REPORT ON

RIDEAU RIVER FLOODLINE MAPPING (SMITHS FALLS TO POONAMALIE)

FOR THE

RIDEAU VALLEY CONSERVATION AUTHORITY

JANUARY, 1979

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26 January 1979

Mr. O. Stirajs
Resources Manager
Rideau Valley Conservation Authority
Box 599
Mill Street
Manotick, Ontario

Rideau River Floodline Mapping Study (Smiths Falls to Poonamalie)

Dear Mr. Stirajs:

By means of an engineering agreement dated October 18, 1977, our firm was authorized to undertake a flood and fill line study of the Rideau River from Old Slys Dam to Poonamalie Dam.

The appropriate studies have been completed and the results of our investigation are presented in the enclosed report and on the accompanying fill and flood line mapping.

During the project, numerous discussions concerning flood conditions and the operation of river control structures were held with members of the Rideau Canal, Parks Canada and the Smiths Falls Water Commission. Their assistance was most helpful and contributed to the successful conclusion of the study.

Yours very truly,

JAMES F. MacLAREN LIMITED

R B Wigle

R. B. Wigle, P. Eng.
Project Manager

REGISTERED PROFESSIONAL ENGINEER
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1.0 INTRODUCTION

The Rideau River drains an area of 1,500 square miles at its confluence with the Ottawa River. Flooding problems along the waterway are documented in the Rideau Valley Conservation Report (1) which also describes the watershed's history and natural resources. Historically, peak flows have occurred as a result of spring snowmelt and have often resulted in notable flood damage to waterfront properties. As part of the Conservation Authority's regulatory program, floodplain mapping was recently completed from Kars to Smiths Falls (2). This mapping is extended upstream during the present study for a distance of 4.3 miles through the Town of Smiths Falls to the Poonamalie Dam.

Within the scope of the investigation, the major tasks to be completed were as follows:

1. Collect and review all existing reports, hydrological data, drawings, maps, and other information related to this study.

2. Carry out a detailed field survey of the study area with particular attention to the existing structures on the watercourse, such as bridges, dams, locks and weirs.

3. Establish river cross-sections and downstream control elevations, and carry out backwater computations using the 1 in 100 year flow at Poonamalie as established in

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1) Rideau Valley Conservation Report, Department of Energy, Mines and Resources Management, Toronto, 1968.

2) James F. MacLaren Limited, "Report on Rideau River Floodline Mapping (Smith Falls to Kars)", for the Rideau Valley Conservation Authority, June 1976.

the previous report, "Rideau River Floodline Mapping (Smiths Falls to Kars)".

4. Produce controlled photomosaic sheets of the study reach at a scale of 1"=400', showing the cross-section locations, flood elevations, flood and fill lines, and spot elevations on selected structures and river locations.

5. Produce a fairdrawn topographic map at a scale of 1"=200' and a contour interval of 5 feet from Old Slys Lock upstream to the C.N.R. bridge.

6. Prepare a draft report summarizing the findings and upon written approval of the Rideau Valley Conservation Authority, submit a final report together with drawings and computer data.

Details of the study methodology are presented in Appendix 1. The following sections summarize the main findings of the study.

2.0 DESIGN FLOW

The 100 year design flow for the Rideau River between the Old Slys Lock and the Poonamalie Dam was estimated during the previously noted study to be 5,050 cfs. This value is based upon a frequency analysis of daily discharge records collected at the latter control structure from 1944 to 1975. While flows from the 497 square mile tributary area are regulated to ensure an adequate water supply during the summer navigation season, high discharges do occur through the Town, particularly in the spring. As recently as April 22, 1972, the estimated flow at Old Slys Dam was 3,929 cfs and on the day of reported flooding of the Water Commission building, April 7, 1974, releases at the Poonamalie Dam reached 2,819 cfs.

3.0 RIVER ANALYSIS MODEL

3.1 Model Preparation

Photogrammetric river cross-sections developed by Kenting Earth Sciences Limited were used in conjunction with bathymetric information obtained from the Canadian Hydrographic Service in defining the shape of the river channel and overbank areas. Subsequently, a field investigation was carried out to augment the existing information on river characteristics and structure dimensions. Photographs of all structures and sketches showing dimensions are presented in the accompanying Photographic Survey.

The water levels within the study reach are controlled by a number of dams and locks for the purpose of summer navigation. During periods of high flow, the current operating procedure for the structures is to remove all stop logs from the dams. Based on this operation, and on the physical dimensions of the structures provided by the Rideau Canal Office, design headwater levels were computed at each of the following:

- Old Slys Dam
- Godlie Dam (Water Commission Dam)
- Smiths Falls Combined Dam
- Goulds Dam
- Dam 17 (immediately upstream of Abbott Street)

Preliminary backwater analysis indicated that the 100 year design flood levels at Goulds Dam and at Dam 17 are influenced by downstream water levels, therefore, neither dam acts as a control section. Accordingly, the study reach was divided into three segments for backwater computations:

- Old Slys Dam to Godlie Dam
- Godlie Dam to Smiths Falls Combined Dam
- Smiths Falls Combined Dam to Poonamalie Dam

Stage-discharge relationships and a summary of physical dimensions of the three control structures are presented in Tables A1 and A2 in Appendix 1. The estimated 100 year headwater elevation at each of the three control structures, given below, was subsequently used to initiate backwater calculations for the corresponding river reaches:

Old Slys Dam	367.7 ft.
Godlie Dam	384.1 ft.
Smiths Falls Combined Dam	390.5 ft.

At present, the computed 100 year design flood level at the Old Slys Dam, 367.7 feet, would be one foot above the deck of the dam but would be contained by the adjacent wing walls. During the study, Parks Canada personnel strongly indicated that emergency bypassing of flood discharges at the accompanying lock structure for purposes of lowering the headwater should not be viewed as a feasible alternative due to the potential damage which would be incurred at the lock. During the previous study, Rideau River Floodline Mapping (Smiths Falls to Kars), it was understood that bypassing of excess flow through Old Slys Lock would be permitted in order to prevent the submergence of Old Slys Dam. The resultant lower headwater elevation at the Dam is now superceded by the operation and flood elevations outlined in this report.

3.2 River Model Calibration

Water level observations recorded within the downstream reach of the study area provided an opportunity to calibrate the backwater model before proceeding with the 100 year stage computation.

The change in the water profile within the channel at the Old Slys Bridge is particularly noticeable during high flow conditions. Since water levels as far upstream as Beckwith Street are largely determined by this phenomenon, special emphasis was placed on calibrating the backwater model at this structure. Simultaneous water levels at the Old Slys Dam and at Old Slys Lock, the latter being representative of stages upstream of the bridge, were obtained from the Rideau Canal Office together with accompanying flow rates for the periods of high discharge in April, 1972 and 1974. During the periods of investigation, operation of the dam was similar to the recommended procedure for the 100 year event in that all stop logs were removed. After adjustment of backwater model parameters, predicted and observed water levels immediately upstream of Old Slys Bridge agreed to within 0.2 feet.

During the flood of April, 1974, an approximate depth of flooding of 2 feet above the ground floor at the Smiths Falls Water Commission building was observed. This stage corresponds to an elevation of 367.5 feet. Subsequent calibration of the backwater model for a flood discharge of 2,819 cfs provided a water surface elevation of 367.5 feet at this location.

A comparison of computed design flood levels with the maximum recorded water levels is given in Table A3.

4.0 FLOOD HAZARD AREAS

Floodlines corresponding to the 100 year flow are presented on the two accompanying 1"=400' photomosaic sheets, as well as the 1"=200' topographic map of Smiths Falls. River discharges passing through Smiths Falls are regulated to a great degree and the design flow, as discussed in the preceding section, reflects the past operation of the Poonamalie Dam. During periods of high runoff, discharges at Poonamalie are optimized in order to minimize flooding both in the upstream basin and in the Rideau River watershed below the control structure. Following a recent study (1) of the Canal operation, a revised operational procedure has been adopted which will reportedly reduce the incidence of spring flooding in the upper portion of the Canal system. However, until the effectiveness of the operation in reducing severe flood peaks is established, operations staff, Parks Canada, have indicated that historic records should still be regarded as representative of the flood hazard.

All structures within the study reach have the hydraulic capacity to accommodate the 100 year design flow, with the exception of Old Slys Dam as discussed on Page 5. However, the channel capacity is exceeded for the design flow at both Goulds Dam and Dam 17 and these structures would be outflanked by floodwaters during this severe event.

In general, the channel capacity of the Rideau River between Old Slys Dam and the Canadian National Railway bridge is sufficient to convey the 100 year design flow with minimal overbank flooding. One major exception would occur at the Smiths Falls Water Commission building in which the esti-

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1) Study of the Operation of the Rideau Cataraqui System; prepared for Rideau Canal Parks Canada, Department of Indian and Northern Affairs, Acres Consulting Engineers Limited, March 1977.

mated 100 year stage, 371.0 feet, would inundate the ground floor and pumping equipment to a depth of 5.5 feet.

Due to the waterway constriction upstream of the Beckwith Street bridge, major channelization from the Water Commission building to a point downstream of the bridge would be required to significantly lower flood levels at the Water Commission building. In the event that headwater elevations were lowered by reconstruction of the Old Slys Dam, backwater investigations indicated that the effective reduction of the 100 year flood level at the Water Commission building would be limited to two feet unless accompanied by the foregoing channelization near the Beckwith Street bridge.

Due to the relative cost of channelization works, especially involving rock excavation, structural floodproofing at the Water Commission building should be given serious consideration. From discussions during the study, it is understood that the floor of the main pump room is constructed to a large extent on a rubble foundation; therefore, floodproofing measures may be expected to take the form of a perimeter wall.

Referring to the photos of the Water Commission building in the accompanying Photographic Survey, the primary section requiring floodproofing appears to be the masonry wall from Godlie Dam to the concrete wall housing the intake pipe. Since bedrock is exposed on the other side of the river, it is likely that bedrock lies near the surface at the Water Commission building. This section of wall could be floodproofed by excavating to bedrock, then constructing a watertight concrete wall to elevation 373 feet; watertight seals would be required at bedrock, Godlie Dam and the concrete wall. Assuming bedrock to be close to the surface, the estimated construction cost for a 75 ft. long x 10 ft. high x 1.5 ft. wide concrete wall is $30,000 (1).

It should be noted that further study of this building would be required at the design stage of floodproofing. Such an investigation must determine the depth to solid bedrock and whether or not other sections of the building require floodproofing; floodwaters might permeate the concrete wall housing the intake pipe, or the masonry wall on the downstream side of the building.

Further upstream, between Goulds Dam and Abbott Street, floodwaters at a depth of 1 or 2 feet would impinge on the Smiths Falls Hydro building during the design flood. Since the Abbott Street bridge is not overtopped, this building could be protected from flooding by raising the retaining wall from Abbott Street to Goulds Dam on the river's north side.

The existing wall from Abbott Street to Goulds Dam appears to be in very poor condition (see photo); therefore, a steel sheet pile wall with concrete cap is recommended. Assuming that there is no shale on the channel bottom, and that sheeting can be driven at least 5 feet into the existing bed, the estimated cost to provide protection to elevation 401 feet along this 150 ft. length is $35,000.

As shown on the accompanying floodline drawings, the 100 year floodwaters at a depth of approximately 0.65 feet would spill from the channel in a southerly direction between the C.N.R. bridge and the Smiths Falls Detached Lock. It is

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(1) All estimated costs given in this report are conceptual and preliminary. They exclude expenses for engineering, contract administration, resident services, escalation, winter work and land and legal costs.

estimated that spill at this location begins when the river flow exceeds 4,300 cfs. Due to the complex pattern of overland flow, it is difficult to define the extent of flooding from this spill. However, it appears that shallow flooding would occur over a considerable area of existing residential development. This spill could be prevented by raising the retaining wall on the south side of the entrance to the lock, from the C.N.R. bridge to the lock. The estimated cost for a 440 ft. x 2 ft. x 1 ft. concrete cap, dowelled to the existing wall and providing protection to elevation 403 feet is $20,000.

Upstream of the C.N.R. bridge, the land is flat and marshy, and the floodplain reaches a maximum width of 4,000 feet. Most of this land is undeveloped at the present time; however a limited area of existing development on the river's north side, near Poonamalie Dam, would experience flooding to a depth of 1 or 2 feet. Due to the shallow depth and relatively low flow velocities which could be expected, it is suggested that floodproofing of individual structures should be considered.

5.0 FILL LINES

Fill and construction limits, as presented on the accompanying topographic and photomosaic drawings identify areas in which special consideration is required before proceeding with development or the dumping of fill. Since erosion potential is not a prime environmental hazard in the study reach, the reservation of sufficient open space to permit the passage of the 100 year flood is considered a reasonable objective. In order to prevent the inundation of developed areas by flood waters, the fill lines in these areas are positioned at a distance of fifty feet from the 100 year floodline. In view of the potential spill of floodwaters from the river channel during the 100 year event, the Conservation Authority requested that the undeveloped area bounded by the C.N.R. embankment, Detached Lock, Abbott Street and Lombard Street should also be included within the fill line.

APPENDIX 1

HYDRAULIC METHODOLOGY

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

The Bernoulli equation takes the following form:

$$\frac{P}{\gamma} + z + \frac{V^2}{2g} = H \qquad 6.1$$

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

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

z = height above datum

H = total energy head above the datum

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

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

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

$$V = \frac{1.49 \; R^{2/3} \; S^{1/2}}{n} \qquad 6.2$$

Where S_f = friction slope

R = hydraulic radius (feet)

n = Manning's resistance coefficient

V = velocity (feet per second)

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

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

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

$$Q = A \; [\frac{2gh}{K}]^{1/2} \qquad 6.3$$

Where H = difference between the energy gradient elevation upstream and tailwater elevation downstream

A = area of the orifice

g = gravitational acceleration

Q = total orifice flow

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

$$K = K_e + K_{ex} + K_b + K_f \tag{6.4}$$

Where K_e = typical entrance loss coefficient is 0.1

K_{ex} = 1.0

$$K_f = \frac{29.1\ n^2 L}{R^{4/3}} \tag{6.5}$$

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

R = hydraulic radius (ft.)

L = culvert length (ft.)

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

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

$$Q = C\ L\ H^{3/2} \tag{6.6}$$

Where C = coefficient of discharge

L = effective length of the weir

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

Q = total flow over the weir

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

<u>APPENDIX 2</u>

TABLE A1

Stage - Discharge Relations for the 3 Control Structures in the Study Reach, Assuming All Stop Logs Out

OLD SLYS DAM		GODLIE DAM		SMITHS FALLS COMBINED DAM	
Water Level (ft.)	Discharge (cfs)	Water Level (ft.)	Discharge (cfs)	Water Level (ft.)	Discharge (cfs)
357.6	0	374.5	0	381.3	0
359.0	330	377.0	550	382.5	110
362.0	1970	379.5	1560	383.2	250
365.9	4780	380.0	1840	385.0	1000
367.6	5000	382.0	3240	387.0	2210
367.8	5220	384.0	4990	390.0	4580
		384.5	5470	390.5	5070
				392.3	7240

TABLE A2

Summary of Physical Dimensions of Control Structures

Location	Flow Capacity* (cfs)	Length of Bay (ft.)	Crest Elev. of Dam (ft.)	WEIR 1: 1 Number of Logs	WEIR 1: 2 Clear Width (ft.)	WEIR 1: 3 Height of Log (inch)	WEIR 1: 4 Sill Elev. (ft.)	WEIR 2: 1	WEIR 2: 2	WEIR 2: 3	WEIR 2: 4	WEIR 3: 1	WEIR 3: 2	WEIR 3: 3	WEIR 3: 4	WEIR 4: 1	WEIR 4: 2	WEIR 4: 3	WEIR 4: 4	WEIR 5: 1	WEIR 5: 2	WEIR 5: 3	WEIR 5: 4
Old Slys Dam	4800	-	-	7	20	12	357.6	7	20	12	357.6	7	20	12	357.6								
Godlie Dam	5500	-	-	9	14	12	374.5	9	14	12	374.5	9	14	12	374.5	4	14	12	379.5	4	14	12	379.5
Smiths Falls Combined Dam	7200	60**	390.4	9	25	12	381.3	8	19	12	382.5	7	20	12	383.2								

* Flow capacity computed assuming all logs out, water level at obvert of structure opening.

** Stage - discharge relation $Q = 2.95\ b\ H^{1.5}$ where Q = discharge (cfs), b = weir width or length of bay (ft.), H = head above crest of weir (ft.). All other weirs, $Q = 3.33\ b\ H^{1.5}$.

TABLE A3

Comparison of Computed Flood Levels with Maximum Recorded Water Levels (Geodetic Survey of Canada Datum)

	MAXIMUM RECORDED WATER LEVEL AT		COMPUTED 100 YEAR FLOOD LEVEL AT	
Location	Upstream Sill (ft.)	Downstream Sill (ft.)	Upstream Sill (ft.)	Downstream Sill (ft.)
Old Slys Lock	367.3 (1926)*	349.9 (1926)	368.4	-
Combined Locks	390.9 (1942)	367.0 (1919)	390.8	368.6
Detached Lock	401.4 (1926)	391.3 (1942)	401.6	391.0
Poonamolie Lock	407.7 (1972)	401.1 (1919)	-	402.5

* Year recorded.