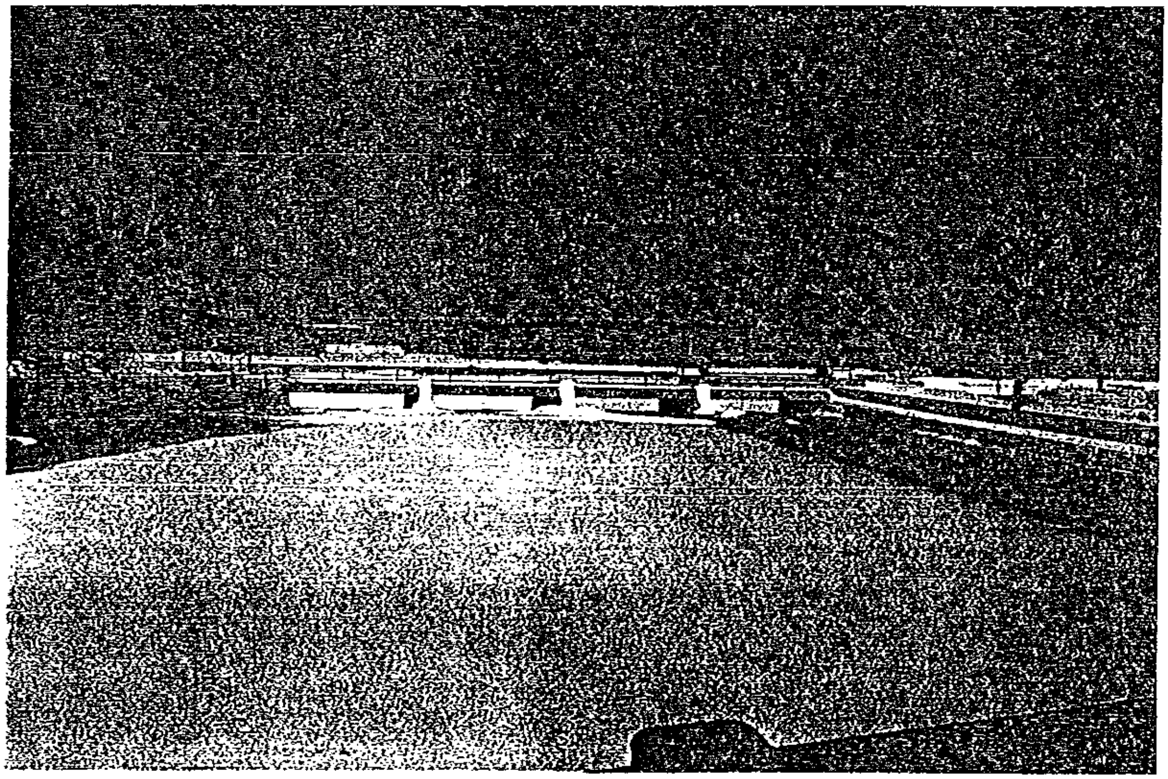
APPENDIX E

BRIDGE DATA TABLES

BRIDGE DATA

Watercourse: RIDEAU RIVER	Location: RIDEAU FALLS DAM (West)

Map Sheet No.: 2	Cross-Section No.: 37



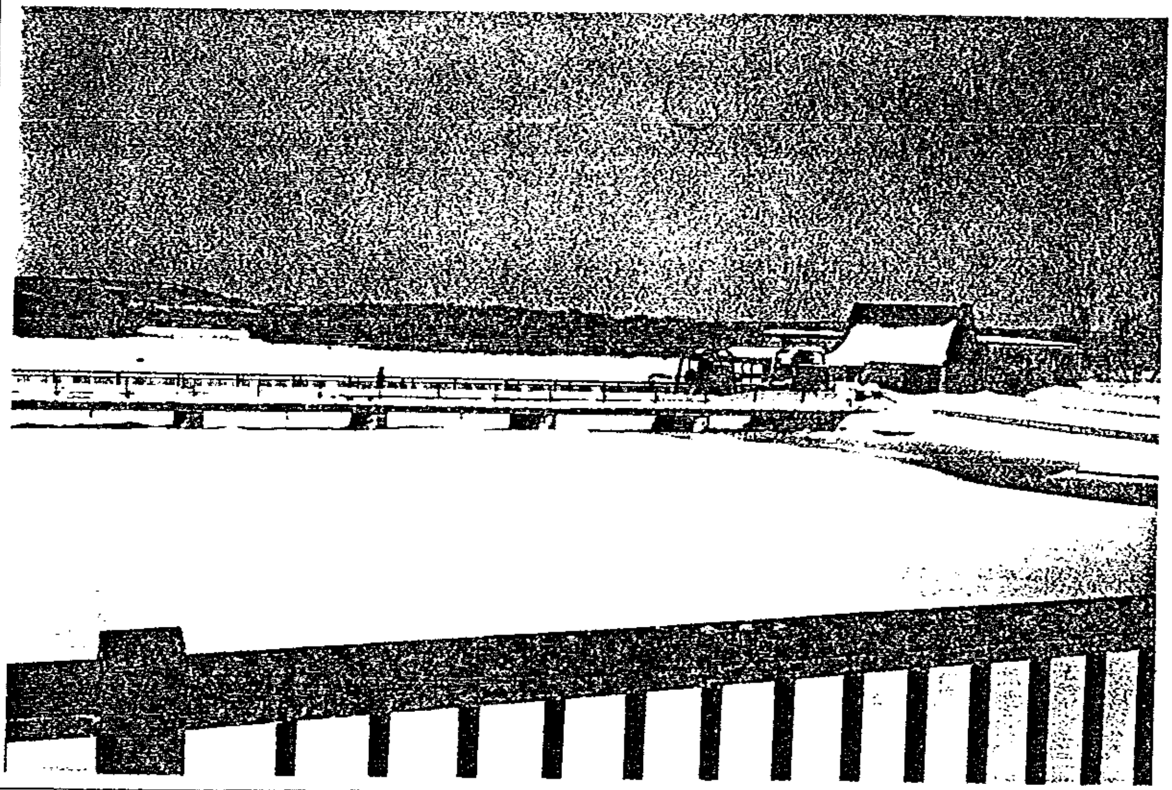
VIEW OF UPSTREAM FACE

SPECIFICATIONS	UPSTREAM	DOWNSTREAM
Length of Structure	7.0 m	
Top of Road	56.26 m	
Span	34.1 m	
Low Chord Elevations	56.06 m	
Inverts	52.26 m	51.71 m
Effective Flow Area	119.0 m^2	
Manning's "n" Values	0.026	
Additional Details		

BRIDGE DATA

Watercourse: RIDEAU RIVER   Location: RIDEAU FALLS DAM (East)

Map Sheet No.: 2   Cross-Section No.: 49



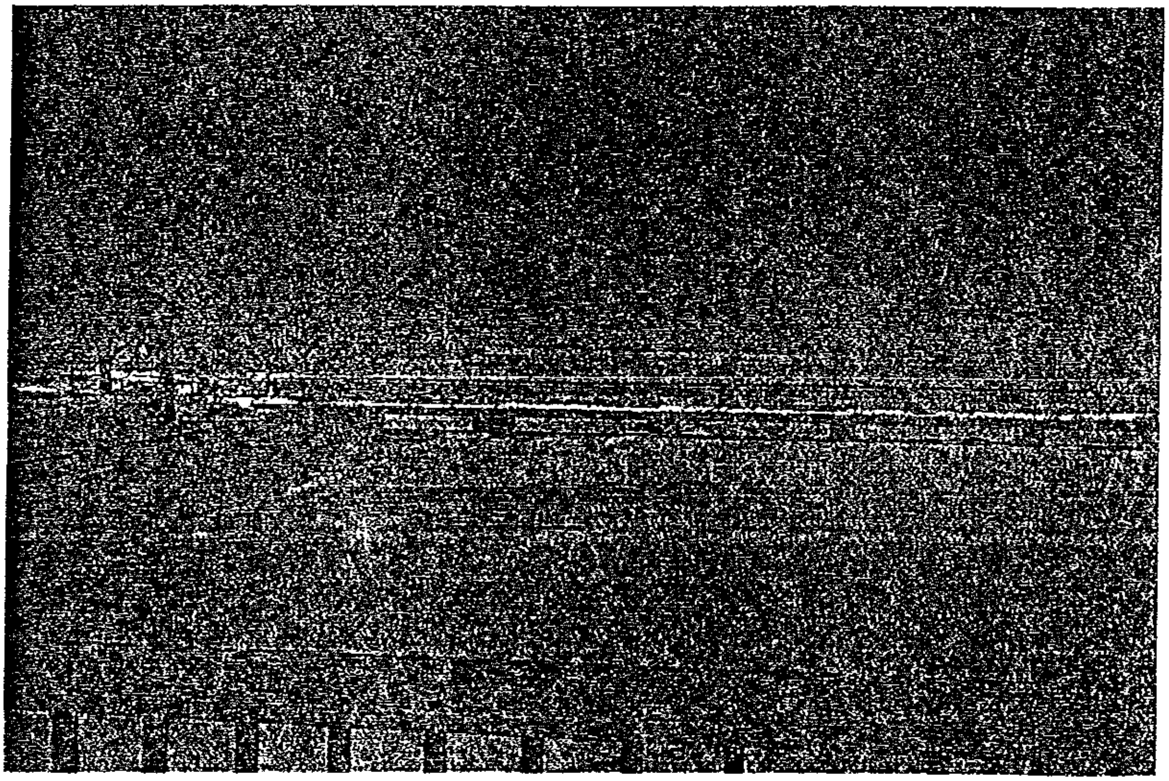
VIEW OF UPSTREAM FACE

SPECIFICATIONS	UPSTREAM	DOWNSTREAM
Length of Structure	7.9 m	
Top of Road	56.48 m	
Span	68.4 m	
Low Chord Elevations	55.57 m	
Inverts	51.83	50.56 m
Effective Flow Area	226.0 m2	
Manning's "n" Values	0.026	
Additional Details		

BRIDGE DATA

Watercourse: RIDEAU RIVER Location: RIDEAU FALLS DAM (East)

Map Sheet No.: 2 Cross-Section No.: 49



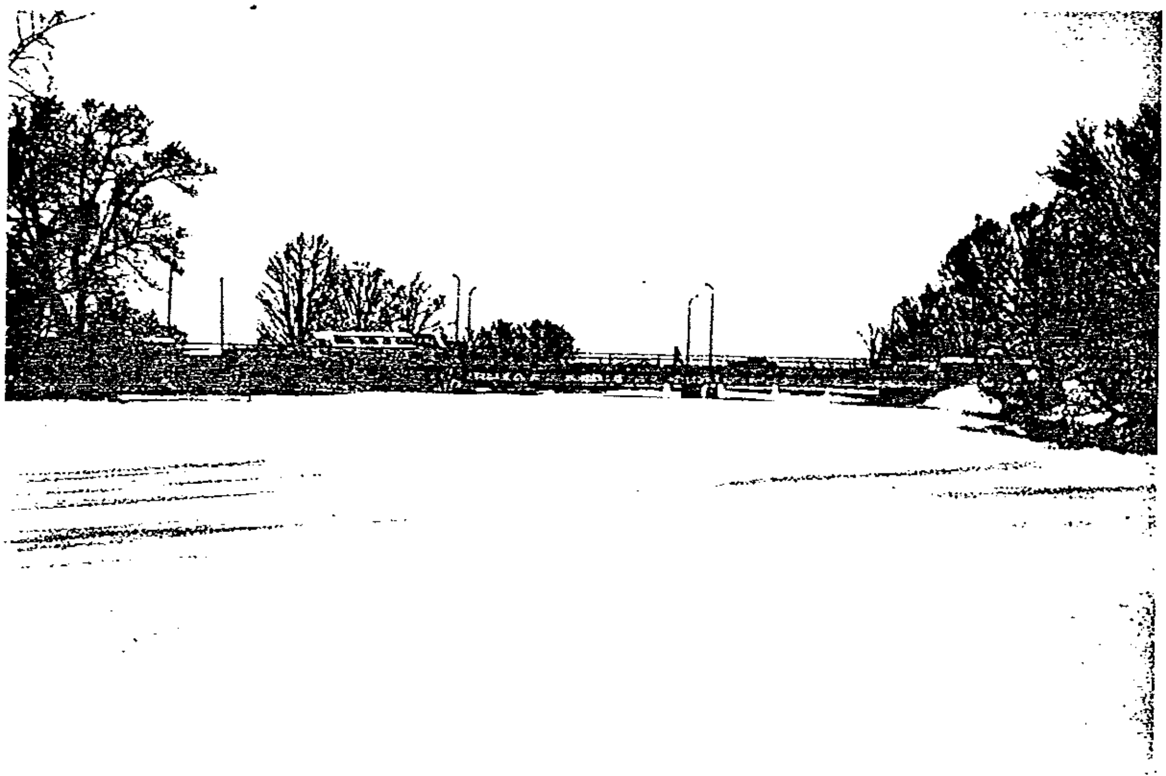
VIEW OF UPSTREAM FACE

SPECIFICATIONS	UPSTREAM	DOWNSTREAM
Length of Structure	7.9 m	
Top of Road	56.48 m	
Span	68.4 m	
Low Chord Elevations	55.57 m	
Inverts	51.83	50.56 m
Effective Flow Area	226.0 m^2	
Manning's "n" Values	0.026	
Additional Details		

BRIDGE DATA

Watercourse: RIDEAU RIVER   Location: BRIDGE @ SUSSEX DR. (W.)

Map Sheet No.: 2   Cross-Section No.: 145



VIEW OF UPSTREAM FACE

SPECIFICATIONS	UPSTREAM	DOWNSTREAM
Length of Structure	20.0 m	
Top of Road	57.4 m	
Span	81.6 m	
Low Chord Elevations	56.25 m	
Inverts	51.83 m	
Effective Flow Area	346 m^2	
Manning's "n" Values	0.026	
Additional Details	Reinforced concrete with 3 piers	

BRIDGE DATA

Watercourse: RIDEAU RIVER

Location: BRIDGE @ SUSSEX DR. (E.)

Map Sheet No.: 2

Cross-Section No.: 134

VIEW OF UPSTREAM FACE

SPECIFICATIONS	UPSTREAM	DOWNSTREAM
Length of Structure	20.0 m	
Top of Road	56.1 m	
Span	81.6 m	
Low Chord Elevations	56.8 m	
Inverts	52.80 m	
Effective Flow Area	305.0 m^2	
Manning's "n" Values	0.026	
Additional Details	Reinforced concrete with 2 piers	

BRIDGE DATA

Watercourse: RIDEAU RIVER | Location: MINTO BRIDGE (WEST)

Map Sheet No.: 2 | Cross-Section No.: 396

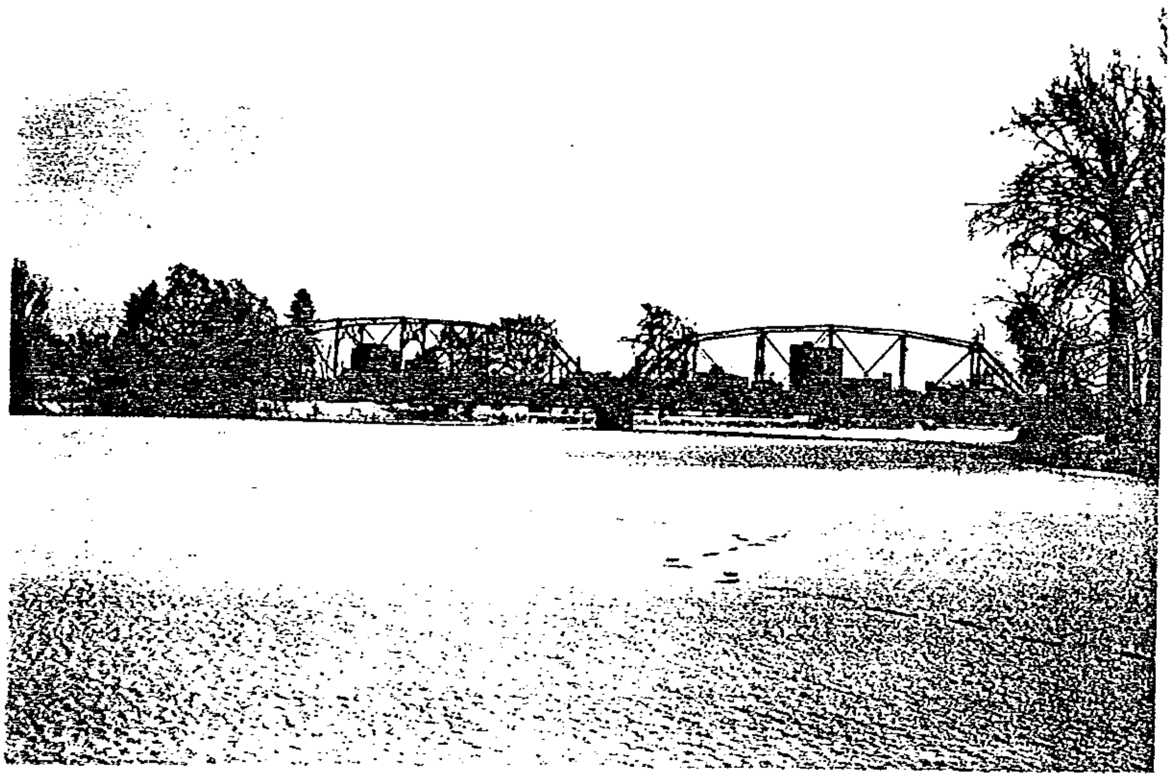
VIEW OF UPSTREAM FACE

SPECIFICATIONS	UPSTREAM	DOWNSTREAM
Length of Structure	11.0 m	
Top of Road	57.2 m	
Span	53.0 m	
Low Chord Elevations	56.56 m	
Inverts	52.71 m	
Effective Flow Area	204.0 m^2	
Manning's "n" Values	0.026	
Additional Details	Single span steel truss	

BRIDGE DATA

Watercourse: RIDEAU RIVER   Location: MINTO BRIDGE (Central)

Map Sheet No.: 2   Cross-Section No.: 396



VIEW OF UPSTREAM FACE

SPECIFICATIONS	UPSTREAM	DOWNSTREAM
Length of Structure	9.0 m	
Top of Road	57.30 m	
Span	69.90 m	
Low Chord Elevations	57.45 m	
Inverts	52.51 m	
Effective Flow Area	295.0 m^2	
Manning's "n" Values	0.026	
Additional Details	Steel truss with 1 pier	

BRIDGE DATA

Watercourse: RIDEAU RIVER

Location: MINTO BRIDGE (East)

Map Sheet No.: 2

Cross-Section No.: 396

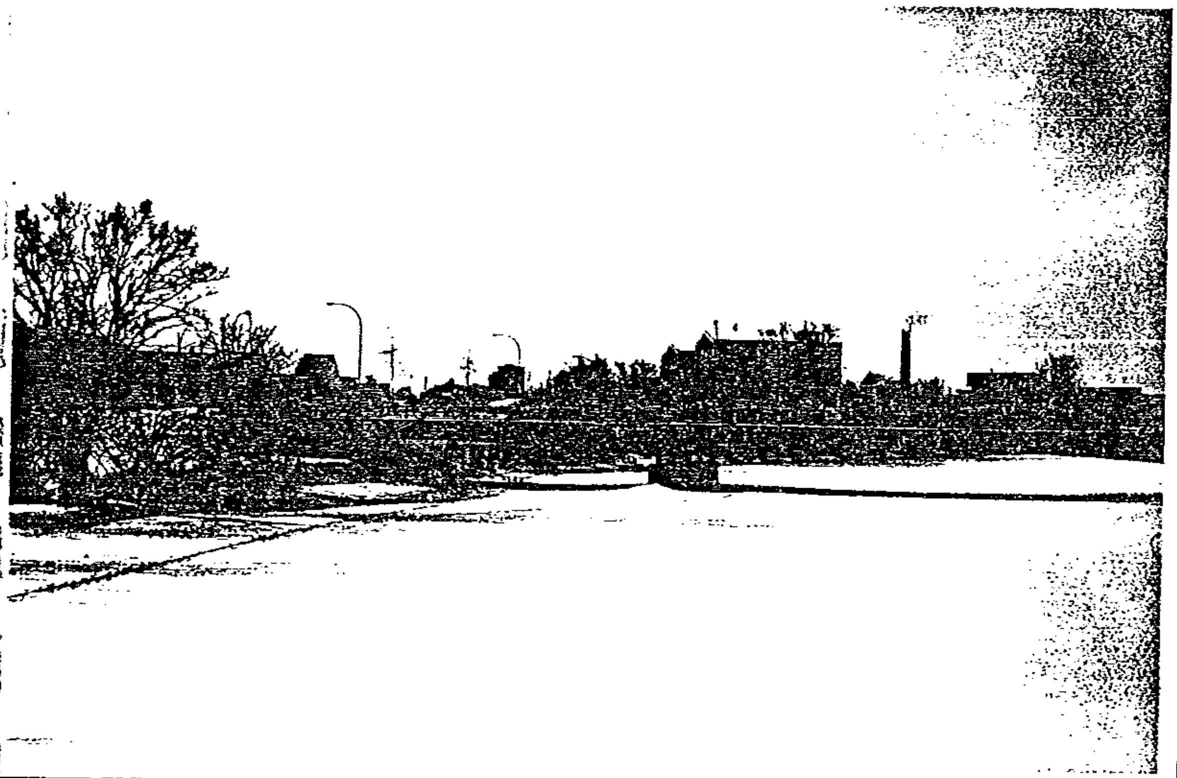
VIEW OF UPSTREAM FACE

SPECIFICATIONS	UPSTREAM	DOWNSTREAM
Length of Structure	9.0 m	
Top of Road	56.98 m	
Span	39.73 m	
Low Chord Elevations	55.98 m	
Inverts	52.51 m	
Effective Flow Area	75.0 m^2	
Manning's "n" Values	0.026	
Additional Details	Single span steel truss	

BRIDGE DATA

Watercourse: RIDEAU RIVER   Location: PORTER ISLAND (North)

Map Sheet No.: 3   Cross-Section No.: 1410



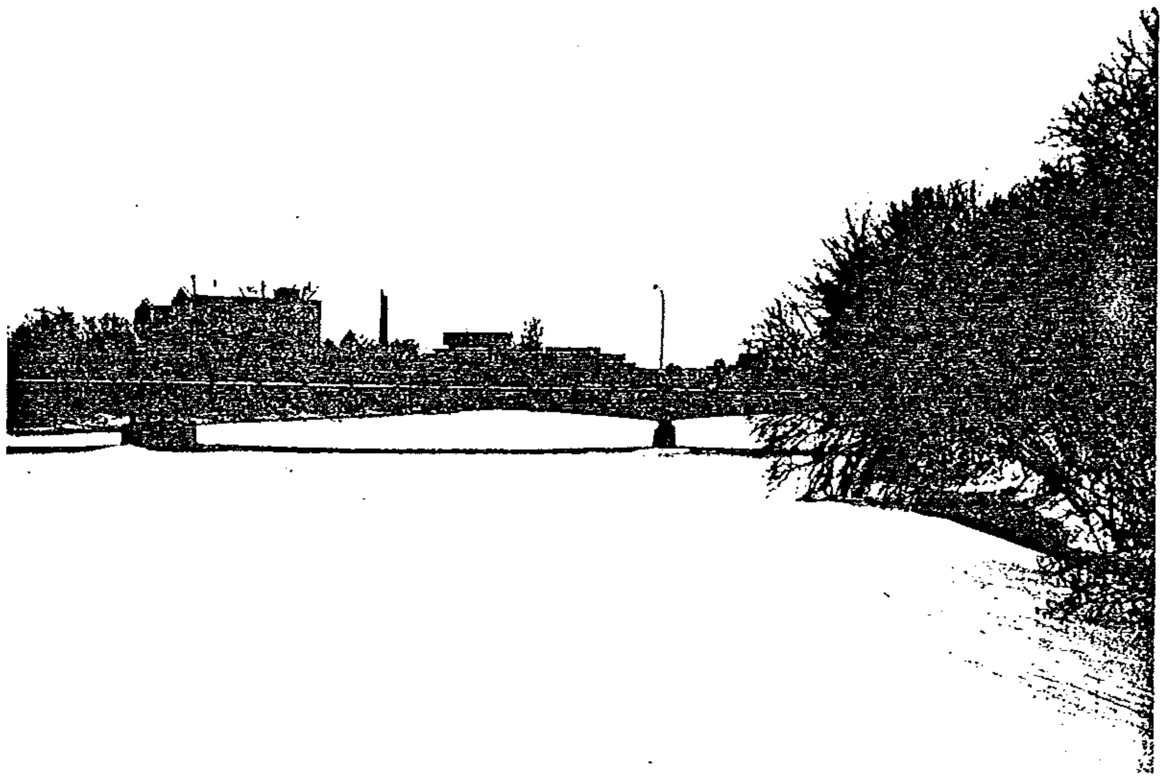
VIEW OF UPSTREAM FACE

SPECIFICATIONS	UPSTREAM	DOWNSTREAM
Length of Structure	11.0 m	
Top of Road	59.1 m	
Span	79.9 m	
Low Chord Elevations	58.45 m	
Inverts	52.50 m	
Effective Flow Area	444.0 m2	
Manning's "n" Values	0.035	
Additional Details	Steel girder with 2 piers	

BRIDGE DATA

Watercourse: RIDEAU RIVER   Location: PORTER ISLAND (North)

Map Sheet No.: 3   Cross-Section No.: 1410



VIEW OF UPSTREAM FACE

SPECIFICATIONS	UPSTREAM	DOWNSTREAM
Length of Structure	11.0 m	
Top of Road	59.1 m	
Span	79.9 m	
Low Chord Elevations	58.45 m	
Inverts	52.50 m	
Effective Flow Area	444.0 m^2	
Manning's "n" Values	0.035	
Additional Details	Steel girder with 2 piers	

BRIDGE DATA

Watercourse: RIDEAU RIVER   Location: PORTER ISLAND (South)

Map Sheet No.: 3   Cross-Section No.: 1515

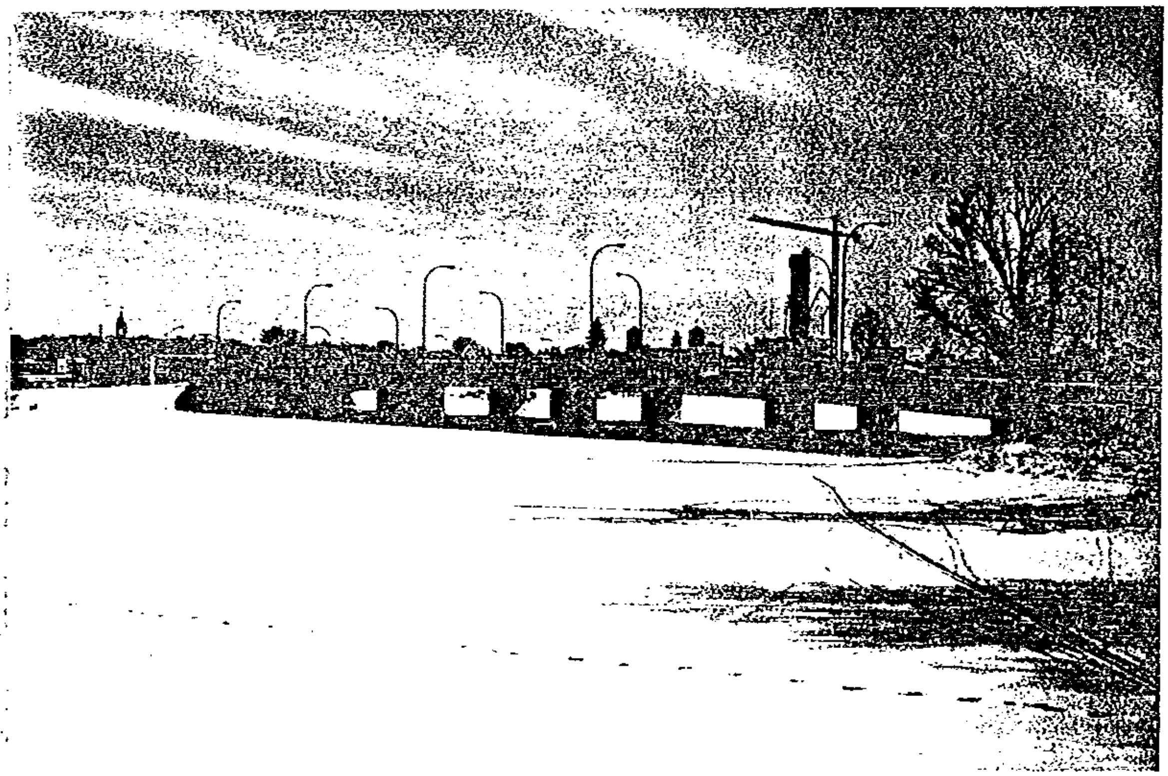
VIEW OF UPSTREAM FACE

SPECIFICATIONS	UPSTREAM	DOWNSTREAM
Length of Structure	4.0 m	
Top of Road	58.40 m	
Span	62.0 m	
Low Chord Elevations	57.91 m	
Inverts	52.34 m	
Effective Flow Area	340.0 m^2	
Manning's "n" Values	0.035	
Additional Details	Steel truss with 1 pier.	

BRIDGE DATA

Watercourse: RIDEAU RIVER   Location: ST. PATRICK ST. BRIDGE

Map Sheet No.: 3   Cross-Section No.: 1720



VIEW OF UPSTREAM FACE

SPECIFICATIONS	UPSTREAM	DOWNSTREAM
Length of Structure	30.0 m	
Top of Road	57.0 m	
Span	154.8 m	
Low Chord Elevations	58.5 m	
Inverts	52.4 m	
Effective Flow Area	444.0 m^2	
Manning's "n" Values	0.035	
Additional Details	Post-Tensioned Concrete with 4 piers	

BRIDGE DATA

Watercourse: RIDEAU RIVER Location: CUMMINGS BRIDGE

Map Sheet No.: 4 Cross-Section No.: 2490

VIEW OF UPSTREAM FACE

SPECIFICATIONS	UPSTREAM	DOWNSTREAM
Length of Structure	16.0 m	
Top of Road	varies between 59.49 & 64.13 m	
Span	146.3 m	
Low Chord Elevations	varies between 57.94 & 62.36 m	
Inverts	52.78 m	
Effective Flow Area	208.5 m^2	
Manning's "n" Values	0.035	
Additional Details	Reinforced concrete arched bridge with 7 piers of var. widths & heights	

BRIDGE DATA

Watercourse: RIDEAU RIVER   Location: CUMMINGS BRIDGE

Map Sheet No.: 4   Cross-Section No.: 2490

VIEW OF UPSTREAM FACE

SPECIFICATIONS	UPSTREAM	DOWNSTREAM
Length of Structure	16.0 m	
Top of Road	varies between 59.49 & 64.13 m	
Span	146.3 m	
Low Chord Elevations	varies between 57.94 & 62.36 m	
Inverts	52.78 m	
Effective Flow Area	208.5 m2	
Manning's "n" Values	0.035	
Additional Details	Reinforced concrete arched bridge with 7 piers of var. widths & heights	

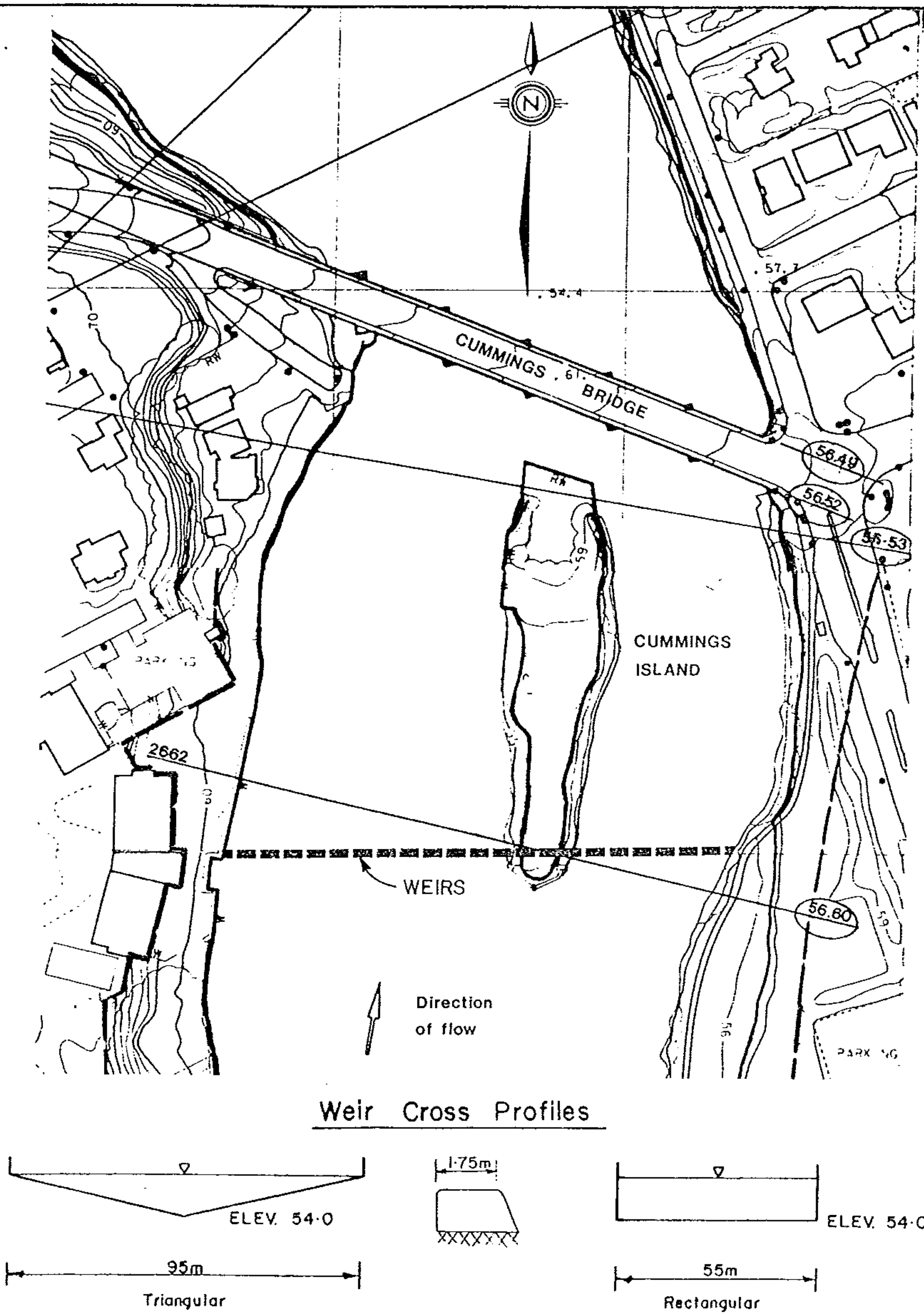
BRIDGE DATA

Watercourse: RIDEAU RIVER Location: OLD HURDMAN BRIDGE

Map Sheet No.: 7 Cross-Section No.: 4406

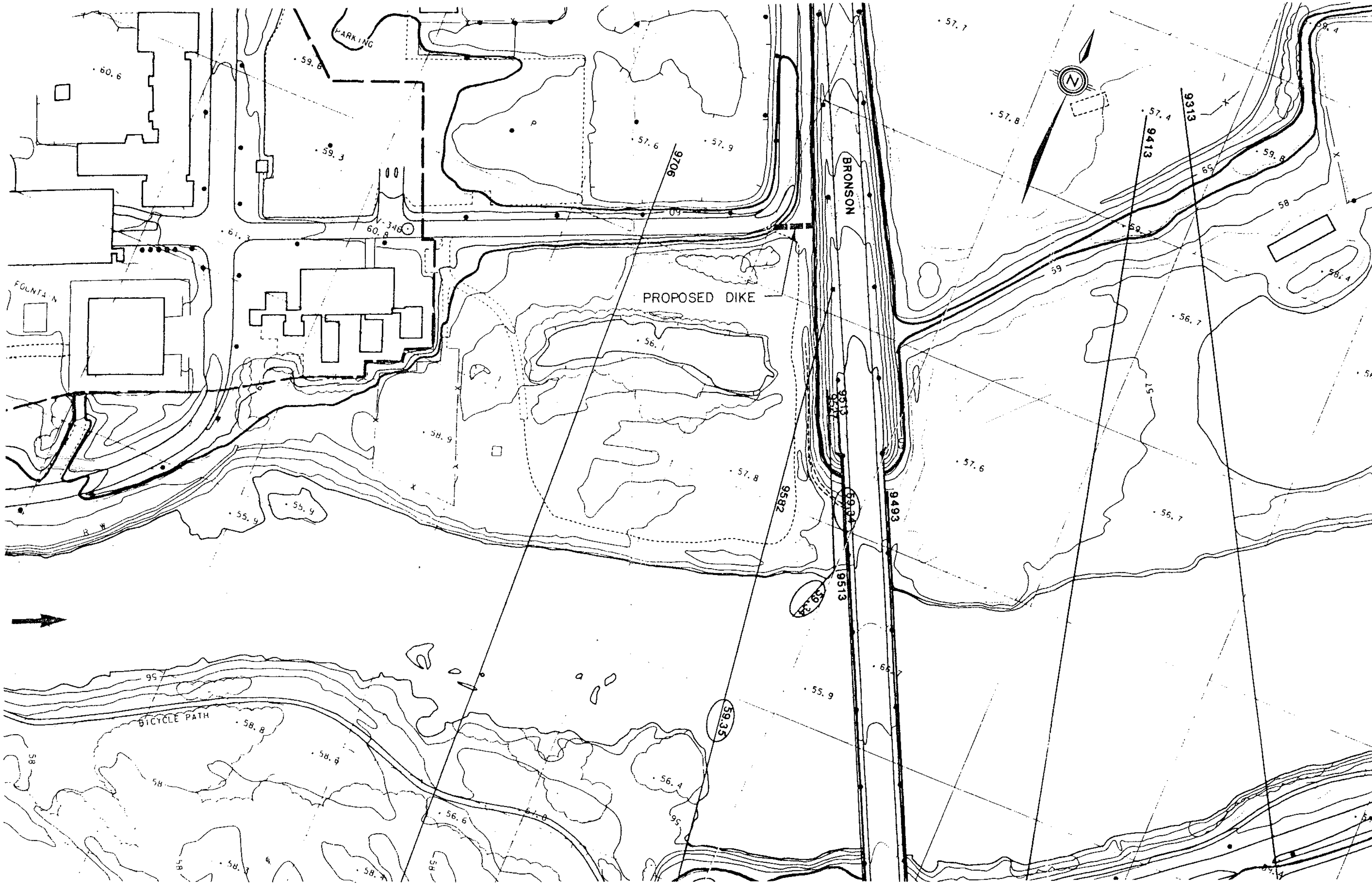
VIEW OF UPSTREAM FACE

SPECIFICATIONS	UPSTREAM	DOWNSTREAM
Length of Structure	7.5 m	
Top of Road	60.23 m	
Span	129.05 m	
Low Chord Elevations	59.25 m	
Inverts	54.93	
Effective Flow Area	530.0 m^2	
Manning's "n" Values	0.035	
Additional Details	Reinforced concrete arches with 6 piers	



LOCATION OF PROPOSED CUMMINGS ISLAND WEIRS

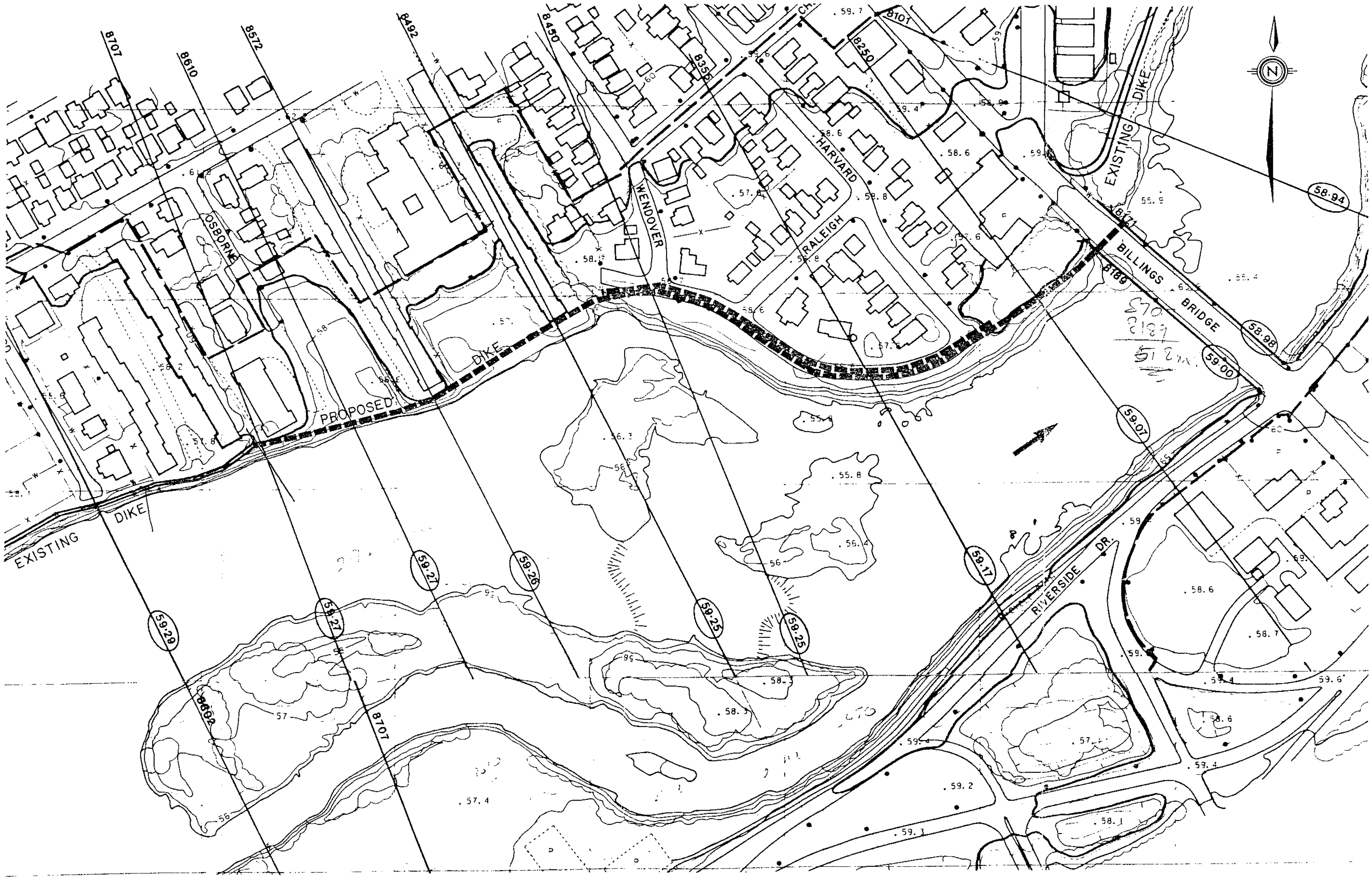
A.J. Robinson & Assoc. Scale 1:2000 Figure 9



Proposed Carleton University Dike

Scale 1:2000

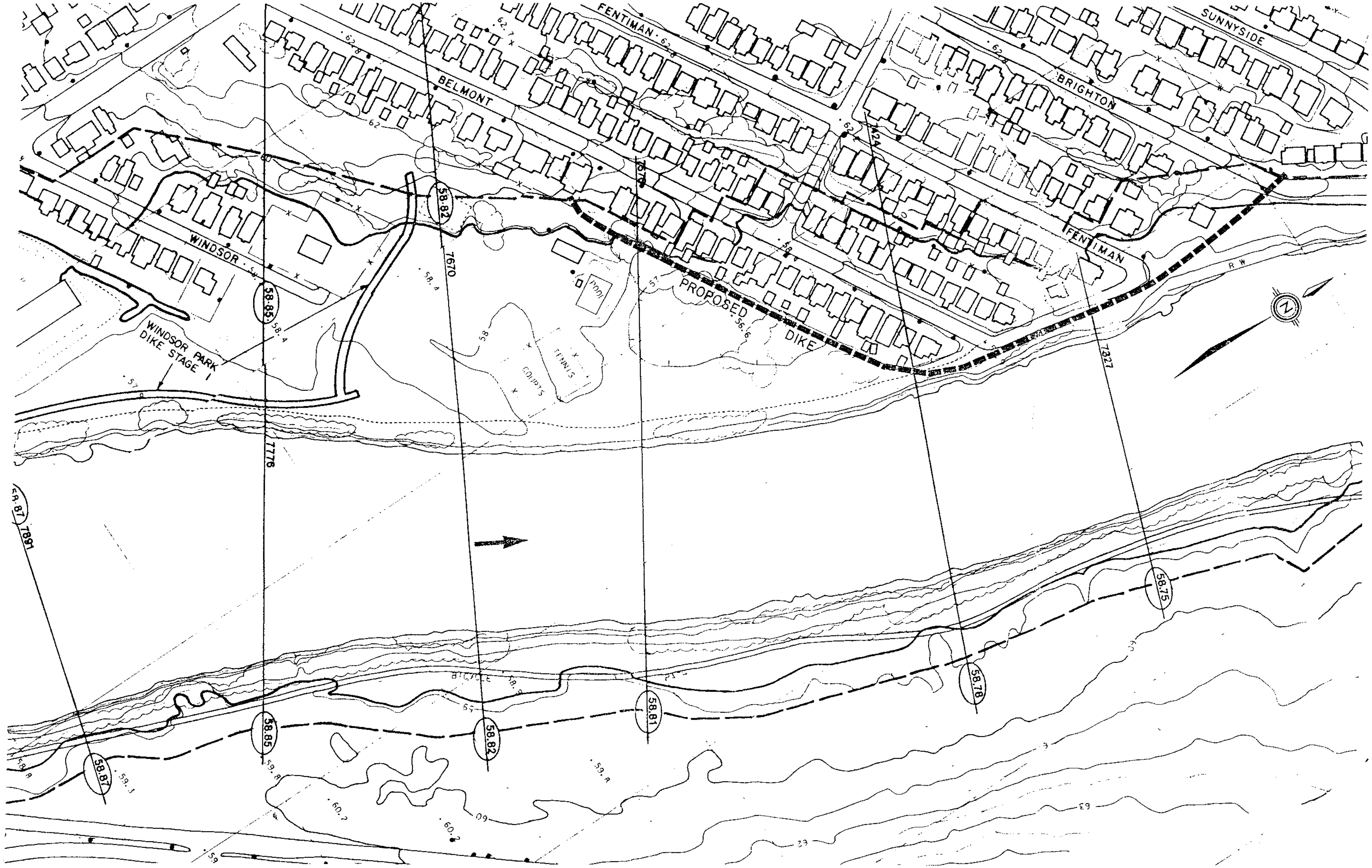
Figure 10



Scale 1:2000

Proposed Warrington Drive Dike

Figure 11



Proposed Windsor Park Stage II Dike

Scale 1:2000

Figure 12

Proposed Brantwood Park Dike

Scale 1:2000

Figure 13

Proposed New Edinburgh Dike

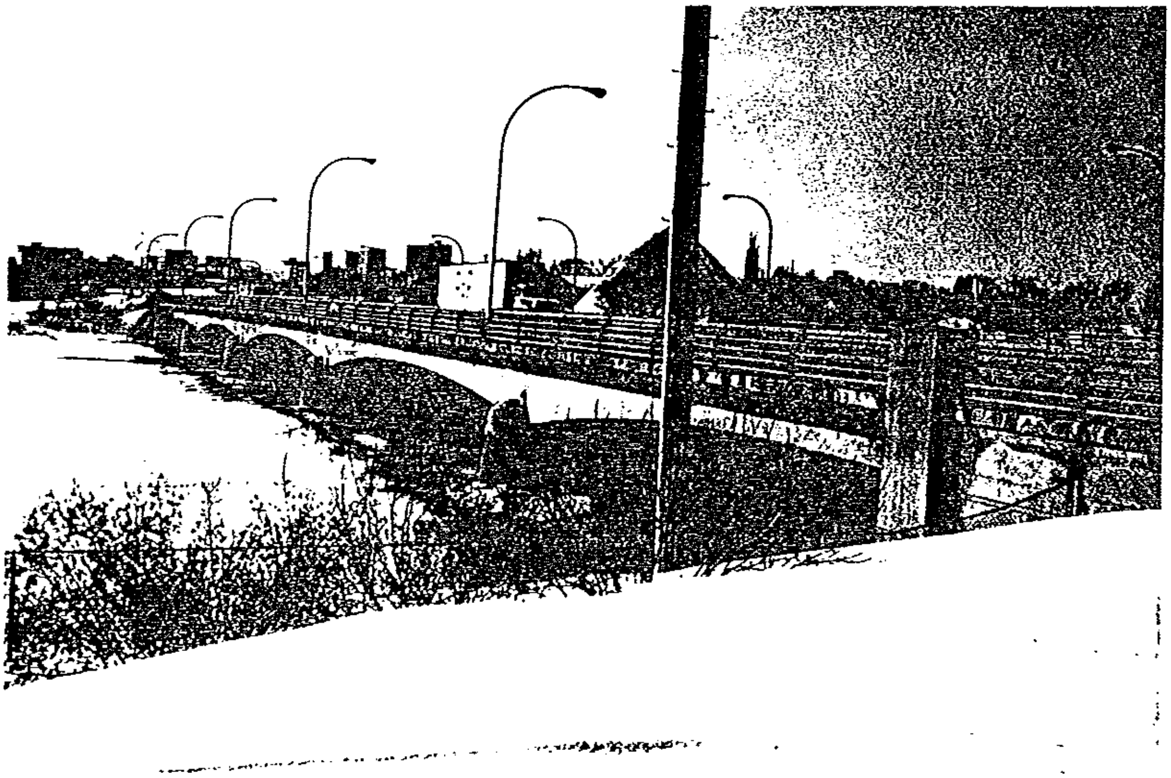
Scale 1:2000

Figure 14

BRIDGE DATA

Watercourse: RIDEAU RIVER　　Location: QUEENSWAY BRIDGE

Map Sheet No.: 7　　Cross-Section No.: 4464



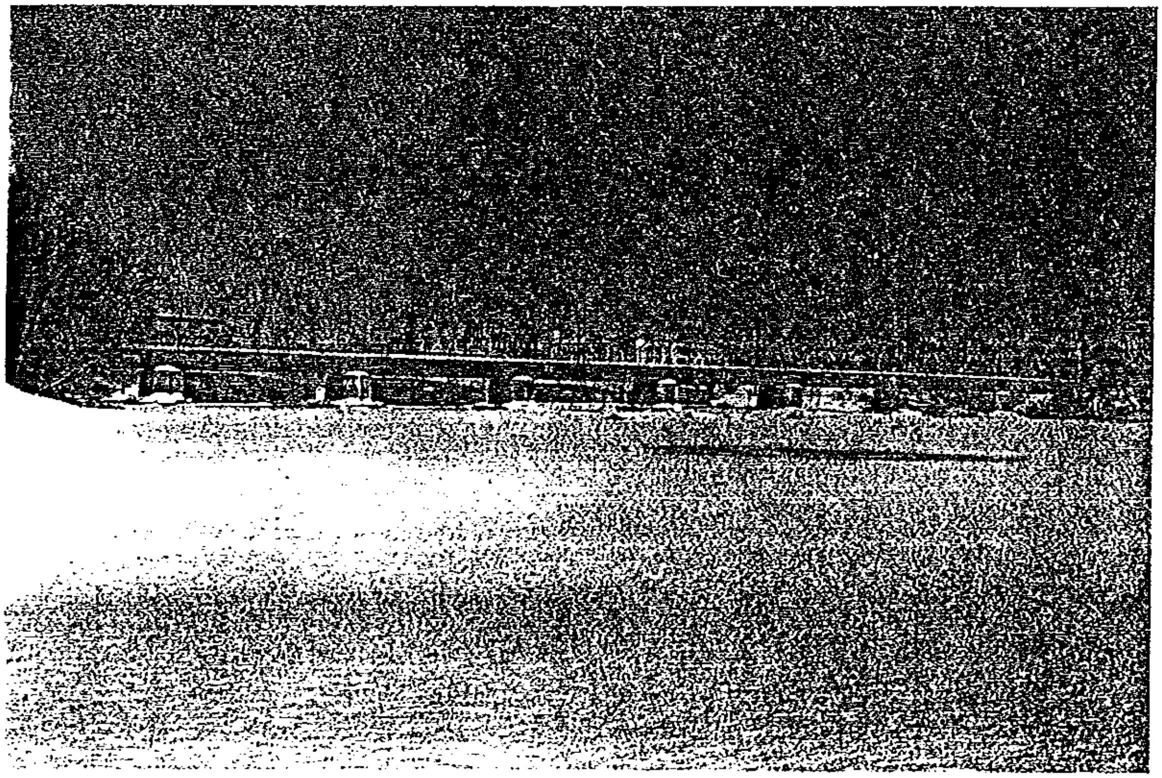
VIEW OF UPSTREAM FACE

SPECIFICATIONS	UPSTREAM	DOWNSTREAM
Length of Structure	30.0 m	
Top of Road	61.33 m	
Span	147.08 m	
Low Chord Elevations	60.13 m	
Inverts	55.17 m	
Effective Flow Area	634.0 m^2	
Manning's "n" Values	0.035	
Additional Details	Concrete beam and girder with 4 piers	

BRIDGE DATA

Watercourse: RIDEAU RIVER   Location: PEDESTRIAN BRIDGE

Map Sheet No.: 7   Cross-Section No.: 4524



VIEW OF UPSTREAM FACE

SPECIFICATIONS	UPSTREAM	DOWNSTREAM
Length of Structure	3.5 m	
Top of Road	60.1 m	
Span	153.6 m	
Low Chord Elevations	59.57 m	
Inverts	54.71 m	
Effective Flow Area	613 m^2	
Manning's "n" Values	0.036	
Additional Details	Steel railway bridge with 7 piers	

BRIDGE DATA

Watercourse: RIDEAU RIVER — Location: TRANSITWAY BRIDGE

Map Sheet No.: 8 — Cross-Section No.: 5052

VIEW OF UPSTREAM FACE

SPECIFICATIONS	UPSTREAM	DOWNSTREAM
Length of Structure	14.5 m	
Top of Road	59.25 m	
Span	144.3 m	
Low Chord Elevations	59.15 m	
Inverts	53.65 m	
Effective Flow Area	855.0 m^2	
Manning's "n" Values	0.033	
Additional Details	Concrete beam & girder with 2 piers	

BRIDGE DATA

Watercourse: RIDEAU RIVER

Location: SMYTH ROAD

Map Sheet No.: 10

Cross-Section No.: 6920

VIEW OF UPSTREAM FACE

SPECIFICATIONS	UPSTREAM	DOWNSTREAM
Length of Structure	21.0 m	
Top of Road	64.0 m	
Span	202.3 m	
Low Chord Elevations	63.07 m	
Inverts	56.42 m	
Effective Flow Area	1296 m^2	
Manning's "n" Values	0.040	
Additional Details	Reinforced concrete with 5 piers	

BRIDGE DATA

Watercourse: RIDEAU RIVER | Location: BILLINGS BRIDGE

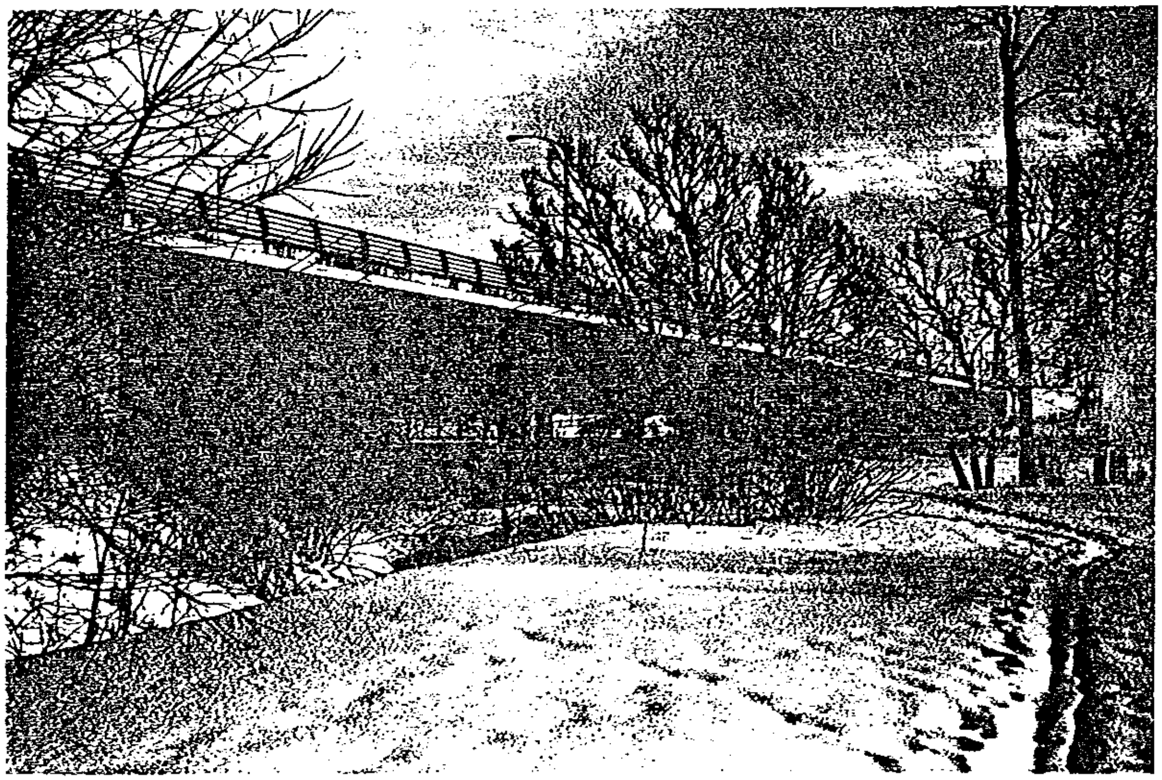
Map Sheet No.: 12 | Cross-Section No.: 8189

VIEW OF UPSTREAM FACE

SPECIFICATIONS	UPSTREAM	DOWNSTREAM
Length of Structure	18.5 m	
Top of Road	60.53 m	
Span	113.25 m	
Low Chord Elevations	59.22 m	
Inverts	55.06 m	
Effective Flow Area	441 m^2	
Manning's "n" Values	0.042	
Additional Details	Reinforced concrete with 4 piers	

BRIDGE DATA

Watercourse:	RIDEAU RIVER	Location:	DUNBAR BRIDGE
Map Sheet No.:	14	Cross-Section No.:	9513



VIEW OF UPSTREAM FACE

SPECIFICATIONS	UPSTREAM	DOWNSTREAM
Length of Structure	20.0 m	
Top of Road	65.8 m	
Span	261.5 m	
Low Chord Elevations	66.12 m	
Inverts	56.67 m	
Effective Flow Area	1496 m^2	
Manning's "n" Values	0.045	
Additional Details	Concrete beam and girder with 6 piers	

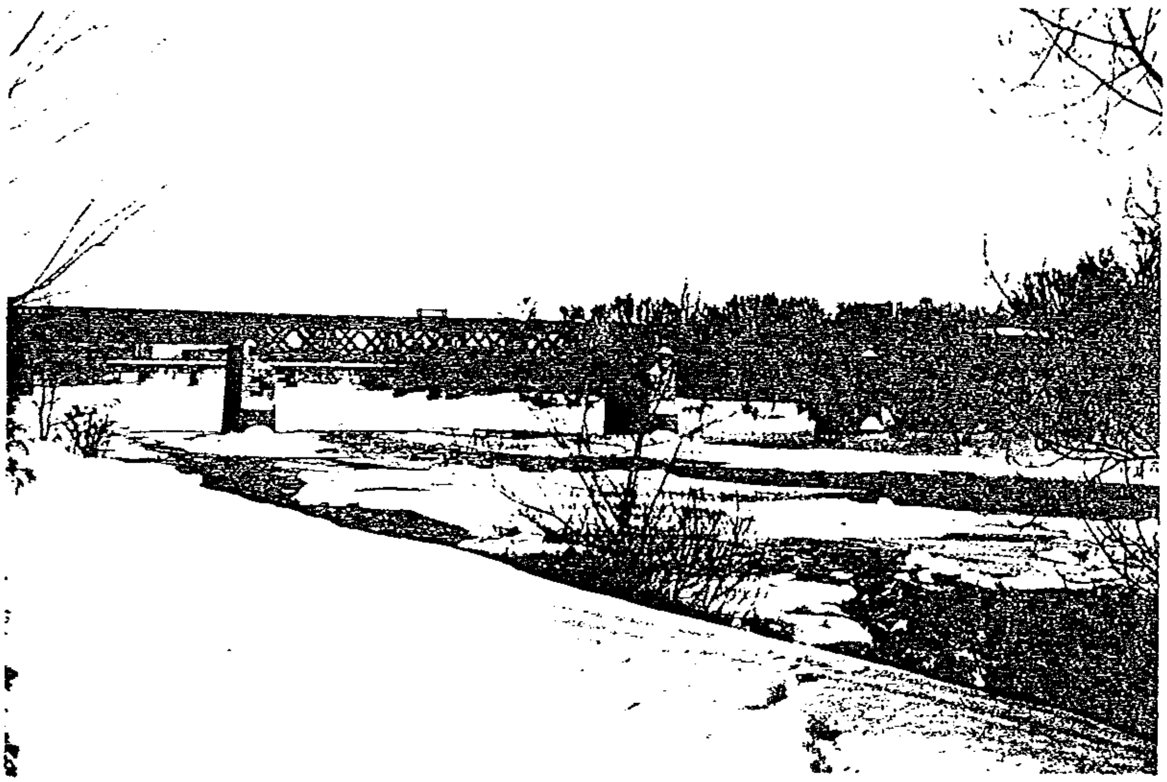
BRIDGE DATA

Watercourse: RIDEAU RIVER

Location: CPR BRIDGE AT CARLETON

Map Sheet No.: 15

Cross-Section No.: 10011



VIEW OF UPSTREAM FACE

SPECIFICATIONS	UPSTREAM	DOWNSTREAM
Length of Structure	5.0 m	
Top of Road	65.8 m	
Span	98.75 m	
Low Chord Elevations	65.3 m	
Inverts	56.86 m	
Effective Flow Area	743.7 m^2	
Manning's "n" Values	0.046	
Additional Details	Structural steel with 4 piers	

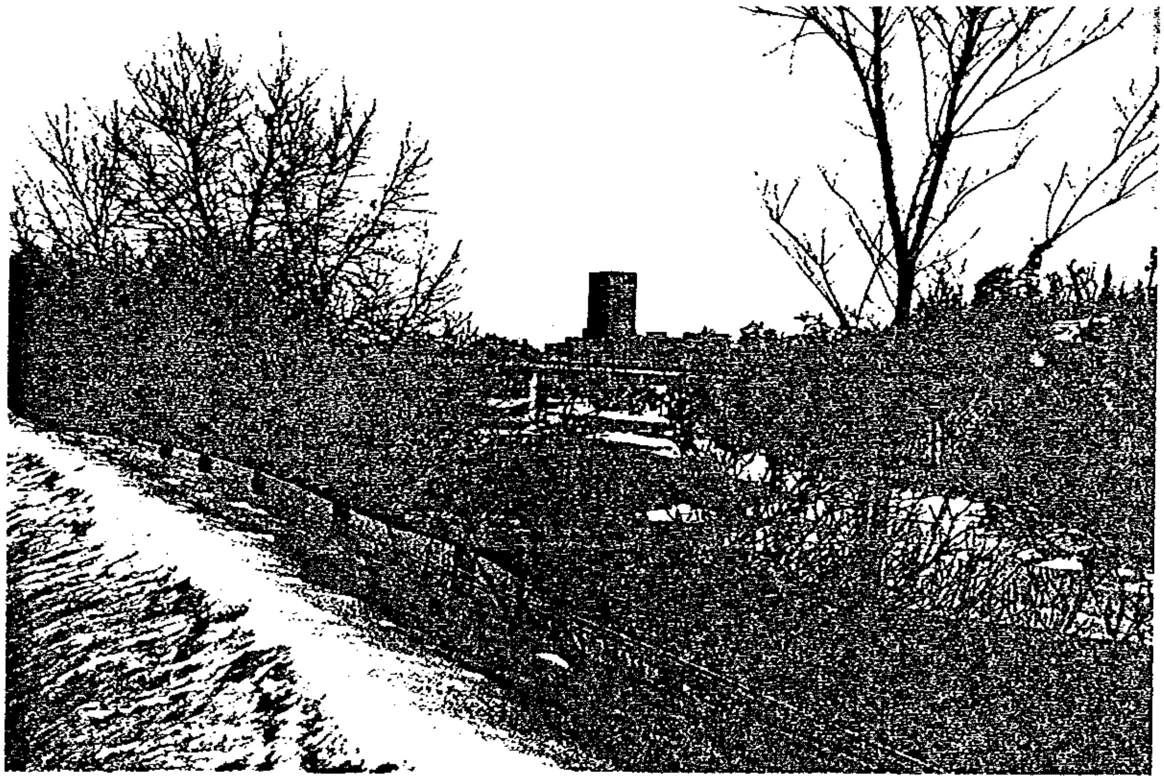
BRIDGE DATA

Watercourse: RIDEAU RIVER

Location: HERON ROAD BRIDGE

Map Sheet No.: 16

Cross-Section No.: 10878



VIEW OF UPSTREAM FACE

SPECIFICATIONS	UPSTREAM	DOWNSTREAM
Length of Structure	44.0 m	
Top of Road	74.80 m	
Span	132.0 m	
Low Chord Elevations	77.11 m	
Inverts	56.35 m	56.30 m
Effective Flow Area	1963.0 m^2	
Manning's "n" Values	0.045	
Additional Details	Concrete beam and girder with 6 concrete piers	

COMPUTER PRINTOUTS

Computer Printout #1

Flood Frequency Analyses of Maximum Instantaneous Flows of Rideau River at Ottawa for 1947; 1949-82 period. (Table II)

02LA004 RIDEAU RIVER AT OTTAWA MAX INSTANTANEOUS 2

YEAR	DATA	ORDERED	RANK	PROB.	RET. PERIOD
1982	15360.	21080.	1	.028	36.000
1981	15750.	20410.	2	.056	18.000
1980	14870.	20010.	3	.083	12.000
1979	14940.	19780.	4	.111	9.000
1978	18610.	18610.	5	.139	7.200
1977	16700.	18330.	6	.167	6.000
1976	21080.	18120.	7	.194	5.143
1975	14590.	16700.	8	.222	4.500
1974	14730.	16620.	9	.250	4.000
1973	16390.	16440.	10	.278	3.600
1972	20410.	16440.	11	.306	3.273
1971	18120.	16390.	12	.333	3.000
1970	16440.	15750.	13	.361	2.769
1969	12200.	15580.	14	.389	2.571
1968	14020.	15360.	15	.417	2.400
1967	11570.	15360.	16	.444	2.250
1966	8000.	15060.	17	.472	2.118
1965	5430.	14940.	18	.500	2.000
1964	4050.	14870.	19	.528	1.895
1963	16440.	14730.	20	.556	1.800
1962	12010.	14590.	21	.583	1.714
1961	7180.	14090.	22	.611	1.636
1960	19780.	14090.	23	.639	1.565
1959	15360.	14020.	24	.667	1.500
1958	11380.	13050.	25	.694	1.440
1957	4950.	12310.	26	.722	1.385
1956	13050.	12200.	27	.750	1.333
1955	18330.	12010.	28	.778	1.286
1954	15060.	11570.	29	.806	1.241
1953	12310.	11380.	30	.833	1.200
1952	14090.	8000.	31	.861	1.161
1951	15580.	7180.	32	.889	1.125
1950	16620.	5430.	33	.917	1.091
1949	14090.	4950.	34	.944	1.059
1947	20010.	4050.	35	.972	1.029

02LA004 RIDEAU RIVER AT OTTAWA MAX INSTANTANEOUS 2

SAMPLE STATISTICS

MEAN = 14271. S.D. = 4271.7 C.S. = -.8288 C.K. = 3.7305

SAMPLE STATISTICS (LOGS)

1/2

MEAN = 9.5040 S.D. = .3959 C.S. = -1.7068 C.K. = 5.8586

SAMPLE MIN = 4050. SAMPLE MAX = 21060. N = 35

PARAMETERS FOR GUMBEL I A = .000212 U = 12017.

PARAMETERS FOR LOGNORMAL M = 9.5040 S = .3959

NO MAXIMUM LIKELIHOOD SOLUTION FOR THREE PARAMETER LOGNORMAL

NO MOMENT SOLUTION FOR THREE PARAMETER LOGNORMAL

PARAMETERS FOR LOG PEARSON III BY MOMENTS A = -.3378 B = .1373E+01 LOG(M) = 9.9678 M = .2133E+05

PARAMETERS FOR LOG PEARSON III BY MAXIMUM LIKELIHOOD A = -.3124 B = .1492E+01 LOG(M) = 9.9670 M = .2131E+05

DISTRIBUTION STATISTICS MEAN = 9.5040 S.D. = .3803 C.S. =-1.6430

	GUMBEL I		LOGNORMAL		THREE PARAMETER LOGNORMAL		LOG PEARSON III MAX. LIKELIHOOD		LOG PEARSON III MOMENTS	
RETURN PERIOD	FLOOD ESTIMATE	ST. ERROR PERCENT	FLOOD ESTIMATE	ST. ERROR PERCENT	FLOOD ESTIMATE	ST. ERROR PERCENT	FLOOD ESTIMATE	ST. ERROR PERCENT	FLOOD ESTIMATE	ST. ERROR PERCENT
1.005	4140.	26.90	4840.	13.90	0.	0.00	0.	0.00	0.	0.00
1.050	6760.	13.40	6990.	10.30	0.	0.00	0.	0.00	0.	0.00
1.250	9770.	8.17	9610.	7.79	0.	0.00	0.	0.00	0.	0.00
2.000	13700.	6.82	13400.	6.69	0.	0.00	0.	0.00	0.	0.00
5.000	19100.	7.53	18700.	7.79	0.	0.00	0.	0.00	0.	0.00
10.000	22600.	8.15	22300.	9.03	0.	0.00	0.	0.00	0.	0.00
20.000	26000.	8.67	25700.	10.30	0.	0.00	0.	0.00	0.	0.00
50.000	30400.	9.22	30200.	11.60	0.	0.00	0.	0.00	0.	0.00
100.000	33700.	9.56	33700.	12.90	0.	0.00	0.	0.00	0.	0.00
200.000	37000.	9.85	37200.	13.90	0.	0.00	0.	0.00	0.	0.00
500.000	41400.	10.20	41900.	15.20	0.	0.00	0.	0.00	0.	0.00

Computer Printout #2

Flood Frequency Analyses of Maximum Instantaneous Flood of Rideau River at Ottawa with those since 1976 increased by seven percent for 1947, 1949-1982 period with five lowest flood excluded. (Table III)

02LA004 RIDEAU RIVER AT OTTAWA MAX INSTANTANEOUS R

YEAR	DATA	ORDERED	RANK	PROB.	RET. PERIOD
1982	16440.	21080.	1	.032	31.000
1981	16850.	20410.	2	.065	15.500
1980	15910.	20010.	3	.097	10.333
1978	19910.	19910.	4	.129	7.750
1977	17870.	19780.	5	.161	6.200
1976	21080.	19330.	6	.194	5.167
1975	14590.	18120.	7	.226	4.429
1974	14730.	17870.	8	.258	3.875
1974	14770.	16850.	9	.290	3.444
1973	16390.	16620.	10	.323	3.100
1972	20410.	16440.	11	.355	2.818
1971	18120.	16440.	12	.387	2.583
1970	16440.	16440.	13	.419	2.385
1969	[illegible]	16390.	14	.452	2.214
1968	14020.	15910.	15	.484	2.067
1967	11570.	15580.	16	.516	1.938
1963	16440.	15360.	17	.548	1.824
1962	12010.	15060.	18	.581	1.722
1960	19790.	14730.	19	.613	1.632
1959	15360.	14730.	20	.645	1.550
1958	11390.	14590.	21	.677	1.476
1956	13050.	14090.	22	.710	1.409
1955	18330.	14090.	23	.742	1.348
1954	15050.	14020.	24	.774	1.292
1953	12310.	13050.	25	.806	1.240
1952	14070.	12310.	26	.839	1.192
1951	15580.	12200.	27	.871	1.148
1950	16620.	12010.	28	.903	1.107
1949	14090.	11570.	29	.935	1.069
1947	20010.	11380.	30	.968	1.033

02LA004 RIDEAU RIVER AT OTTAWA MAX INSTANTANEOUS R

SAMPLE STATISTICS
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15 246 448.7

MEAN = 16446. S.D. = 2735.5 C.S. = .2093 C.K. = 2.6229

SAMPLE STATISTICS (LOGS)

MEAN = 9.6551 S.D. = .1741 C.S. = -.1076 C.K. = 2.6052

SAMPLE MIN = 11380. SAMPLE MAX = 21080. N = 30

PARAMETERS FOR GUMBEL I A = .000418 U = 14530.

PARAMETERS FOR LOGNORMAL M = 9.6561 S = .1741

STATISTICS OF LOG(Y-A)

MEAN = 9.8992 S.D. = .1365 C.S. = -.0389 C.K. = 2.5907

PARAMETERS FOR THREE PARAMETER LOGNORMAL A = -4248. M = 9.8992 S = .1365

PARAMETERS FOR LOG PEARSON III BY MOMENTS A = -.0094 B = .3456E+02 LOG(M) = 10.8926 M = .7974E+[illegible]

PARAMETERS FOR LOG PEARSON III BY MAXIMUM LIKELIHOOD A = -.0228 B = .5688E+02 LOG(M) = 10.9917 M = [illegible]

DISTRIBUTION STATISTICS MEAN = 9.6561 S.D. = .1718 C.S. = -.2652

	GUMBEL I		LOGNORMAL		THREE PARAMETER LOGNORMAL		LOG PEARSON III MAX. LIKELIHOOD		LOG PEARSON III MOMENTS	
RETURN PERIOD	FLOOD ESTIMATE	ST. ERROR PERCENT	FLOOD ESTIMATE	ST. ERROR PERCENT	FLOOD ESTIMATE	ST. ERROR PERCENT	FLOOD ESTIMATE	ST. ERROR PERCENT	FLOOD ESTIMATE	ST. ERROR PERCENT
1.005	[illegible]	5.78	9970.	6.60	9760.	11.50	9610.	11.30	9800.	9.22
1.050	11900.	4.19	11700.	4.68	11700.	5.70	11600.	6.05	11700.	5.04
1.250	13400.	3.26	13500.	3.70	13500.	3.84	13600.	3.92	13500.	3.61
2.000	[illegible]	3.33	[illegible]	3.18	15700.	3.42	15700.	3.36	15700.	3.44
5.000	18100.	4.34	18100.	3.70	18100.	3.59	18100.	3.40	18100.	3.83
10.000	19900.	5.07	19500.	4.29	19500.	4.19	19400.	3.79	19500.	4.58
20.000	21600.	5.71	20800.	4.66	20700.	5.06	20400.	4.49	20700.	5.68
50.000	23900.	6.44	22300.	5.60	22100.	6.44	21700.	5.72	22100.	7.48
100.000	[illegible]	6.91	[illegible]	6.12	23100.	7.55	22500.	6.61	23100.	9.00
200.000	27200.	7.34	24500.	6.60	24100.	8.71	23300.	7.97	[illegible]	[illegible]
500.000	29400.	7.87	[illegible]	7.21	25300.	10.30	24300.	9.58	25200.	12.80