# Summary Report – June 2005

# Jock River Flood Risk Mapping (within the City of Ottawa)

Prepared for: **Rideau Valley Conservation Authority**
Prepared by: **PSR Group Ltd.** in association with **JF Sabourin and Associates Inc**

## Acknowledgements

This report was based on two technical documents prepared by staff of PSR Group Ltd, JF Sabourin and Associates Inc. and David McManus Engineering Ltd..

In particular:

- field data, statistical analysis and hydrologic modeling were completed by Josee Forget, Philippe Perron, P.Eng. and JF Sabourin, P.Eng. of JF Sabourin and Associates

- rainfall data and report preparation was provided by Paul Frigon, P.Eng. and Philippa Winters of PSR Group Ltd.

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- Report preparation was undertaken by Philippe Perron of JF Sabourin and Associates, and Paul Frigon, P.Eng. and Philippa Winters of PSR Group Ltd.

- Assistance in obtaining historical water levels was provided by Rideau Valley Conservation Authority staff Patrick Larson and Bruce Reid, P.Eng.

- The development of new base mapping was completed by Stephen Perkins with the City of Ottawa and Rideau Valley Conservation Authority staff Ewan Hardie. Flood Risk Maps were assembled by staff of David McManus Engineering under the supervision of their senior staff Sean Czaharynski, P.Eng. and John Burns, CET

- Project review and co-ordination was provided by Bruce Reid, P.Eng. and Ferdous Ahmed, P.Eng. from the Rideau Valley Conservation Authority

**Table of Contents** **Page**



**List of Figures**



**List of Tables**

## 1.0 Introduction

The Rideau Valley Conservation Authority (RVCA) requires new flood risk mapping for the Jock River and its major tributaries within the City of Ottawa (Monahan Drain, Smith Creek, Leamy Creek, Flowing Creek and Van Gaal Drain).

The regulatory flood level, used for flood risk mapping within the Rideau River watershed, is defined as the 100 Year flood level: the water level associated with the river discharge that has a 1% probability of being equaled or exceeded each year, or occurs, on average, once every 100 years. Since flood risk mapping requires the development of hydraulic simulation models to define the 100 year water level, reliable flow estimates must be developed as major inputs to the hydraulic model.

Two technical reports have been prepared that form the basis of the flood risk mapping for the Jock River within the City of Ottawa: the first is the **" Hydrology Report – July 2004 – Jock River Flood Risk Mapping (within the City of Ottawa) "** that provides appropriate flow estimates and the other the **" Hydraulics Report – November 2004 – Jock River Flood Risk Mapping (within the City of Ottawa) "** that determines flood levels based on the flow estimates. The report in hand is a summary of these two reports and the Flood Risk Mapping process for the Jock River

The Jock River is a tributary of the Rideau River with the subwatershed having mainly rural land use; river slopes less than 0.5%; and no flow regulation. Its 556km$^2$ drainage area is illustrated in Figure 1 and forms roughly 15% of the Rideau River watershed. The Jock River Watershed Plan – Background Report (JL Richards, 2000) delineated four distinct reaches of the watershed as characterised in Table 1. In this flood risk mapping study, reaches three and four are being addressed as the **lower reach**, between Richmond and the Rideau River, and reach two is being addressed as the **middle reach**, upstream of Richmond, between the Richmond Fen and Ashton.

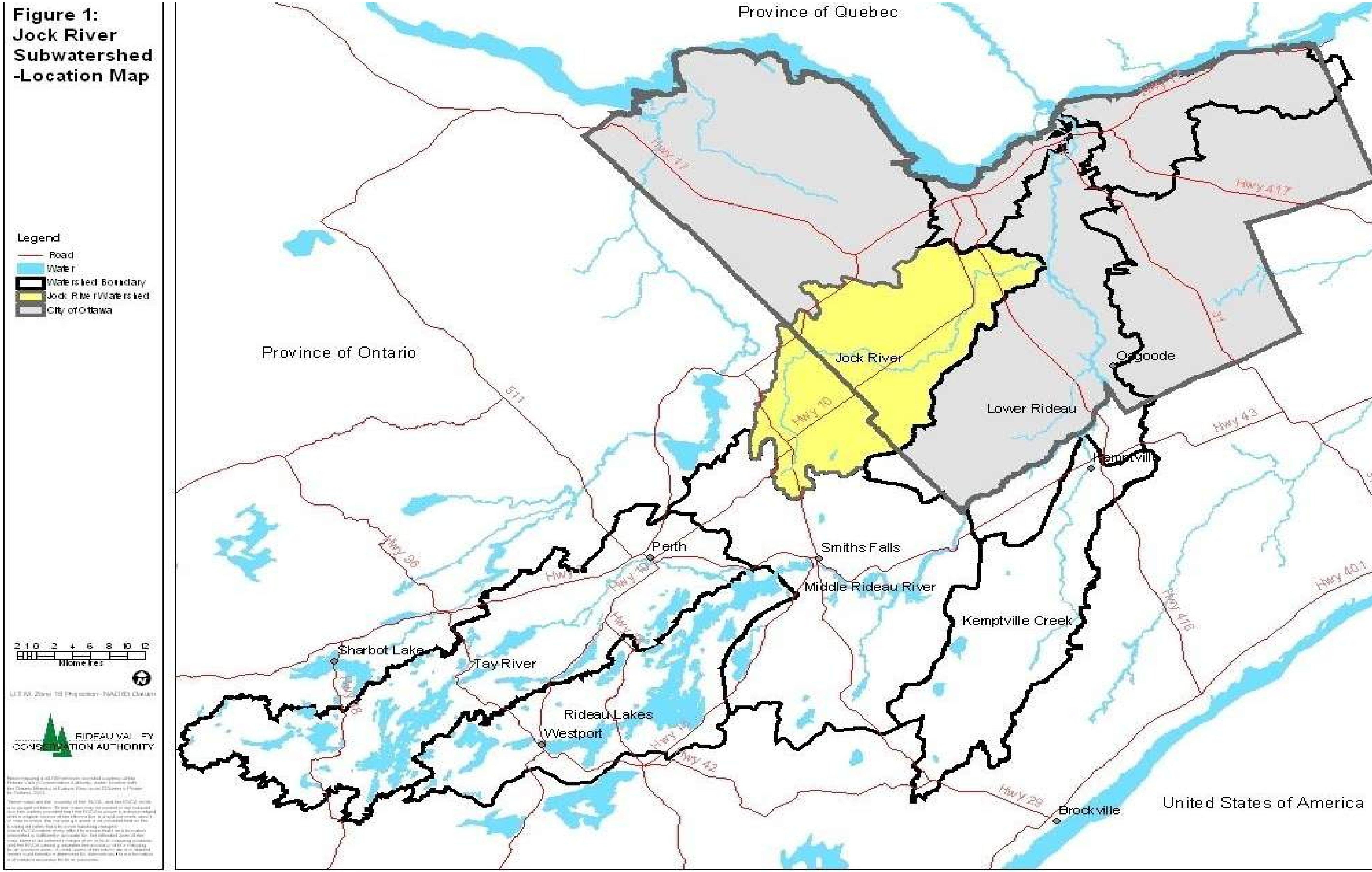
## 2.0 Flow Estimates

Flow estimates can be provided by:

1. single station frequency analysis (SSFA) of observed or simulated peak flows. ie. a statistical analysis of maximum instantaneous peak flows
2. prorating SSFA flows, based on area, for points of interest other than the single station location
3. hydrologic modeling using simulated events as inputs: either snowmelt+rainfall (spring) or rainfall (summer).

In this project it is proposed to use flows derived from SSFA, where applicable, and hydrologic modeling where SSFA would not apply.

# Figure 1: Jock River Subwatershed -Location Map



For the **lower reach**, the SSFA, derived from 34 years of record at the WSC gauge at Moodie Drive provides a good estimate of the 100 Year flow. All annual maximum peak flows have occurred during the Spring Runoff and proration techniques can be used to determine 100 year flow elsewhere in this reach.

Flows in the lower reach reflect the influence of the Richmond Fen while flows in the **middle reach** do not. Because of this influence, it cannot be assumed that the flows derived from SSFA can be prorated to the middle reach; they might provide a lower estimate of the 100 year flow than would actually occur. For the middle reach, a calibrated and validated hydrologic model, with spring snowmelt+rainfall events as an input, should provide the best estimate of the 100 Year flow.

It is anticipated that maximum flood levels on **a tributary** will be influenced by flood levels on the Jock River: whether this occurs during a Spring or Summer event is not known. The maximum 100 Year flood level for a tributary would be based on hydraulic analysis that would consider flows on the Jock River and the tributary that, together, have a combined probability of once in 100 years. A calibrated and validated hydrologic model, for both Spring and Summer events, assisted in providing flows for the required hydraulic analysis.

It is important to note that the calibration/validation effort concentrated on the simulation of high flows for the purpose of flood risk mapping: the estimates of more frequent flows, such as the 2 year and 5 year flows, should be used with caution.

**Table 1: Jock River - Watershed Characteristics**
(from the “Jock River Watershed Plan –Surface Water Quantity Component - 1996”; with updated drainage areas - 2004)*

| **Characteristic⇒** | **Area* ($km^2$)** | | **Length (km)** | **Slope (m/m)** | **Landuse (%)** | | | | **Predominant Soils** |
|---|---|---|---|---|---|---|---|---|---|
| **Reach ⇓** | **1996** | **2004** | | | **Forest** | **Farm** | **Wetland** | **Urban** | |
| 1. Headwaters to Ashton | 113 | 95 | 26 | .001 | 62 | 21 | 17 | 0 | sandy loam |
| 2. Ashton to Richmond Fen | 227 | 221 | 19 | .002 | 35 | 48 | 16 | 1 | sandy loam |
| 3. Richmond Fen to Flowing Creek | 126 | 138 | 10 | .002 | 36 | 39 | 23 | 2 | loam |
| 4. Flowing Creek to Rideau River | 88 | 102 | 17 | .002 | 20.5 | 78 | 0.2 | 1.3 | clay loam |
| **Total** | **554** | **556** | | | | | | | |

***** drainage areas were updated for the current study and may differ from those previously recorded; this includes differences in interpretation of Reach locations.

## 2.1 Single Station Frequency Analysis (SSFA)

Since 1970, flows from the Jock River subwatershed have been monitored at the Water Survey of Canada (WSC) streamflow gauge (02LA007) at Moodie Drive: the drainage area above this site is approximately 95% of the subwatershed. The RVCA has installed a similar streamflow gauge, in 2003, on the Jock River at Franktown Road that represents flow from approximately 32% of the subwatershed. 34 years of annual maximum instantaneous peak flows from the WSC gauge at Moodie Drive were used in the statistical analysis to determine the 100 Year flow: these are illustrated in Figure 2.

Consolidated Frequency Analysis (CFA) software (version 3.1), developed by Environment Canada, was used to undertake the statistical analysis.

Fundamental tests for independence, trend, homogeneity and randomness confirmed the quality and usefulness of the data. After a review of the SSFA results, including examination of data "fit" to four different probability distributions, and comparison with the results from other flow estimation techniques (including regional frequency analysis, index flood and watershed classification methods), the Log Pearson Type III (LP3) distribution was selected as the best distribution to provide an estimate of the 100 Year flow at Moodie Drive. The 100 year peak flow at Moodie Drive is estimated to be 196 $m^3$/s.

The SSFA results are presented in Table 2. Prorated flows were determined using the commonly employed relationship $Q1/Q2=(A1/A2)^{0.75}$ (MTO Drainage Manual: pg H4-7) where Q1 and Q2 are flows and A1 and A2 are their respective drainage areas.

## 2.2 Hydrologic Model

Two distinct hydrologic models have been developed for the Jock River subwatershed: one reflects Spring conditions and the other Summer conditions. They vary with regards to essential hydrologic parameters such as time to peak, soil infiltration rates and channel routing characteristics. Each model was calibrated based on observed runoff in 2003 at the Moodie Drive and Franktown streamflow gauges: the Spring model was validated using observed hydrographs from 1978, 1993, 1997 and 1998 at Moodie Drive.

Both models use Return Period design events to provide peak flow estimates at various points of interest in the subwatershed: for Spring, a snowmelt+rainfall event was developed; for Summer, the design event was a summer storm.

SSFA results from observed flows at Moodie Drive were used for additional validation of both the Spring and Summer models: design event peak flows, for various flood frequencies, were compared to SSFA of observed peak flows. As well, SSFA of peaks derived from continuous simulation of 38 years of meteorological record, on an annual basis, were

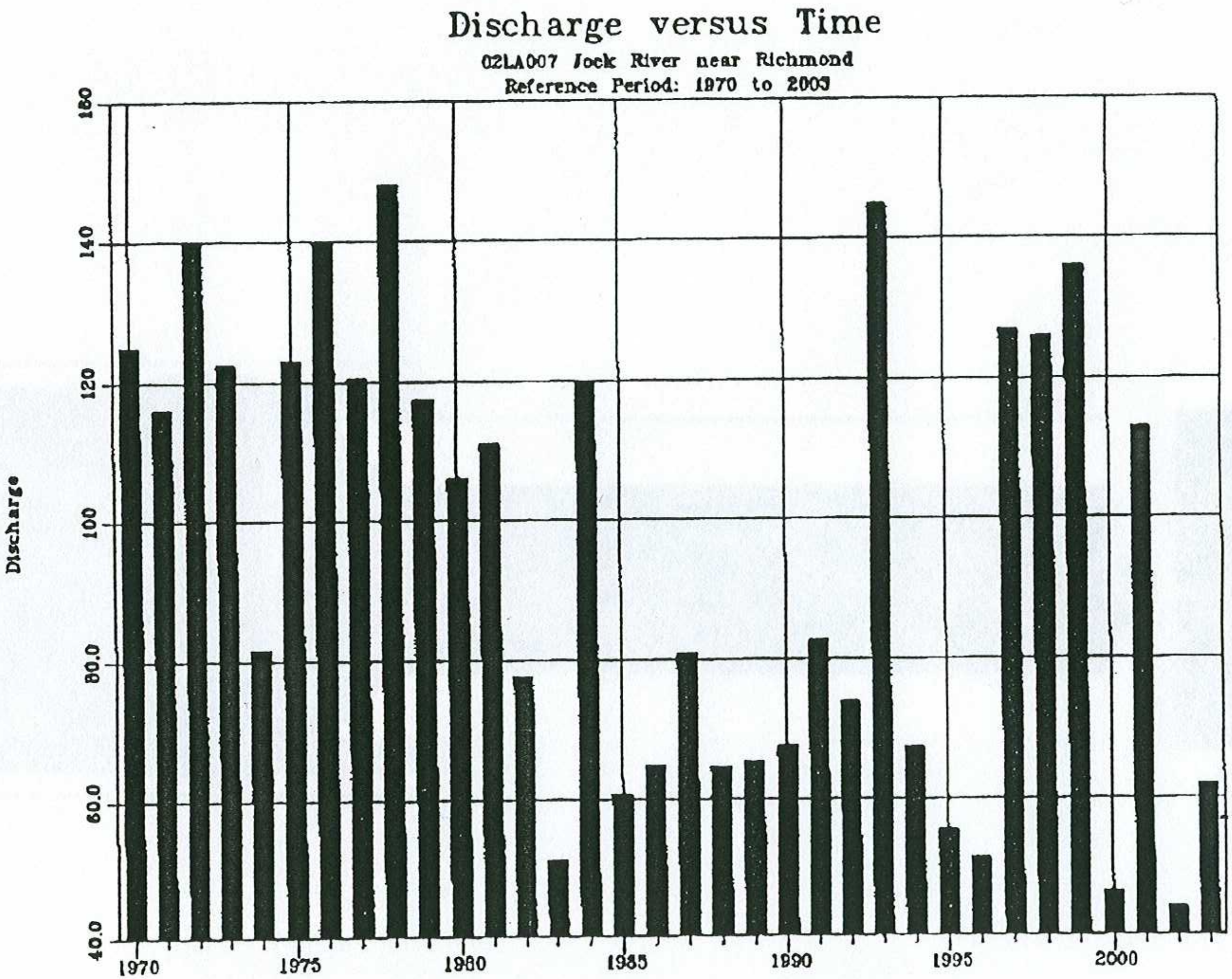



Figure 2 – Plot of Annual Peak Flow by Year (1970-2003)

**Table 2: Recommended Spring and Summer Flows – Jock River and Tributaries**

**Flows (m3/s)**

**Location and Hydrologic Model Reference #**

(Spring – SSFA – observed/prorated)*****

*(Spring event – 10 day volume - modeled)******

*(Summer event – SCS 24 hour - modeled)******

| **Return Period=> (years)** | **2** | **5** | **10** | **20** | **50** | **100** |
|---|---|---|---|---|---|---|
| Rideau River (N1) | 95 | 129 | 148 | 167 | 189 | 205 |
| Moodie Drive and d/s Monoghan Drain(N2) | 91 | 123 | 142 | 160 | 181 | 196 |
| d/s Flowing Creek (N5) | 82 | 110 | 127 | 144 | 162 | 176 |
| d/s Richmond Fen (N7) | 72 | 98 | 113 | 127 | 144 | 156 |
| d/s King Creek (N10) (u/s Richmond Fen) | *46* | *70* | *86* | *107* | *125* | *141* |
| Franktown Road (N10-KC) | *27* | *42* | *51* | *64* | *74* | *83* |
| Ashton (N12) | *8* | *11* | *13* | *16* | *18* | *20* |
| Monaghan Drain | *11* | *18* | *22* | *29* | *34* | *40* |
| Flowing Creek | *15* | *22* | *28* | *37* | *44* | *51* |
| King Creek | *11* | *16* | *20* | *25* | *30* | *34* |

*** font type and underlining indicate technique used in deriving flow**

compared to those observed. This additional validation provides a level of confidence that Return Period design events produce peak flows of reasonable magnitude.

### 2.2.1 Watershed Delineation

The watershed was divided into 14 major catchments and 11 minor catchments with points of interest as identified in Figure 3.

Major sources of topographic data included 1:50,000 NTS and 1:10,000 OBM maps. Catchment identification was undertaken by visual assessment of the topographic data, field investigation, as well as automated delineation using GIS techniques as provided by the RVCA. For hydrologic analysis, the catchment delineation is appropriate and, on average, represents the best current information available.

### 2.2.2 Hydrologic Characteristics

A hydrologic model of the watershed was developed using SWMHYMO (version 5.02 - beta). A summary of the hydrologic characteristics of the watershed are provided in Table 3.

The Richmond Fen and the reach, upstream of Ashton, that transects the Goodwood Marsh, were modeled as reservoirs to more accurately reflect their impacts on attenuating downstream peak flows.

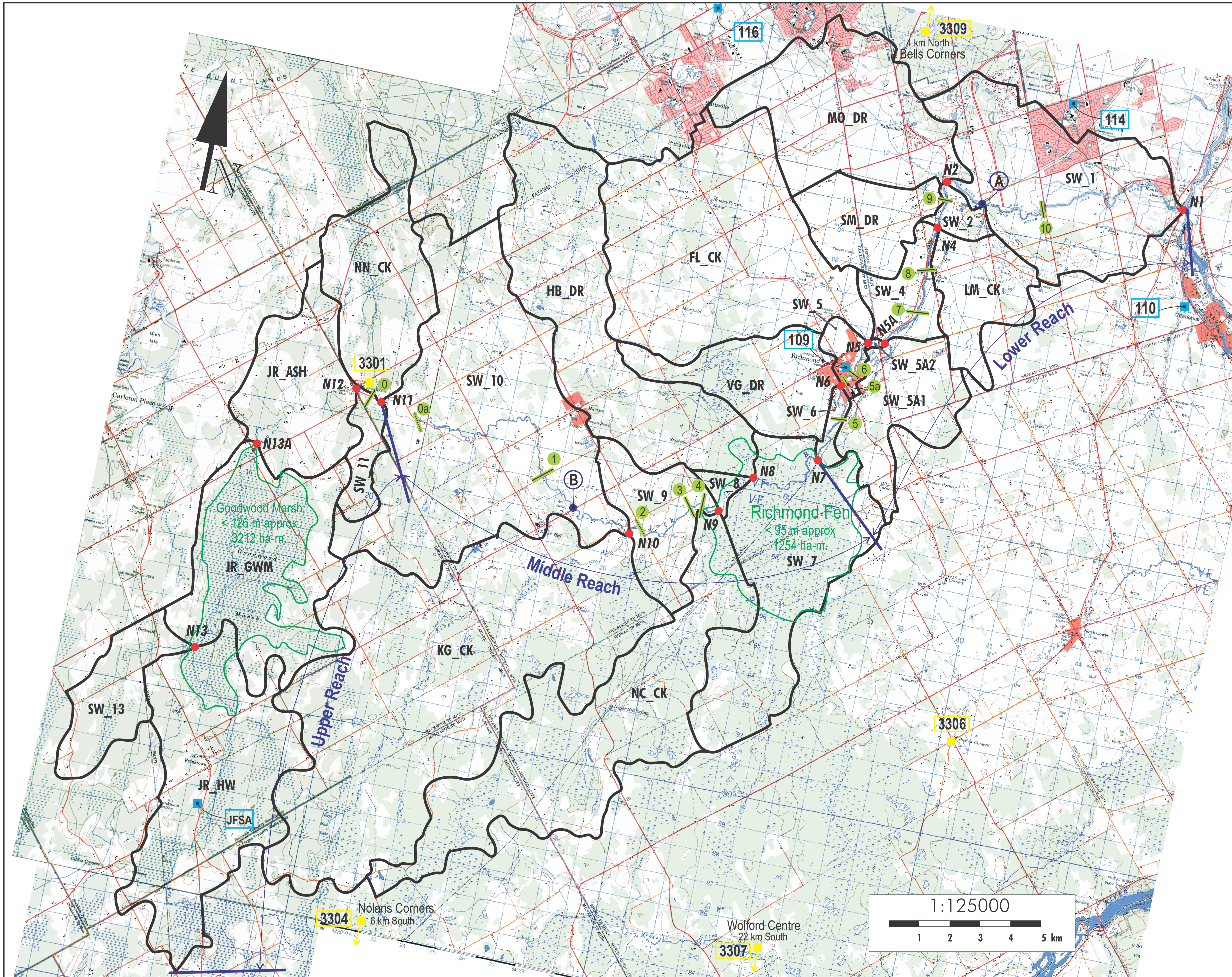
The following studies have been reviewed for consistency with the hydrologic model being currently developed and, with one exception noted below, no major discrepancies or ambiguities have been found:

- Rideau River watershed model - HSPF (1990).
- Stittsville MDP (1994)
- the Richmond MDP (1995)
- Monaghan Drain MDP (1998)
- Upper Poole Creek - Subwatershed Study (2000)
- Van Gaal Drain - Erosion Study (2001)
- Dwyer Hill Training - Centre SWM plan (2002)

Of note, however, are the headwaters of Poole Creek, where it has been determined that approximately 625ha drain to the Jock River via Hobbs Drain, rather than to Poole Creek.

**Table 3: Summary - Hydrologic Parameters**

| Parameter => | hydrologic model ID | drainage area (ha) | Spring CN | Spring Tp(hr) | Summer CN | Summer Tp(hr) |
|---|---|---|---|---|---|---|
| **subcatchment** | | | | | | |
| headwaters | JR_HW | 3680 | 35 | 5.4 | 64 | 7.1 |
| subcathcment 13 | SW_13 | 971 | 35 | 2.9 | 61 | 3.8 |
| Goodwood Marsh | JR_GWM | 3074 | 35 | 6.3 | 55 | 11.3 |
| Ashton | JR_ASH | 1781 | 35 | 3.9 | 72 | 3.9 |
| unnamed creek | NN_CK | 1917 | 35 | 2.9 | 66 | 5.3 |
| subcatchment 11 | SW_11 | 500 | 35 | 1.2 | 66 | 1.2 |
| Kings Creek | KG_CK | 8376 | 35 | 6.7 | 66 | 11.7 |
| subcatchment 10 | SW_10 | 5666 | 35 | 5.3 | 72 | 8.0 |
| Nicholls Creek | NC_CK | 4464 | 35 | 6.2 | 62 | 11.3 |
| subcatchment 9 | SW_9 | 1132 | 35 | 1.5 | 70 | 2.5 |
| subcatchment 8 | SW_8 | 131 | 35 | 0.5 | 63 | 0.9 |
| Hobbs Drain | HB_DR | 3854 | 35 | 5.1 | 66 | 8.4 |
| subcatchment 7 | SW_7 | 3197 | 35 | 3.7 | 57 | 6.7 |
| Van Gaal Drain | VG_DR | 1332 | 35 | 3.6 | 72 | 6.0 |
| subcatchment 6 | SW_6 | 165 | 35 | 2.4 | 67 | 4.2 |
| subcatchment 5 | SW_5 | 224 | 35 | 0.8 | 77 | 0.8 |
| Flowing Creek | FL_CK | 4945 | 35 | 3.7 | 74 | 4.5 |
| subcatchment 5A1 | SW_5A1 | 1412 | 35 | 5.0 | 75 | 8.0 |
| subcatchment 5A2 | SW_5A2 | 20 | 35 | 0.6 | 81 | 0.6 |
| subcatchment 4 | SW_4 | 585 | 35 | 1.8 | 81 | 1.8 |
| Leamy Creek | LM_CK | 1021 | 35 | 2.5 | 80 | 2.5 |
| subcatchment 2 | SW_2 | 177 | 35 | 0.8 | 77 | 0.8 |
| Smith Drain | SM_DR | 1122 | 35 | 3.3 | 81 | 3.3 |
| Monoghan Drain | MO_DR | 2737 | 35 | 3.0 | 76 | 3.0 |
| subcatchment 1 | SW_1 | 3176 | 35 | 3.6 | 78 | 3.6 |
| **Total** | | **55659** | | | | |




Legend:

Watershed
Bogs (Reservoir)
Nodes
Flow gauges
Rain gauges
Snow course stations
River Cross-Sections (based on topo. maps)
River Cross-Sections adjusted with field data
Reaches limits

**Flow gauges ID**

A 02LA007- Jock River near Richmond
B Jock River at Franktown Rd

**Rain gauges ID**

109 Richmond
110 Manotick
114 Barrhaven
116 Maple Grove
JFSA JFSA Inc, Temporary Rain Gauge 2003

**Snow course stations ID**

3301 Ashton
3304 Nolans Corners
3306 Pierces Corners
3307 Bells Corners
3309 Wolford Centre

Client:

RIDEAU VALLEY CONSERVATION AUTHORITY

Project:

Jock River Flood Plain Mapping Study

Title:

Watershed Delineation

J.F. Sabourin & Associates Inc.
WATER RESOURCES AND ENVIRONMENTAL CONSULTANTS
OTTAWA (613) 727-5199
GATINEAU (819) 243-6858

Figure 2

Ref. File: Base Map Jock River B.cdr

### 2.2.3 Spring Model (December 1st to April 30th)

**Calibration and Validation**

Using snowpack and temperature data measured during Spring 2003, snowmelt+rainfall hyetographs were developed by converting the snowpack to daily runoff volumes based on snowmelt estimates provided by degree-day equations. These volumes were then appropriately distributed over the day and formed an input to the model. Hydrologic model parameters such as time to peak, antecedent moisture condition, subcatchment runoff coefficients and channel routing were modified to achieve model calibration so that simulated and observed hydrographs had a best fit for peak magnitude, runoff volume and time to peak: the final results are adequate in terms of peak time, magnitude and volume. Based on these results, it can be assumed that the model can be used to derive a reasonable estimate of the 100 year maximum instantaneous peak flow on the Jock River between Richmond and Ashton.

The calibrated spring model has been validated by comparing the simulated flows for peak Spring events in 1978, 1993, 1997 and 1998 with observed flows at the Moodie Drive gauge. Although there were variations in the timing of the peak between simulated and observed, the hydrologic model adequately reflects the magnitude and volume of the Spring event.

Further validation of the model is provided by comparison of maximum instantaneous flows determined by the Spring design event with those determined by SSFA of maximum instantaneous observed flows.

**Inputs/Results**

The 100 year peak Spring flow was simulated using a synthetic 100 year snowmelt+rainfall event developed from AES snowmelt+rainfall IDF curves. These relationships have been developed for one through thirty day durations, with Return Periods from 2 through 100 years: The 10 day event was selected as being the appropriate duration for a synthetic Spring snowmelt+rainfall: it correlated well with the 100 year SSFA results for the Moodie Drive gauge.

A summary of the modeled peak flows for the Spring event are provided in Table 2 and suggest a 100 year maximum instantaneous peak flow, upstream of the Richmond Fen, of 141 $m^3/s$ (versus 116 $m^3/s$ using SSFA proration). Modeled 100 year peak at Moodie Drive is 205 $m^3/s$ which agrees well with the SSFA estimate of 196$m^3/s$.

### 2.2.4 Summer Model (May 1st to November 30th)

**Calibration and Validation**

Hourly streamflow data from the gauges at Moodie Drive and Franktown Road, in conjunction with hourly rainfall data from a temporary gauge at Franktown and the Richmond and Maple Grove gauges, were used in the calibration of the hydrologic model.

Validation of the model (and the design event) is provided by comparison of maximum instantaneous flows determined by the Summer design event with those determined by SSFA of continuous simulation results: peak flows are generally within 5% to 10% for the two modeling techniques.

For additional validation, Summer peak flows, from the 34 years of daily record at Moodie Drive, were reviewed to identify the annual maximum daily peak summer flow (maximum instantaneous flows are not readily available). SSFA of these annual daily maximums were compared to SSFA of annual daily maximums derived from hourly continuous simulation over 38 years of record: there is adequate agreement (within 15%) between the maximum daily observed flow and maximum daily simulated flow.

**Inputs/Results**

The 100 year peak Summer flow was estimated using a 100 year Design Storm. Ten different Design Storm distributions were assessed, along with various durations and included Chicago, SCS, AES and Huff distributions.

The Return Period flows derived from the various design storms were compared with the SSFA Return Period flows derived from the series of annual Summer instantaneous peak flows developed from continuous simulation. The best agreement occurs using the SCS 24 hour distribution in which the average ratio, for the six Return Period flows (2, 5, 10, 25, 50 and 100 years), is 1.001.

Using the SCS 24 hour distribution as input, the 100 year peak summer flow at Moodie Drive is estimated to be $141 m^3/s$. A summary of Return Period peak flows for various points of interest in the subwatershed is provided in Table 2.

## 3.0 Flood Level Estimates

Flood risk mapping requires the development of hydraulic simulation models to estimate the 100 year water level based on reliable flow estimates: these flow estimates have previously been established in a technical report "Hydrology Report – July 2004 - Jock River Flood Risk Mapping (within the City of Ottawa)".

The following sections outline the process required for hydraulic simulation, using HEC-RAS software (version 3.1.1 – May 2003), to estimate the required water levels so they may be plotted on appropriate base maps. This has been completed in conformance with the HEC-RAS manual and MNR approved technical guidelines for floodplain mapping. Cross sections for the hydraulic model have been developed based on two components: a channel cross section and related overbank (floodplain) sections. Some channel sections were field surveyed while all overbank sections were derived from digital base mapping. Appropriate digital base maps have been developed and provided by the City of Ottawa.

It is important to note that the calibration/validation effort, in both hydrology and hydraulics, concentrated on the simulation of high flows for the purpose of flood risk mapping: the estimates of more frequent Return Period flows and flood levels, such as the 2 year and 5 year, should be used with caution.

### 3.1 Base Mapping

The City of Ottawa has provided 1:2000 mapping, with 0.5m contours, for the Lower and Middle reaches of the Jock River. The digital maps have been photogrammetrically derived from air photography acquired by the RVCA in the Fall of 2001.

Two different scales of air photo were acquired, covering different areas, to ensure accuracy in areas where the topography was relatively flat and in the largely urban area of Richmond. This provides a level of confidence in the accuracy of the contours. The Lower Reach, that included Richmond, was photographed at 1:3600 while the Middle Reach was photographed at 1:6000.

Vertical and Horizontal Control was provided so that the 1:3600 photos were triangulated with an accuracy of .025m and .045m, respectively and the 1:6000 photos had a triangulation accuracy of .037 and .059 respectively.

Field checks for completeness and accuracy were completed by the City of Ottawa and confirm that the digital mapping has 0.3m horizontal accuracy and 0.25m vertical accuracy for the Middle Reach and 0.12m horizontal accuracy and 0.08m vertical accuracy for the Lower Reach. These are within acceptable limits as defined in "surveys and mapping" procedures for floodplain mapping; a component of MNR's Technical Guidelines for Floodplain Management in Ontario. In this document, 0.33m vertical accuracy and 1.0m horizontal accuracy are prescribed.

Flood Risk Maps were developed at a scale of 1:5000 for rural areas and 1:2000 for the urban area of Richmond: both with 0.5 m contouring. The layout is illustrated in Figure 4.

### 3.2 Cross Sections

Cross sections were developed using a combination of field survey, interpolation and the 0.5m contour digital base mapping to develop the necessary channel and overbank components. All channel sections relating to bridges and culverts were field surveyed while all overbanks were derived from digital mapping. Other sections were developed, at minimum intervals of 500m., by interpolating the channel section and deriving the relevant overbank elements from the digital base mapping. Cross sections were also defined at locations where significant changes in stream alignment and slope occurred as well as at locations where the stream width/floodplain significantly increased or decreased.

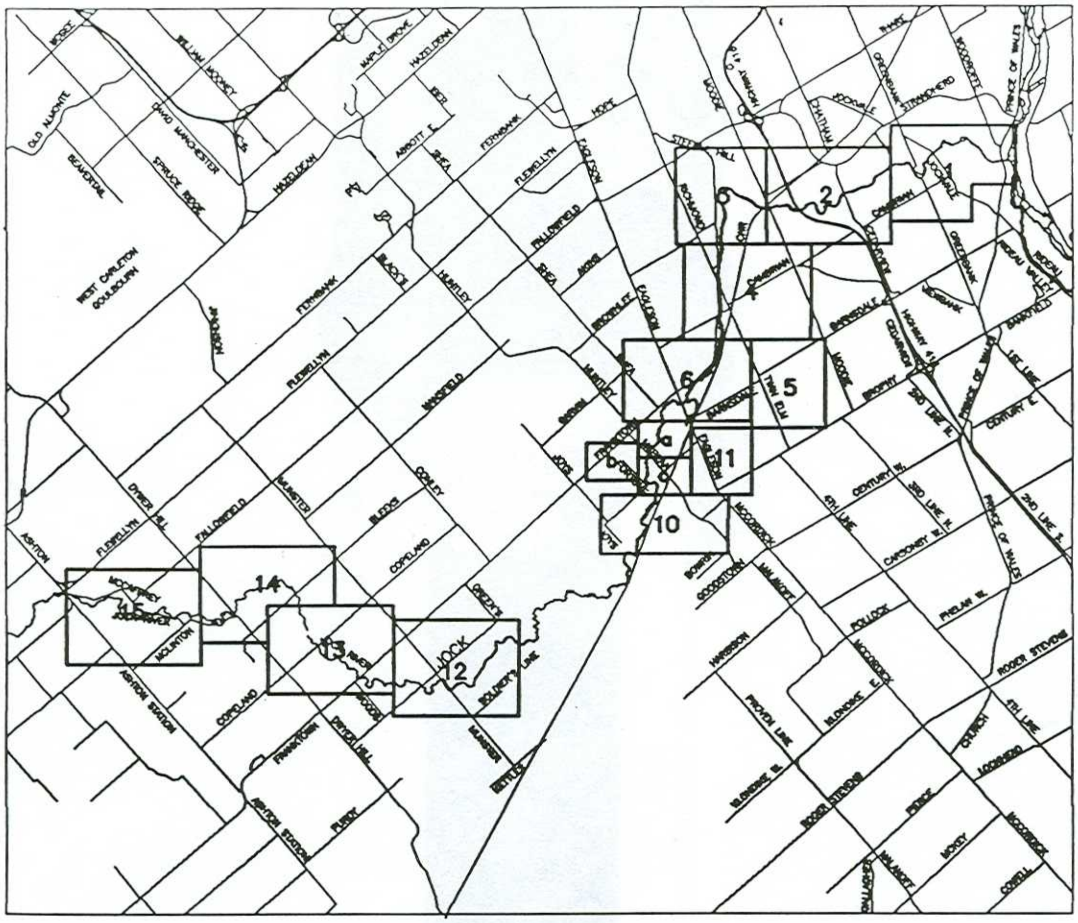
Field survey relied heavily on the use of GPS equipment, not only to establish a series of accurate temporary bench marks on each bridge and culvert but also, where topography and bridge layout permitted, to survey channel sections, both upstream and downstream of these bridge locations, from bank to bank. All overbanks were based on data from the 0.5m contour mapping and were extracted from the digital maps using EAGLE III software. The elevations and co-ordinates obtained from GPS were found to be within 0.01m. – 0.03m., both vertically and horizontally, when compared to geodetic bench marks.

Manning’s ‘n’ values, which are used to characterise the friction effect of the channel and overbank material on flow, were derived from field and air photo investigation. Both Chow (1959) and the USGS (2001) were used in estimating the appropriate values through comparison with observations.

Expansion and contraction coefficients of 0.1 and 0.3 respectively, were used for all cross sections except those upstream and downstream of bridges and culverts where 0.3 and 0.5 were used: these larger coefficients reflect the more rapid change in velocity due to flow contraction through the bridge or culvert opening.

Starting water surface levels, or boundary conditions, for the Lower Reach, were based on maximum water levels for the Rideau River for the relevant 2 through 100 year event: this implies an assumption of concurrent peaks. For the Middle Reach, it was assumed that the corresponding Return Period water levels for the Lower Reach would form the starting water surface level: this implies that the water level through the wetland separating the two reaches is constant.

SHEET INDEX TABLEAU D'ASSEMBLAGE




15 — 1:5000 Map Sheet

e — 1:2000 Map Sheet

**Figure 4 – Mapsheet Layout**

### 3.3 Structures

All bridges and culverts were surveyed in the field – standard methods, rather than GPS, were used in some cases due to bridge configuration and a resulting poor satellite reception. Typically, the channel under the bridge and the roadway was surveyed; the abutment width and soffit elevation were also determined. More detail may be found in the technical document: **"**Hydraulics Report – November 2004 – Jock River Flood Risk Mapping (within the City of Ottawa) "

### 3.4 Calibration

The hydraulic model was calibrated, as effectively as possible (given data limitations both in the number of events and the locations monitored), using the following observed water levels:

- Jock River at Moodie Drive – WSC Rating Curve
- observed water levels in Richmond in 1999 (136$m^3$/s at Moodie) at
    - Flowing Creek at Perth Street
    - Jock River at McBean Street Bridge
    - Van Gaal Drain at Fowler Street
- Jock River at Greenbank Road in 1998 (126 $m^3$/s at Moodie).

Starting from field observations and standard values described in the HEC-RAS Users Manual , Chow and USGS manual, both Manning's 'n' and expansion/contraction coefficients were assigned to each cross section; these were subsequently modified, during calibration runs, until the estimated values approximated observed values.

A comparison of observed and estimated values for water levels at Moodie Drive is provided in Figure 5 and shows good correlation between observed and simulated water levels. The observed values are based on the Water Survey of Canada rating curve at Moodie Drive which has been derived from observed flow rates and corresponding water levels recorded, in the field, over the past 30 years. The maximum flows of record, in 1978 and 1993, were 145$m^3$/s +/- and compare extremely well with the simulated values; there is some minor divergence at the upper end between simluated and observed values (@200$m^3$/s - 0.01m+/-) but this may be considered negligible.

### 3.5 Water Surface Profiles

The recommended flows for generating 100 year water levels are summarised in Table 2. The application of theses flows to the hydraulic model is described below.

#### 3.5.1 Lower Reach

Flows for the 2 through 100 year Spring events were identified for various locations in the Lower Reach: they were estimated using areal proration techniques as applied to the results of Single

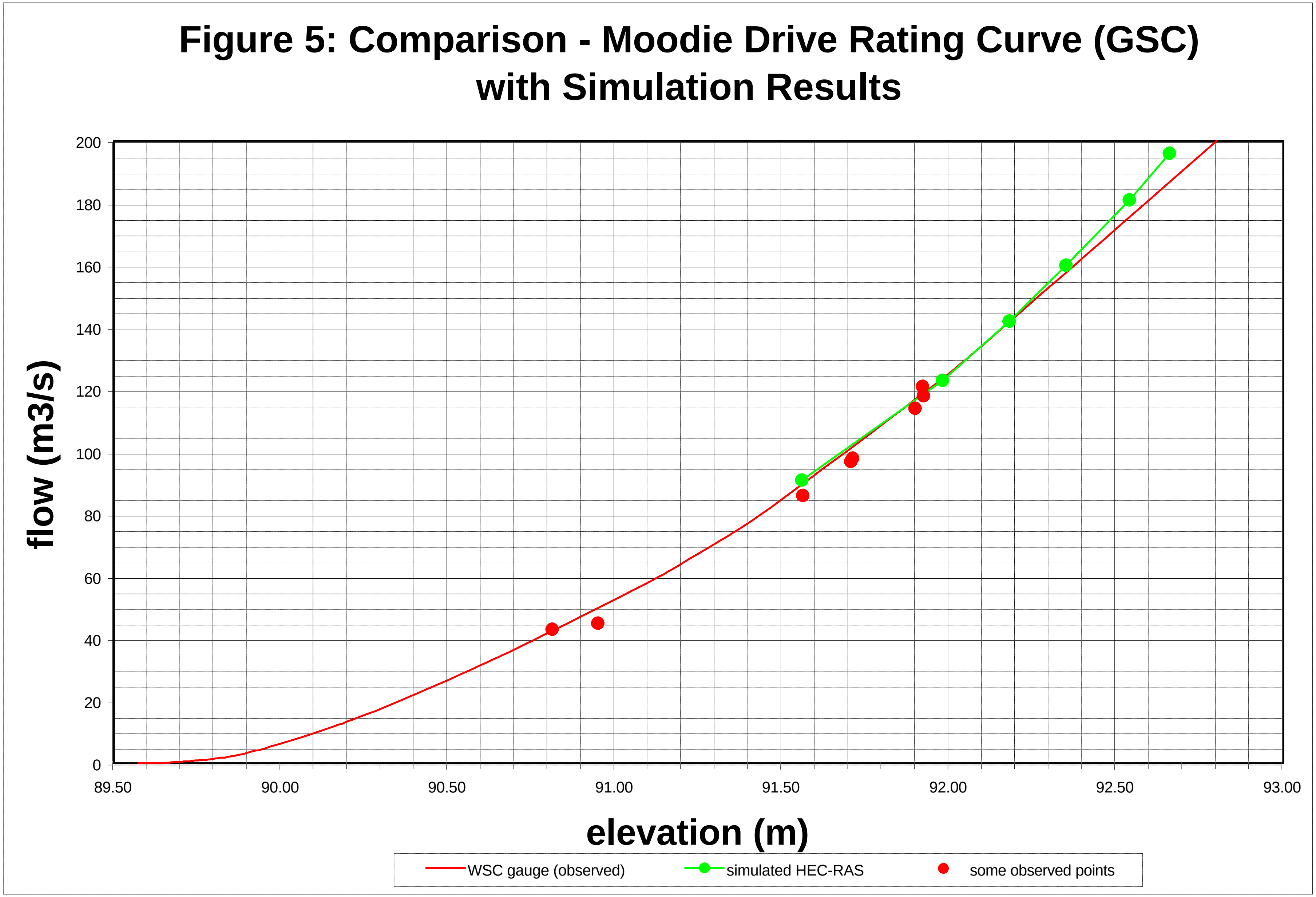


Figure 5: Comparison - Moodie Drive Rating Curve (GSC)
with Simulation Results
flow (m3/s)
0
20
40
60
80
100
120
140
160
180
200
89.50
90.00
90.50
91.00
91.50
92.00
92.50
93.00
elevation (m)
WSC gauge (observed)
simulated HEC-RAS
some observed points

Station Frequency Analysis of maximum instantaneous peak flows at Moodie Drive. The flows were then used as inputs to the hydraulic model which resulted in estimates of water levels for every cross section. Water surface profiles for the 5 year and 100 year Spring events are provided in Figure 6.

### 3.5.2 Middle Reach

Flows for the 2 through 100 year Spring events were identified for various locations in the Middle Reach: they were estimated using a hydrologic model of maximum instantaneous peak flows for the Spring event; areal proration techniques were used to supplement the data. The flows were then used as inputs to the hydraulic model which resulted in estimates of water levels for every cross section. Water surface profiles for the 5 year and 100 year Spring events are provided in Figure 7.

### 3.5.3 Tributaries

The 100 Year flood level for a tributary will be influenced by water levels in the Jock River. The maximum 100 Year flood level for a tributary is based on hydraulic analysis that considers flows on the Jock River, and the tributary, that have a combined probability of once in 100 years.

For Summer events, the 5 , 10, 20 and 50 year flows on the tributaries were modeled with corresponding 20, 10, 5 and 2 year flows on the tributary; each combination having a combined probability of a 100 year water level on the tributary. For the 100 year event, it was assumed that a corresponding 2 year water level would approximate a combined 100 year probability.

For the Spring event, it was assumed that, as a result of the timing of tributary peaks determined in the hydrologic modeling and with confirmation by RVCA staff observations, the main peak on the Jock was observed to occur at least 12 hours after the tributary peak.

A review of the results of combined probability modeling, compared with those of the Spring modeling, suggest that the Spring flows and water level on the Jock River produce the maximum 100 year water levels on the tributaries. The flood levels are summarised in Table 4.

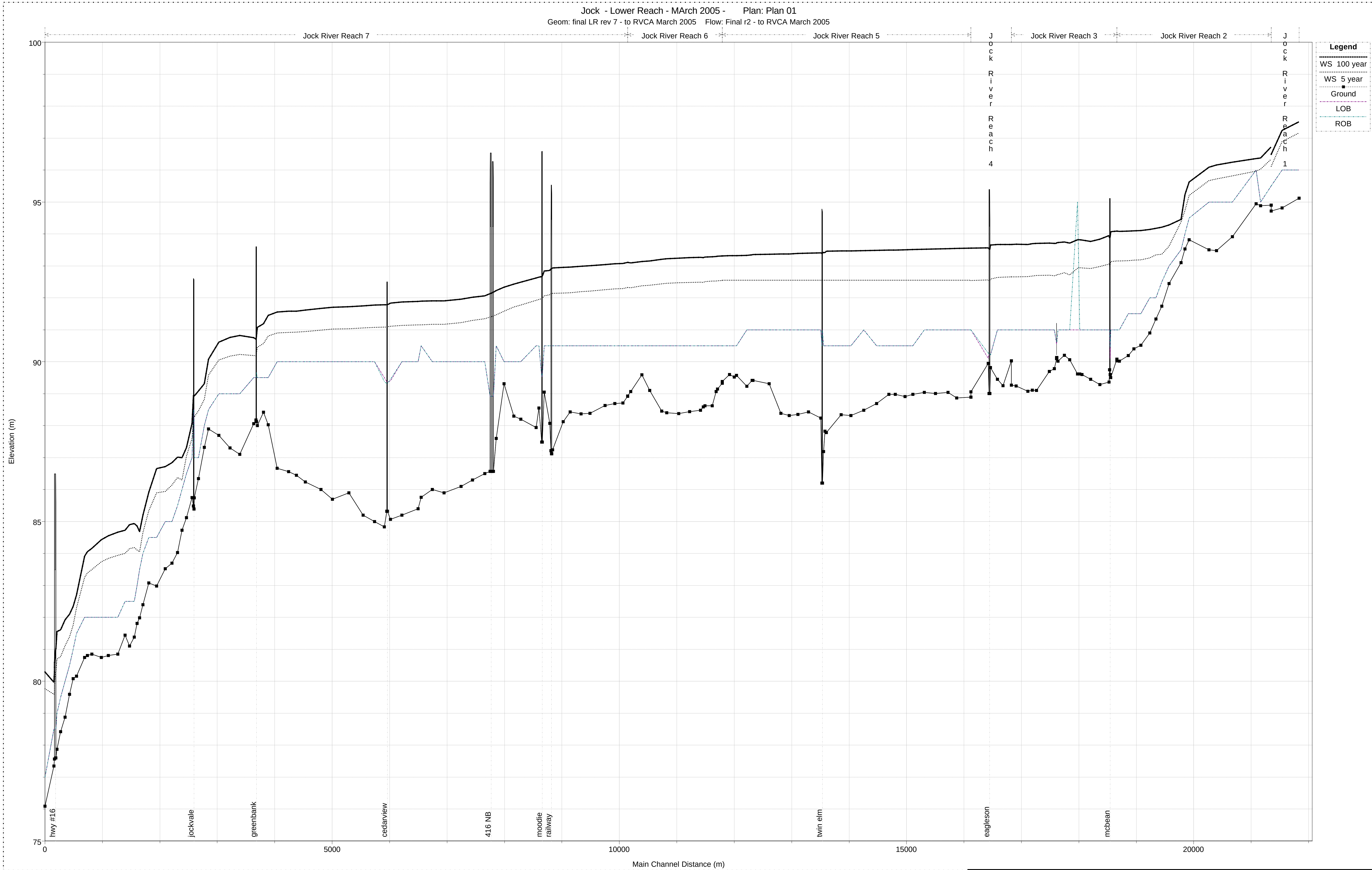



Figure 6 - Water Surface Profile - Lower Reach

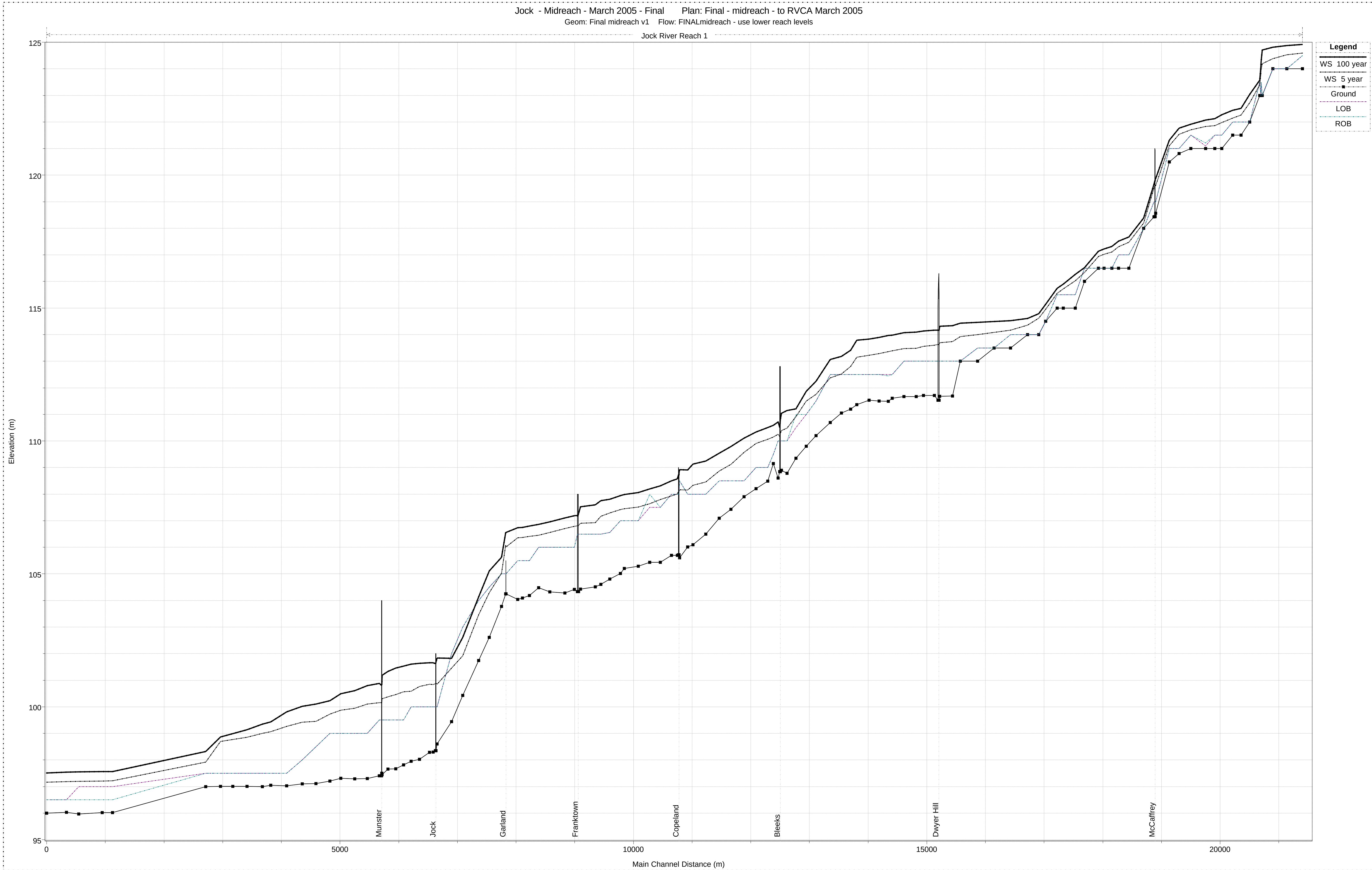



Figure 7 - Water Surface Profile - Middle Reach

## 4.0 Flood Risks

### Water Crossings

- based on a review of the 100 Year Jock River water surface profile in the Lower Reach, all watercourse structures appear to be adequately sized to convey the 100 Year event without increasing upstream water levels.
- based on a review of the 100 Year Jock River water surface profile in the Middle Reach, structures at Munster Road, Jock Trail, Franktown Road, Copeland Road and Bleeks Road all appear slightly undersized for the 100 Year event and have a minor impact on upstream water levels.

### Roadways

- based on a review of the 100 Year Jock River water surface profile in the Lower Reach, the major roadways affected by the 100 Year event are: Richmond Road between Steeple Hill and Richmond - portions of the roadway are estimated to experience minor flooding (up to 0.1m in depth); Eagleson Road .
- based on a review of the 100 Year Jock River water surface profile in the Middle Reach, no major roadways appear to be affected by the 100 Year event.

### Structures

- based on a review of the 100 Year Jock River water surface profile in the Lower Reach, there are 25 +/- buildings in the floodplain
- based on a review of the 100 Year Jock River water surface profile in the Middle Reach, there are no buildings in the floodplain

## 5.0 Flood Risk Maps

The 100 Year water levels identified in the hydraulic analyses were plotted on the 1:2000 and 1:5000 scale base maps for both urban and rural areas, respectively.

In the developed residential areas of Richmond, several low lying areas were identified that were directly connected to the Jock River, either by swales or culverts. Their floodprone characteristics were identified in the floodplain mapping by including them in the floodplain: these areas are generally not considered important from a flood conveyance point of view since they are separated from the main channel by raodways or other features; but it is approporiate to identify them as being flood susceptible during the 100 Year flood.

Reduced scale (50% +/-) versions of these maps are provided in Appendix A.

## References

1. Hydrology Report – July 2004 – Jock River Flood Risk Mapping (within the City of Ottawa)

2. Hydraulics Report – November 2004 – Jock River Flood Risk Mapping (within the City of Ottawa)

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5. HEC-RAS River Analysis System – Users Manual v3.1 – USACE - November 2002

6. Rideau River Watershed Model (1990). – RVCA – Marshall, Macklin, Monoghan

7. Stittsville MDP (1994) – Township of Goulbourn – AJ Robinson and Associates

8. Richmond MDP Update (1998) – Township of Goulbourn – Robinson Consultants

9. Monaghan Drain MDP (1993) – City of Kanata – Gore and Storrie

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11. Van Gaal Drain - Erosion Study (2001) – Cedarstone Homes – PSR Group Ltd

12. Dwyer Hill Training Centre - SWM plan (2002) – DND – McManus Engineering

13. Jock River Floodplain Mapping (1981) – RVCA – Acres Consulting Services Ltd.

14. Jock River Watershed Plan – Component Report – Surface Water Quantity (1996) – RVCA – JL Richards

15. Consolidated Frequency Analysis – User Manual - Version 1 (1985)– Environment Canada

16. MTO Drainage Manual – Volume C (1986)– Ministry of Transportation Ontario

17. Statistical Analysis of Extreme Floods (2002) – University of South Africa - WJR Alexander

# Appendix A – Flood Risk Maps (reduced 50%+/-)