WATER RESOURCES SURVEY

Contents

Eastern Ontario
WATER RESOURCES MANAGEMENT STUDY

Water Resources Survey

To help us with the study, please take a few minutes to answer the following questions. Please use the postage-paid return envelope to return your survey. If you would like more information about the study, please feel free to contact any of the project managers listed in the enclosed newsletter.

1. Please indicate if you are responding to this survey as a:

   ❒ household OR ❒ business OR ❒ other ____________________

2. How concerned are you about the quality of water you drink?

   ❒ very concerned ❒ somewhat concerned ❒ not really concerned

3. How concerned are you about the quantity of water that is available?

   ❒ very concerned ❒ somewhat concerned ❒ not really concerned

4. If you have a concern, could you please indicate to us the nature of your concern by checking all of the factors that apply to your household or to your business.

   ❒ dry well
   ❒ water shortage (other than a dry well)
   ❒ high salt content
   ❒ high iron content
   ❒ high sulphur content
   ❒ bad taste
   ❒ bad smell
   ❒ discoloured water
   ❒ unacceptable fecal coliform content
   ❒ sediment in the water
   ❒ do not have any problems with water quality
   ❒ do not have any problems with water quantity
   ❒ other (please describe)
   ____

5. Have you experienced changes in your water quality? ❒ Yes ❒ No

   If yes, what do you think was the cause? ____________________

6. How often do you use bottled water in your household or your business?

   ❒ daily ❒ 2-3 times a week ❒ once a week
   ❒ seasonally ❒ never use bottled water

7. Do you treat your drinking water? ❒ Yes ❒ No

   If yes, what type of treatment? ____________________

8. What is the source of your water supply for your household or business?

   ❒ municipal piped water ❒ my own drilled well
   ❒ my own dug well or sand point ❒ my own intake from a lake or river

9. What conservation measures would you support in your community to better manage water resources? (Please check all items that apply.)

   ❒ install water meters to encourage more efficient use
   ❒ introduce restrictions for watering gardens and lawns
   ❒ charge more for water so that users pay the full cost of the water they use
   ❒ promote water conservation measures suitable for urban and rural areas
   ❒ conserve wetlands
   other (please describe) ____________________
   ____

10. Do you have a septic system? ❒ Yes ❒ No
11. If so, what is the age of the septic sysetem? __________ years
12. Do you have a regular maintenance schedule for your septic system? ❒ Yes ❒ No

13. What measures do you think would help to improve the quality or quantity of water? (Please check all items that apply to your household, your business, or your community.)

My household	My business	My community	
❒	❒	❒	install a piped water system
❒	❒	❒	treat or improve the treatment of the municipal water supply
❒	❒	❒	repair sewer pipes to eliminate water infiltration
❒	❒	❒	ensure that proposed land development does not increase erosion
❒	❒	❒	ensure that land use or land development does not contaminate groundwater
❒	❒	❒	improve or upgrade the treatment of sewage
❒	❒	❒	conserve or construct wetland areas to clean and filter water
❒	❒	❒	protect aquifers and recharge areas from potential contamination
❒	❒	❒	replace or retrofit defective septic tanks and tile fields
❒	❒	❒	retain or return shorelines of lakes and rivers to their natural state
❒	❒	❒	control the quantity and quality of drainage for all new development
❒	❒	❒	control or reduce the use of pesticides and herbicides
❒	❒	❒	require nutrient management plans for farming operations
❒	❒	❒	eliminate sources that may contaminate water

Other (please describe) ____

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____

14. Are you interested in receiving future newsletters? ❒ Yes ❒ No

15. Are you interested in attending upcoming open houses and public meetings associated with this study?

❒ Yes ❒ No

16. Do you wish to be contacted to receive more information? ❒ Yes ❒ No

Please complete this survey today and return it to us using the enclosed postage-paid envelope.

The information collected for this survey is confidential and is protected under the Municipal Freedom of Information and Protection of Privacy Act. However, by providing us with as much of the following information as possible, you can assist us to identify communities or areas where measures can be undertaken to better manage water resources. As a minimum, please indicate your municipality.

Name (please print) ____

Mailing Address ____

____ Postal Code ________________

For rural areas only: Lot _____ Concession _____

Municipality ____

Thank you for your participation!

All completed surveys that contain contact information will be entered in a draw to win a copy of Water: The Fate of Our Most Precious Resource by Marq De Villiers

Eastern Ontario Water Resources Management Study (EOWRMS) Infrastructure Survey

Part I
Water Supply Survey

Prepared for:

United Counties of Prescott & Russell
United Counties of Stormont, Dundas & Glengarry
Region of Ottawa-Carleton

Prepared by:

Neil A. Levac Engineering Ltd/LTÉE

In Association with:

CH2M Gore & Storrie Limited

January 2000

1. General Questionnaire

County: ____

Municipality: ____

Total population in 1999: _________

Total households in 1999: _________

List all the villages, towns, and rural areas in your municipality. Indicate if each is serviced by a distribution system and the type of water sources in use.

Village, Town, or Rural Area	Distribution System	Communal Groundwater System	Communal Surface Water System	Private Wells	Other Water Supply Systems

Please complete the following questionnaires for each village, town, or rural area. Make photocopies as required for multiple facilities.

2. Distribution System Questionnaire

General

Town, village, or rural area served: ____________________________

Total population of town, village, or rural area in 1999: __________________

Total population served by this system in 1999: _________________

Total land area served by this system in 1999: ______________

Owner: ____

Operating authority: __________________________________

Growth forecast of the population in:
5 years: _________
10 years: _________
20 years: _________

Service Areas:
Number of residential units: _________
Number of commercial units: _________
Number of institutional units: _________
Number of industrial units: _________

Water Main

Estimated original capital cost ($): ______________

Total length (m): ______________

Diameter range (mm): from _______ to _________

Type of pipe: Plastic Polyethylene Concrete Other ____________
Wood Asbestos cement Cast iron

Number of hydrants: ______________

Age range (years): from _______ to ____________

Life expectancy (years): 5 years 10 years 15 years More than 20 years

Anticipated major repairs or replacement in: Less than 5 years 5 to 10 years
11 to 20 years More than 20 years

Capital cost investment for last 5 years ($): _____________

Proposed capital expenditure for expansion in the next: 5 years: $_________
10 years: $_________
20 years: $_________

Do you have in-house as-built drawings for the system? Yes No

If no, what are possible sources: ______________________________

If yes, in what format are they? ______________________________

Booster Stations

Estimated original capital cost ($): ____________

Number of booster stations: _____________

Range capacity (m^3/day): from _______ to ___________

Age range (years): from _______ to __________

Life expectancy (years): 5 years 10 years 15 years More than 20 years

Anticipated major repairs or replacement in: Less than 5 years 5 to 10 years 11 to 20 years More than 20 years

Capital cost investment for last 5 years ($): _____________

Proposed capital expenditure for expansion in the next: 5 years: $_________
10 years: $_________
20 years: $_________

Do you have in-house as-built drawings for the system? Yes No

If no, what are possible sources: ______________________________

If yes, in what format are they? ______________________________

Comments

3. Communal Groundwater System Questionnaire

Town, village, or rural area served: ____________________________

Total population served by this communal groundwater system in 1999: ___________

Total land area served by this system in 1999: ______________

Owner: _______________________________________

Operating authority: ___________________________

Estimated original capital cost ($): _______________ **Year:** ___________

Number of wells in operation: ______________

Design capacity of each well(s) (m^3/day): _______________________

Total reserve capacity (m^3/day): ________________

Average 1999 production (m^3/day): ______________

Maximum day demand in 1999 (m^3/day): __________________

Average annual production (m^3/year): ________________ **Years**: _______ to _______

Storage reservoir: Yes type: ____________ total capacity (m^3): ________
No

Type of treatment: ___________________

Treatment process: ___________________

Treatment reserve capacity: _________________

Water meters: Yes No

Water shortage during high demand or dry weather: Yes No

Is system designed and used for fire protection? Yes No

Are there any water restrictions (by-law)? Yes No

Have you recently (in the past 10 years) encountered problems with the following:

Water supply quantity: give year(s): _______ reason _______________________

Water supply quality: give year(s): _______ reason _______________________

Number of water use restriction days in 1999: ___________

Number of boil-water days in 1999: ___________

Expected growth: _____________________

Age range of major system components (years): from _______ to _________

Average yearly operation cost ($/year): _______________

Life expectancy (years): 5 years 10 years 15 years More than 20 years

Anticipated major repairs or replacement in: Less than 5 years 5 to 10 years 11 to 20 years More than 20 years

Capital cost investment for last 5 years ($): _____________

Proposed capital expenditure for expansion in the next:
5 years: $_________
10 years: $_________
20 years: $_________

Do you have in-house as-built drawings for the system? Yes No

If no, what are possible sources: ____

If yes, in what format are they? ____

Comments

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4. Communal Surface Water System Questionnaire

Town, village, or rural area served: ______________________________

Total population served by this communal surface water system in 1999: _____________

Total land area served by this system in 1999: _______________

Owner: ____

Operating authority: _____________________________

Estimated original capital cost ($): ________________ **Year:** ____________

Source(s) of water: _______________

Design capacity (m^3/day): _________________________

Total reserve capacity (m^3/day): _________________

Average 1999 production (m^3/day): _______________

Maximum day demand in 1999 (m^3/day): ___________________

Average annual production (m^3/year): _________________ **Years**: _______ to _______

Storage reservoir: Yes type: _____________ total capacity (m^3): _________
No

Type of treatment: ____________________

Treatment process: ____________________

Treatment reserve capacity: __________________

Water meters: Yes No

Water shortage during high demand or dry weather: Yes No

Is system designed and used for fire protection? Yes No

Are there any water restrictions (by-law)? Yes No

Have you recently (in the past 10 years) encountered problems with the following:

Water supply quantity: give year(s): _______ reason _________________________

Water supply quality: give year(s): _______ reason _________________________

Number of water-use restriction days in 1999: ___________

Number of boil-water days in 1999: ___________

Expected growth: _____________________

Age range (years): from _______ to ________

Average yearly operation cost ($/year): _______________

Life expectancy (years): 5 years 10 years 15 years More than 20 years

Anticipated major repairs or replacement in: Less than 5 years 5 to 10 years 11 to 20 years More than 20 years

Capital cost investment for last 5 years ($): _____________

Proposed capital expenditure for expansion in the next:
5 years: $_________
10 years: $_________
20 years: $_________

Do you have in-house as-built drawings for the system? Yes No

If no, what are possible sources: ____

If yes, in what format are they? ____

Comments

5. Private Wells Questionnaire

Town, village, or rural area: ______________________________

Total population served by private wells in 1999: _______________

Total land area served by private wells in 1999: ___________________

Approximate number of wells in 1999: _________________

Water quality: Good Treatment required

Water quantity: Adequate Storage required

Comments

6. Other Water Supply Systems Questionnaire

Town, village, or rural area served: ______________________________

Type of water supply: _________________________________

Type of water treatment: _________________________________

Total population served by this system in 1999: _______

Total land area served in 1999: ___________________

Comments____

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Eastern Ontario Water Resources Management Study (EOWRMS) Infrastructure Survey

Part II
Wastewater Treatment Survey

Prepared for:

United Counties of Prescott & Russell
United Counties of Stormont, Dundas & Glengarry
Region of Ottawa-Carleton

Prepared by:

Neil A. Levac Engineering Ltd/LTÉE

In Association with:

CH2M Gore & Storrie Limited

January 2000

Contents

1. General Questionnaire

County: ____

Municipality: ____

Total population in 1999: ________

Total households in 1999: ________

List all the villages, towns, and rural areas in your municipality. Indicate if each is serviced by a collection system and the type of wastewater treatment system(s) in use.

Village, Town, or Rural Area	Collection System	Treatment Plant	Lagoon	Recirculating Sand Filter	Communal Septic System	Private Septic System	No Treatment	Other Systems

Please complete the following questionnaires for each village, town, or rural area. Make photocopies as required for multiple facilities.

2. Collection System Questionnaire

General

Town, village, or rural area served: ____________________________

Total population of town, village, or rural area in 1999: __________________

Total population served by this system in 1999: _________________

Total land area served by this system in 1999: ______________

Owner: ____

Operating authority: __________________________________

Growth forecast of the population in:

5 years:	_________
10 years:	_________
20 years:	_________

Service areas:

Number of residential units:	_________
Number of commercial units:	_________
Number of institutional units:	_________
Number of industrial units:	_________

Sewer

Type of collection system: Gravity Step Other ________________

Estimated original capital costs ($): ______________

Total length (m): ______________

Range in diameter (mm): from _______ to _________

Type of pipe:

Plastic	Clay	Concrete	Corrugated metal
Wood	Asbestos cement	Iron or steel	Other_____________

Number of manholes: ______________

Age range (years): from _______ to ____________

Life expectancy (years): 5 years 10 years 15 years More than 20 years

Anticipated major repairs or replacement in:

Less than 5 years	5 to 10 years
11 to 20 years	More than 20 years

Capital cost investment for last 5 years ($): _____________

Proposed capital expenditure for expansion in the next: 5 years: $_________
10 years: $_________
20 years: $_________

Are there combined storm-sanitary sewers in the system? Yes No

Is there any known cross-connection to the system? Yes No

Are there any known or possible infiltration/exfiltration problems? Yes No

Do you have in-house as-built drawings for the system? Yes No

If no, what are possible sources? ______________________________________

If yes, in what format are they? ______________________________________

Pumping Station

Number of pumping stations: _______________

Estimated original capital cost ($): ____________

Range capacity of the pumping stations (m^3/day): from _______ to _________

Are there any backup generators? Yes No

Age range (years): from _______ to ________

Life expectancy (years): 5 years 10 years 15 years More than 20 years

Anticipated major repairs or replacement in: Less than 5 years 5 to 10 years
11 to 20 years More than 20 years

Capital cost investment for last 5 years ($): _____________

Proposed capital expenditure for expansion in the next: 5 years: $_________
10 years: $_________
20 years: $_________

Do you have in-house as-built drawings for the system? Yes No

If no, what are possible sources? ______________________________________

If yes, in what format are they? ______________________________________

Force Main

Force main length (m): ______________

Estimated original capital costs ($): ______________

Range in diameter (mm): from _______ to ________

Type of pipe: Plastic Iron or steel Other__________________
Asbestos cement Concrete

Age range (years): from _______ to _________

Life expectancy (years): 5 years 10 years 15 years More than 20 years

Future major repairs or replacement in: Less than 5 years 5 to 10 years
11 to 20 years More than 20 years

Capital cost investment for last 5 years ($): _____________

Proposed capital expenditure for expansion in the next: 5 years: $_________
10 years: $_________
20 years: $_________

Do you have in-house as-built drawings for the system? Yes No

If no, what are possible sources? ____

If yes, in what format are they? ____

Comments____

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3. Wastewater Treatment Plant Questionnaire

Town, village, or rural area served: ____________________________

Total population served by this sewage treatment plant in 1999: ___________

Total land area served by this system in 1999: ______________

Owner: _______________________________________

Operating authority: ___________________________

Plant name: ____________________________________

Estimated original capital cost ($): _______________

Age range (years): from ________ to __________

Life expectancy (years): 5 years 10 years 15 years More than 20 years

Type of treatment: Primary Secondary Tertiary

Treatment process: ____

____

Sludge management/treatment: Yes No

If yes, what type? ____

Design capacity of the treatment plant (m^3/day): ______________

Reserve capacity (m^3/day): ________________

Total inflow (m^3/day): ____________________

Average inflow (m^3/year): _________________

Total discharge (m^3/day): _________________

Maximum discharge (m^3/year): _____________

Average discharge (m^3/day): _______________

Average daily BOD_5 influent (mg/L): _________________

Average daily BOD_5 effluent (mg/L): _________________

Final effluent disinfection is: _____________________

Final effluent point of discharge: _________________

Discharge criteria: ______________________________

Expected growth: _________________

Average yearly operation cost ($/year): _______________

Capital cost investment for last 5 years ($): _____________

Proposed capital expenditure for expansion in the next:
5 years: $_________
10 years: $_________
20 years: $_________

Future major repairs or replacement in:
Less than 5 years
5 to 10 years
11 to 20 years
More than 20 years

Do you have in-house as-built drawings for the system? Yes No

If no, what are possible sources? ____

If yes, in what format are they? ____

Comments

4. Lagoon Questionnaire

General

Town, village, or rural area served: ____________________________

Total population served by this lagoon in 1999: ___________

Total land area served by this system in 1999: ______________

Owner: _______________________________________

Operating authority: ___________________________

Estimated original capital cost ($): _______________

Age range (years): from _______ to _________

Life expectancy (years): 5 years 10 years 15 years More than 20 years

Level of treatment: ____________________________

Sludge management/treatment: Yes No

If yes, what type? ____

Design capacity (m^3/day): ______________________

Available capacity (m^3/day): ___________________

Total inflow (m^3/day): _________________________

Average inflow (m^3/year): _____________________

Total discharge (m^3/day): ______________________

Maximum discharge (m^3/day): _________________

Average discharge (m^3/day): ___________________

Average daily BOD_5 influent (mg/L): __________________

Average daily BOD_5 effluent (mg/L): __________________

Final effluent disinfection is: _____________________

Final effluent point of discharge: _________________

Discharge criteria: _______________________________

Expected growth: ________________

Average yearly operation cost ($/year): ______________

Capital cost investment for last 5 years ($): ____________

Proposed capital expenditure for expansion in the next: 5 years: $_________
10 years: $_________
20 years: $_________

Future major repairs or replacement in: Less than 5 years 5 to 10 years
11 to 20 years More than 20 years

Do you have in-house as-built drawings for the system? Yes No

If no, what are possible sources? ____

If yes, in what format are they? ____

Comments

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5. Recirculating Sand Filter Questionnaire

Town, village, or rural area served: ____________________________

Total population served by this recirculating sand filter in 1999: ___________

Total land area served by this system in 1999: ______________

Owner: _______________________________________

Operating authority: ___________________________

Estimated original capital cost ($): _______________

Age range (years): from _______ to _________

Life expectancy (years): 5 years 10 years 15 years More than 20 years

Treatment process: ___________________________

Sludge management/treatment: Yes No

If yes, what type? ____

Design capacity (m^3/day): ______________________

Available capacity (m^3/day): ___________________

Total inflow (m^3/day): ________________________

Average inflow (m^3/year): _____________________

Total discharge (m^3/day): ______________________

Maximum discharge (m^3/day): _________________

Average discharge (m^3/day): ___________________

Average daily BOD_5 influent (mg/L): _________________

Average daily BOD_5 effluent (mg/L): _________________

Final effluent disinfection is: _____________________

Final effluent point of discharge: _________________

Discharge criteria: _______________________________

Expected growth: _________________

Average yearly operation cost ($/year): _______________

Capital cost investment for last 5 years ($): ____________

Proposed capital expenditure for expansion in the next: 5 years: $_________
10 years: $_________
20 years: $_________

Future major repairs or replacement in: Less than 5 years 5 to 10 years
11 to 20 years More than 20 years

Do you have in-house as-built drawings for the system? Yes No

If no, what are possible sources? ____

If yes, in what format are they? ____

Comments

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If the recirculating sand filter is accompanied by a communal septic system, treatment plant or lagoon, complete and attach the corresponding questionnaire.

6. Communal Septic System Questionnaire

General

Town, village, subdivision, or rural area served: ______________________________

Total population serviced by this communal septic system in 1999: _______________

Total land area served by this system in 1999: _______________

Owner: ____

Operating authority: _____________________________

Estimated original capital cost ($): ________________

Total tank volume (m^3): ________________

Age range (years): from _______ to ________

Life expectancy (years): 5 years 10 years 15 years More than 20 years

Treatment process: _____________________________

Sludge management/treatment: Yes No

If yes, what type? ____

Design capacity (m^3/day): _______________________

Available capacity (m^3/day): ____________________

Total inflow (m^3/day): __________________________

Average inflow (m^3/year): _______________________

Total discharge (m^3/day): _______________________

Maximum discharge (m^3/day): ___________________

Average discharge (m^3/day): ____________________

Average daily BOD_5 influent (mg/L): __________________

Average daily BOD_5 effluent (mg/L): __________________

Expected growth: __________________

Average yearly operation cost ($/year): _________________

Capital cost investment for last 5 years ($): _____________

Proposed capital expenditure for expansion in the next: 5 years: $_________
10 years: $_________
20 years: $_________

Future major repairs or replacement in: Less than 5 years 5 to 10 years
11 to 20 years More than 20 years

Do you have in-house as-built drawings for the system? Yes No

If no, what are possible sources? ____

If yes, in what format are they? ____

Comments____

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7. Private Septic System Questionnaire

Town, village, or rural area served: ______________________________

Total population serviced by private septic systems in 1999: ______________

Total land area served by private septic systems in 1999: _______________

Comments____

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8. No Treatment Questionnaire

Town, village, or rural area: ___________________________

Total population in 1999 with no wastewater treatment: _____________

Total land area with no wastewater treatment in 1999: _____________

Receiving water body: _____________________________________

Number of discharge points: ____________

Comments

9. Other Wastewater Treatment Systems Questionnaire

Town, village, or rural area served: ____________________________

Type of wastewater treatment system: ___________________

Total population served by this type of wastewater treatment in 1999: ______________

Owner: _______________________________________

Operating authority: ___________________________

Estimated original capital cost ($): _______________

Age range (years): from _______ to ________

Life expectancy (years): 5 years 10 years 15 years More than 20 years

Treatment process: ___________________________

Sludge management/treatment: Yes No

If yes, what type? ____

Design capacity (m^3/day): _____________________

Available capacity (m^3/day): ___________________

Total inflow (m^3/day): ________________________

Average inflow (m^3/year): _____________________

Total discharge (m^3/day): ______________________

Maximum discharge (m^3/day): _________________

Average discharge (m^3/day): ___________________

Average daily BOD_5 influent (mg/L): _________________

Average daily BOD_5 effluent (mg/L): _________________

Final effluent disinfection is: _____________________

Final effluent point of discharge: _________________

Discharge criteria: ______________________________

Expected growth: _________________

Average yearly operation cost ($/year): ________________

Capital cost investment for last 5 years ($): _____________

Proposed capital expenditure for expansion in the next: 5 years: $_________
10 years: $_________
20 years: $_________

Future major repairs or replacement in: Less than 5 years 5 to 10 years 11 to 20 years More than 20 years

Do you have in-house as-built drawings for the system? Yes No

If no, what are possible sources? ____

If yes, in what format are they? ____

Comments

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Groundwater Recharge Potential Classes Based on Surficial Geology

Appendix B: EOWRMS Groundwater Recharge Potential Classes Based on Surficial Geology.

CENOZOIC - QUATERNARY

POST-CHAMPLAIN SEA DEPOSITS

Category		Description	Recharge Potential
ORGANIC DEPOSITS	7	Mainly muck and peat in bogs, fens, swamps, and poorly drained areas	Low
Eolian Dunes	13	Medium to medium fine sand	High
ALLUVIAL DEPOSITS: stratified sand, silt, minor gravel, disseminated organic matter, and marl	6a	Silty sand, silt, sand and clay; deposits of present floodplains and of alluvial fans in areas of low relief	Medium
	6b	Medium grained stratified sand with some silt; in the form of fluvial terraces and channels cut in marine clay, and bars and spits within abandoned channels	Medium
	6ab	Combination of above	Medium
CHAMPLAIN SEA SEDIMENTS			
NEARSHORE SEDIMENTS: gravel, sand and coarser material, generally well sorted	5a	Gravel, sand and boulders; beaches commonly fossiliferous; nature of sediment controlled by underlying material (gravel, sand and boulders where developed from till and glaciofluvial deposits; slabs and shingles where developed from sedimentary bedrock)	High
	5b	Fine-to medium-grained sand, calcareous and commonly fossiliferous; nearshore sand generally occurs as a sheet or as bars or spits associated with glaciofluvial materials	High
	5ab	Combination of above	High
DELTAIC AND ESTUARINE DEPOSITS:	4	medium-to fine-grained sand, in some places fossiliferous; lies outside abandoned channels; most common deposit is a combined strip delta-sand plain that developed as water levels fell	High
OFFSHORE MARINE DEPOSITS:	3	Clay, silty clay and silt, commonly calcareous and fossiliferous ; locally overlain by thin sands. Upper parts are generally mottled or laminated reddish brown and bluish grey and may contain lenses and pockets of sand, but at depth the clay is uniform and blue-grey	Low
	3a	Clay and silt underlying erosional terraces; upper part of marine deposits removed to variable depths by fluvial erosion so in places clay is uniform blue-	Low

Category		Description	Recharge Potential
		grey; unit includes lenses, bars and channel fills to sand and pockets of nonmarine silt that were formed during terrace (or channel) cutting	
GLACIOFLUVIAL DEPOSITS:			
ICE CONTACT STRATIFIED DRIFT: Gravel and sand, poorly to well sorted and bedded, mainly coarse-to medium-grained with numerous cobbles, boulders and lenses of till;	2a	Gravel and sand in the form of outwash plains, valley trains, kame terraces, outwash fans and ridges; surface commonly pitted by closed depressions; occurs at or above marine limit (>200 to 220m)	High
	2b	"Subaqueous outwash" sediments: sand, gravel, boulder gravel, and minor diamicton; locally fossiliferous; commonly capped by a discontinuous fossiliferous gravel and sand < 2 m thick; interpreted as ice contact stratified drift deposited below wave base in the Champlain Sea	High
	2c	Kame moraine sand gravel and minor till	Medium
TILL: Sandy and silty compact diamicton, grey at depth but brown where oxidized; calcareous where derived from sedimentary rocks and not leached; consists dominantly of lodgment till. In areas that lie below marine limit (approx. 198 m (650 ft) a.s.l.) it is overlain by a discontinuous lag consisting of gravel, sand and boulders.	1a	Till, plain; local relief <5m (<15ft).	Low
	1b	Till, drumlinized.	Medium
	1c	Till, hummocky to rolling; local relief 5 to 25m (15 to 80 ft)	Medium
	1 abc	Combination of above	Medium
	1d	Thin till over bedrock	Medium
GLACIOLACUSTRINE DEPOSITS: Stratified to massive, clay to gravel deposited in deltaic, littoral, nearshore, and deep water environments of glacial lakes			
	12:	Beach, bar, and related deposits	High
	11	Gravel, gravelly sand, sand, and minor silt and clay	High
	10	Clay, silty clay, silt, and minor sand	Low
	10a	Clay ridges	Low
	10b	Thin and/or discontinuous clay and silt	Medium
BEDROCK:			
PALEOZOIC	Pa	Limestone, dolomite, sandstone, and locally shale; relatively flat lying; mainly occurring as bare, tabular outcrops; includes areas thinly veneered by unconsolidated Quaternary sediments up to 1 m (3	Medium

Category		Description	Recharge Potential
		ft) thick	
PRECAMBRIAN AND CRETACEOUS	Pr	Intrusive and metamorphic rocks (Precambrian); mainly bare, hummocky, rolling or hilly rock knob upland; includes areas thinly veneered by unconsolidated sediments up to 2 m. thick	Low

SURFACE FEATURES – These features should be designated above.

- Landslide area showing location of headscarp and general trend of slump ridges. Ridges generally consist of clay with overlying or admixed sand.
- Dunes (largely stabilized) and sand deposits generally reworked by the wind.
- Gullies, ravines; shown where undercutting of steep slopes could cause slumping and / or sliding.
- Areas of deformed marine sediments.

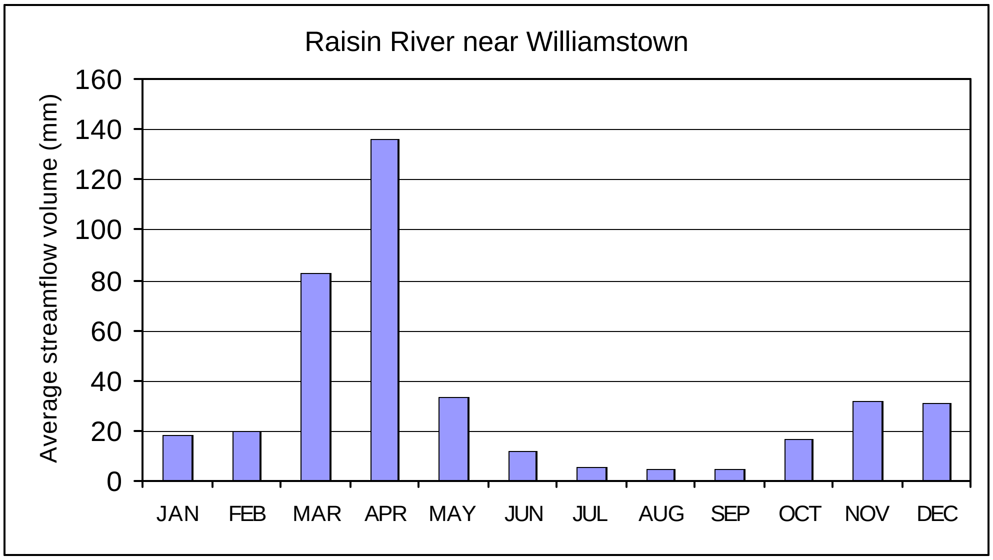
APPENDIX C

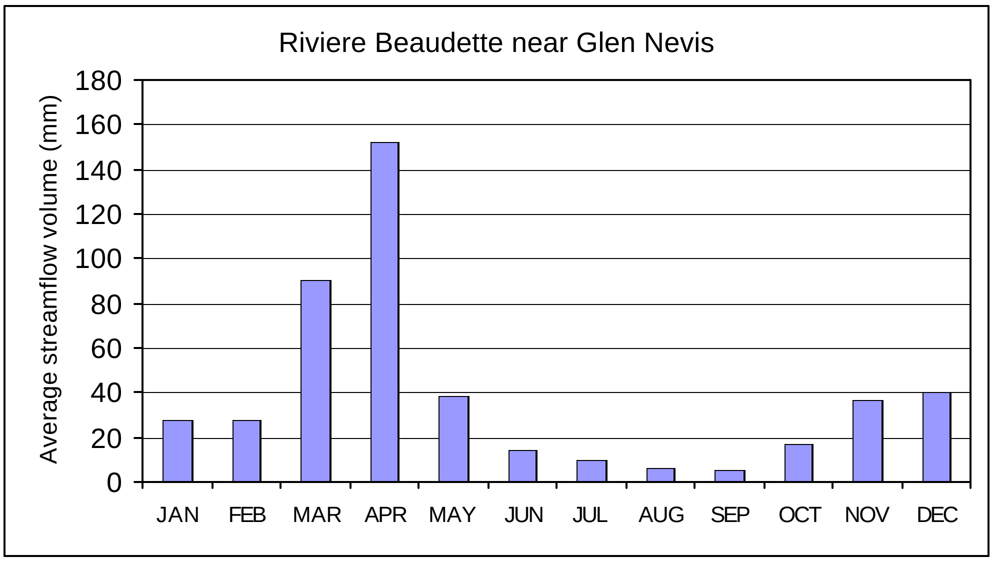
STREAMFLOW STATISTICS

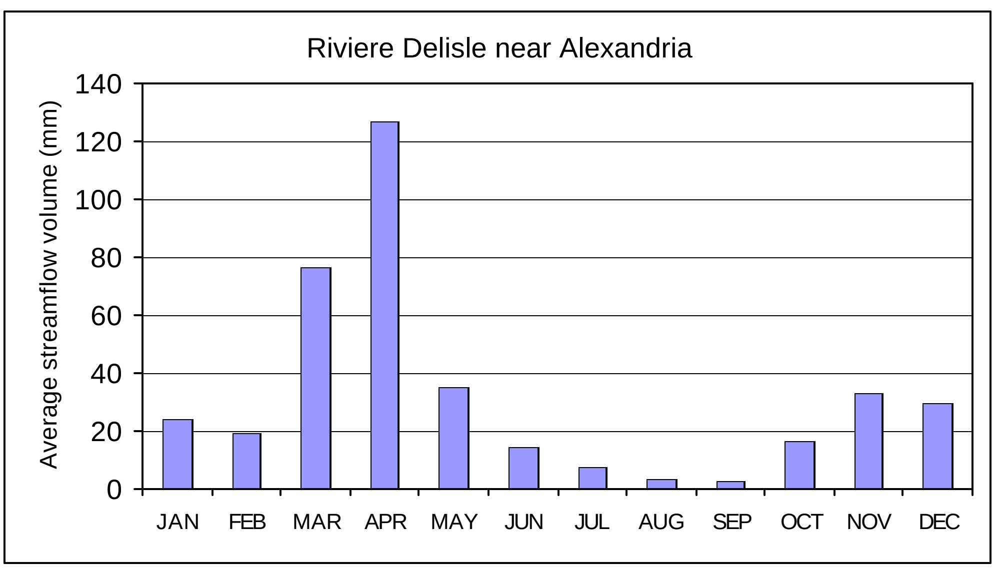
MONTHLY AVERAGE STREAMFLOW VOLUMES

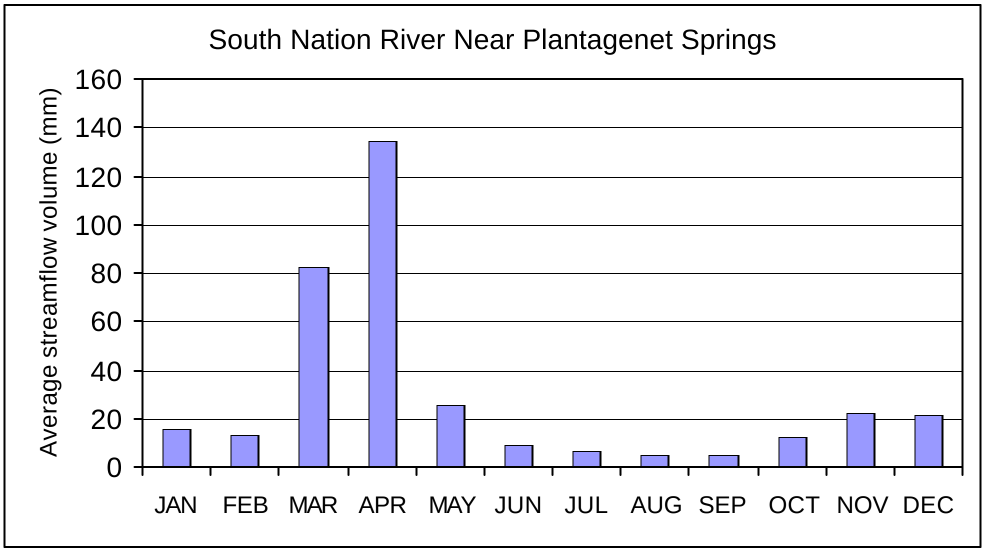
Monthly average streamflow volumes
As depth (mm) over the drainage area

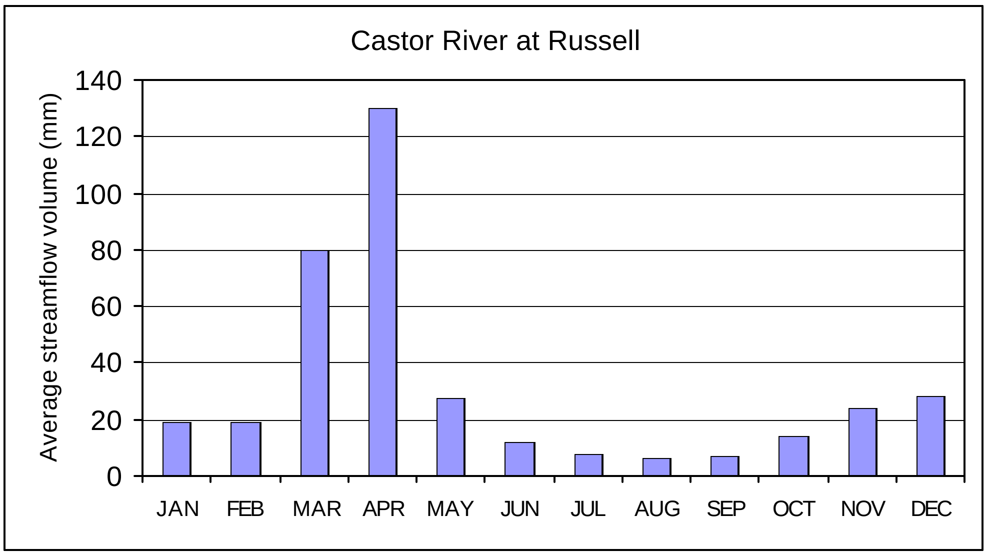
Station		Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
02LB005	South Nation at Plantagenet	15.9	13.5	82.4	134.6	25.8	9.5	6.3	5.0	4.5	12.2	22.1	21.7	355.4
02LB006	Castor River at Russell	18.8	19.4	80.2	130.1	27.5	11.8	7.9	6.6	6.7	14.4	23.7	28.6	396.9
02LB007	South Nation at Spencerville	24.9	22.1	94.5	121.1	32.8	12.1	6.4	3.6	3.3	10.5	25.2	29.9	386.1
02LB008	Bear Brook near Bourget	17.2	23.9	84.4	144.6	27.7	13.7	5.9	7.6	9.0	20.0	30.8	28.7	426.8
02LB009	South Nation at Chesterville	22.8	16.5	79.2	135.0	40.5	29.5	10.8	2.0	0.4	10.3	30.7	25.7	497.6
02LB012	East Branch Scotch River at St. Isidore	25.6	17.7	79.2	129.9	28.7	12.7	5.7	6.0	7.5	18.8	30.6	29.0	432.5
02LB013	South Nation at Casselman	14.2	24.1	99.7	114.9	28.2	7.2	3.9	5.7	8.4	19.0	25.8	28.3	384.1
02LB016	Little Castor near Embrun	18.5	35.1	73.6	137.0	23.4	11.4	8.5	8.8	12.4	14.8	28.9	45.4	417.9
02LB017	North branch South Nation near Heckston	23.7	27.8	81.8	117.5	33.5	11.0	3.7	4.6	7.4	14.9	30.9	34.7	392.0
02LB018	West branch Scotch R. near St. Isidore	15.1	49.7	104.3	107.1	28.3	16.1	7.0	12.6	22.6	24.2	46.8	62.5	472.2
02LB019	South Indian Creek near Limoges	11.8	42.1	94.9	110.5	35.7	27.1	3.9	19.7	14.8	18.8	34.0	35.3	462.5
02LB020	South Castor at Kenmore	17.9	23.0	74.3	109.9	27.3	10.4	5.9	6.8	9.6	15.6	23.7	28.8	356.0
02LB022	Payne R. near Berwick	20.8	26.1	81.2	104.9	24.5	10.1	5.8	4.2	6.5	17.4	30.0	30.5	361.0
02LB101	Bear Brook at Carlsbad Springs	39.6	23.4	202.3	154.3	21.8	6.8	4.5	4.5	16.3	47.0	37.5	28.4	543.4
02MC001	Raisin River near Williamstown	18.7	20.1	83.1	136.5	33.1	12.3	5.9	4.6	5.1	16.8	31.5	31.4	400.4
02MC026	Riviere Beaudette near Glen Nevis	27.9	27.3	90.7	152.6	38.2	14.4	9.7	6.0	5.6	17.3	37.0	40.2	462.9
02MC028	Riviere Delisle near Alexandria	24.1	19.5	76.5	127.2	35.1	14.3	7.8	3.8	3.0	16.6	32.8	29.5	387.7

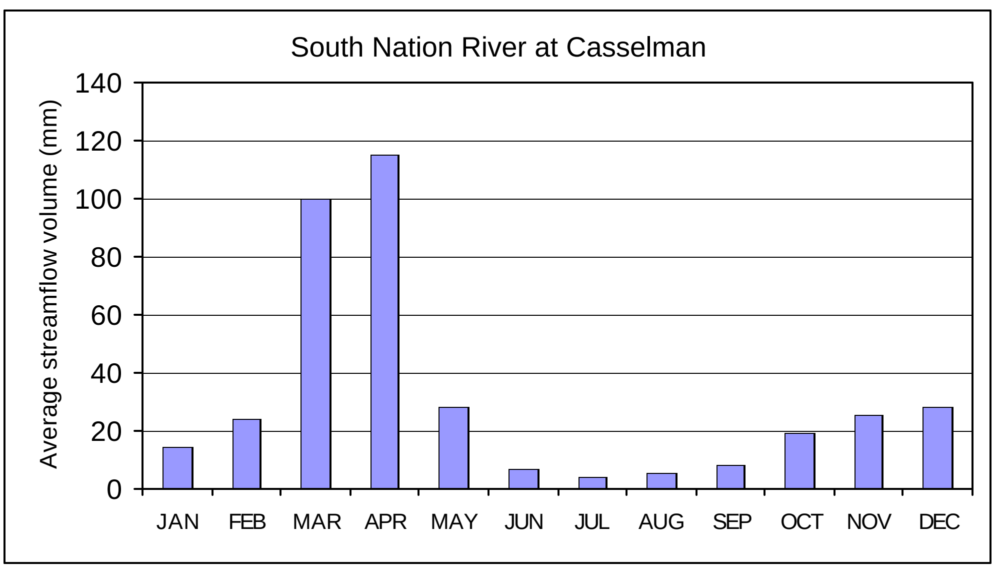

Raisin River near Williamstown
Average streamflow volume (mm)
160
140
120
100
80
60
40
20
0
JAN
FEB
MAR
APR
MAY
JUN
JUL
AUG
SEP
OCT
NOV
DEC

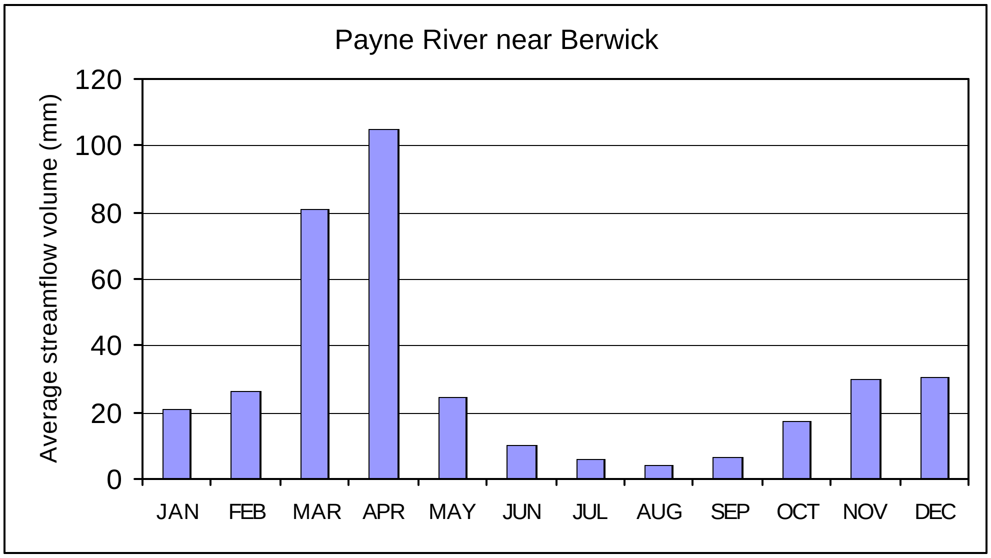

Riviere Beaudette near Glen Nevis
Average streamflow volume (mm)
180
160
140
120
100
80
60
40
20
0
JAN
FEB
MAR
APR
MAY
JUN
JUL
AUG
SEP
OCT
NOV
DEC

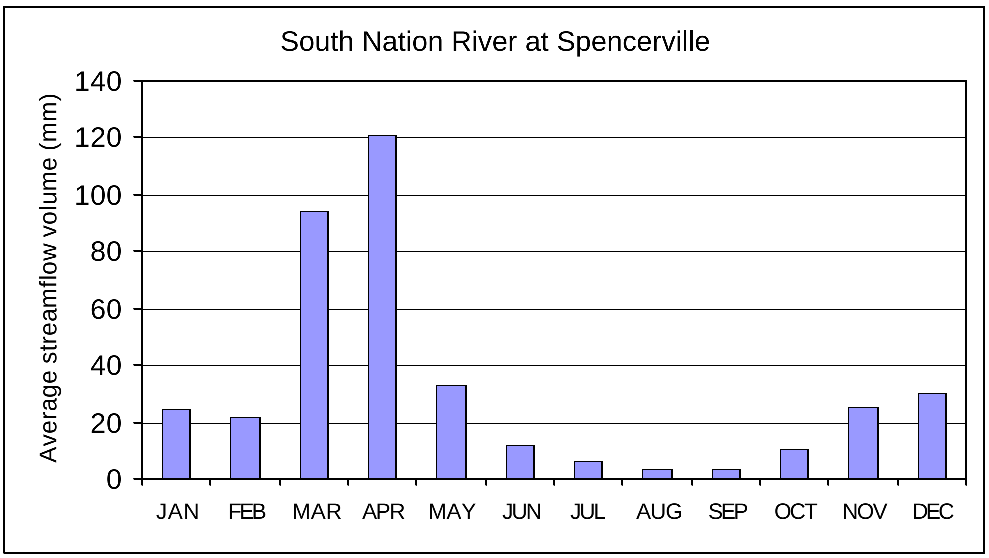

Riviere Delisle near Alexandria
Average streamflow volume (mm)
140
120
100
80
60
40
20
0
JAN
FEB
MAR
APR
MAY
JUN
JUL
AUG
SEP
OCT
NOV
DEC


South Nation River Near Plantagenet Springs
Average streamflow volume (mm)
160
140
120
100
80
60
40
20
0
JAN
FEB
MAR
APR
MAY
JUN
JUL
AUG
SEP
OCT
NOV
DEC


Castor River at Russell
Average streamflow volume (mm)
140
120
100
80
60
40
20
0
JAN
FEB
MAR
APR
MAY
JUN
JUL
AUG
SEP
OCT
NOV
DEC


South Nation River at Casselman
Average streamflow volume (mm)
140
120
100
80
60
40
20
0
JAN
FEB
MAR
APR
MAY
JUN
JUL
AUG
SEP
OCT
NOV
DEC


Payne River near Berwick
Average streamflow volume (mm)
0
20
40
60
80
100
120
JAN
FEB
MAR
APR
MAY
JUN
JUL
AUG
SEP
OCT
NOV
DEC


South Nation River at Spencerville
Average streamflow volume (mm)
0
20
40
60
80
100
120
140
JAN
FEB
MAR
APR
MAY
JUN
JUL
AUG
SEP
OCT
NOV
DEC

LOW-FLOW STATISTICAL ANALYSIS:
Results for selected stations

South Nation River near Plantagenet Springs 02LB005
Based on Years 1950 to 1998

LOW FLOW FREQUENCY ANALYSIS RESULTS

STATISTICS SUMMARY

STAT	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
MEAN	7.461	6.746	18.369	37.908	11.172	4.885	3.009	2.43	2.828	6.459	13.326	10.444
STD.	7.447	5.285	20.487	23.12	8.587	4.663	3.563	2.734	2.851	7.311	10.868	8.371
SKEW	2.257	1.493	1.873	1.007	2.027	2.595	4.812	3.244	2.465	1.794	0.925	1.05

FREQUENCY ANALYSIS SUMMARY

METHOD OF MOMENTS

T(YEARS	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
2	5.039	5.396	12.306	33.651	8.518	3.288	1.721	1.444	1.868	4.348	11.478	8.843
10	1.175	1.427	-0.511	11.899	3.536	1.224	1.078	0.536	0.528	-0.402	0.883	1.116
20	0.863	0.949	-1.771	8.475	3.084	1.083	1.059	0.489	0.43	-0.888	-0.862	-0.071
RMS	0.074	0.016	1.239	0.056	0.028	0.069	0.078	0.107	0.091	1.087	0.729	0.588

METHOD OF LOWEST OBSERVED FLOW

T(YEARS	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
2	5.032	5.407	11.357	33.015	8.621	3.384	1.716	1.439	1.855	3.931	10.586	8.353
10	1.227	1.395	2.223	12.892	3.291	0.885	0.636	0.551	0.569	0.819	2.307	1.921
20	0.924	0.907	1.591	10.012	2.771	0.677	0.585	0.506	0.48	0.613	1.296	1.127
MHrms	0.067	0.015	0.034	0.029	0.038	0.099	0.101	0.102	0.073	0.041	0.082	0.03

Castor River at Russell 02LB006
LOW FLOW FREQUENCY ANALYSIS RESULTS

STAT SUMMARY

STAT	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
MEAN	1.007	0.881	2.323	5.17	1.643	0.677	0.388	0.303	0.34	1.035	1.818	1.662
STD.	0.813	0.654	2.371	3.211	1.119	0.628	0.424	0.218	0.224	1.081	1.378	1.065
SKEW	2.013	1.619	1.666	1.329	1.585	3.195	4.178	2.014	2.193	2.061	1.117	0.394

FREQUENCY ANALYSIS RESULTS

METHOD OF MOMENTS

T(YEARS	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
2	0.757	0.704	1.67	4.42	1.345	0.451	0.233	0.236	0.268	0.698	1.54	1.588
10	0.282	0.242	0.031	1.812	0.541	0.238	0.137	0.109	0.148	0.081	0.305	0.32
20	0.239	0.19	-0.148	1.468	0.448	0.227	0.133	0.097	0.138	0.026	0.122	0.043
RMS	0.064	0.072	0.887	0.075	0.055	0.081	0.073	0.05	0.015	0.509	0.329	0.492

METHOD OF LOWEST OBSERVED FLOW

T(YEARS	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
2	0.773	0.681	1.533	4.297	1.313	0.464	0.235	0.241	0.268	0.669	1.437	1.456
10	0.242	0.295	0.371	2.043	0.609	0.179	0.099	0.096	0.149	0.172	0.49	0.48
20	0.187	0.259	0.283	1.792	0.539	0.159	0.092	0.08	0.139	0.136	0.387	0.332
MHrms	0.058	0.019	0.027	0.033	0.024	0.047	0.143	0.07	0.014	0.05	0.09	0.04

South Nation River at Spencerville 02LB007
LOW FLOW FREQUENCY ANALYSIS RESULTS
From Years 1969-1998
STAT SUMMARY

STAT	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
MEAN	0.903	0.921	2.045	3.512	0.996	0.36	0.139	0.131	0.12	0.41	1.225	1.158
STD.	0.704	0.678	1.608	2.319	0.718	0.351	0.191	0.357	0.236	0.635	0.919	0.86
SKEW	2.255	1.439	1.147	1.477	1.243	0.83	1.845	4.724	2.957	2.526	0.335	1.223

FREQUENCY ANALYSIS RESULTS

METHOD OF MOMENTS

T(YEARS	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
2	0.674	0.753	1.712	2.924	0.837	0.306	0.083	0.002	0.037	0.195	1.172	0.97
10	0.308	0.23	0.291	1.169	0.23	-0.049	-0.038	-0.065	-0.052	-0.095	0.057	0.237
20	0.279	0.165	0.084	0.956	0.146	-0.111	-0.05	-0.067	-0.057	-0.116	-0.194	0.135
RMS	0.033	0.054	0.365	0.057	0.165	0.09	0.121	0.326	0.43	0.328	0.391	0.092

METHOD OF LOWEST OBSERVED FLOW

T(YEARS	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
2	0.688	0.76	1.667	2.93	0.857	0.257	0.07	0.023	0.037	0.178	1.038	0.975
10	0.272	0.217	0.367	1.156	0.199	0.037	0.01	0.003	0.003	0.011	0.216	0.228
20	0.233	0.146	0.196	0.938	0.099	0.015	0.007	0.003	0.002	0.004	0.094	0.122
MHrms	0.051	0.048	0.116	0.055	0.079	0.326	0.156	0.049	0.069	0.133	0.385	0.084

Bear Brook near Bourget 02LB008
LOW FLOW FREQUENCY ANALYSIS RESULTS
From Years 1977-1995
STAT SUMMARY

STAT	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
MEAN	0.856	0.836	1.991	4.25	1.516	0.572	0.311	0.3	0.332	1.243	2.4	1.509
STD.	0.583	0.577	2.619	2.626	1.001	0.573	0.226	0.223	0.226	1.041	1.54	0.96
SKEW	2.447	1.33	2.748	1.545	1.619	3.056	2.631	2.425	2.224	0.994	0.405	1.05

FREQUENCY ANALYSIS RESULTS

METHOD OF MOMENTS

T(YEARS	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
2	0.661	0.701	1.079	3.562	1.245	0.367	0.233	0.225	0.258	1.054	2.289	1.325
10	0.385	0.233	-0.002	1.638	0.537	0.161	0.134	0.118	0.139	0.069	0.462	0.438
20	0.364	0.171	-0.071	1.413	0.458	0.149	0.128	0.11	0.129	-0.087	0.065	0.302
RMS	0.111	0.036	0.936	0.063	0.018	0.152	0.052	0.048	0.04	0.703	0.287	0.083

METHOD OF LOWEST OBSERVED FLOW

T(YEARS	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
2	0.698	0.705	1.046	3.59	1.247	0.363	0.234	0.224	0.265	0.927	2.185	1.27
10	0.288	0.226	0.199	1.58	0.533	0.207	0.129	0.122	0.123	0.301	0.575	0.529
20	0.242	0.162	0.156	1.333	0.452	0.201	0.121	0.114	0.109	0.242	0.284	0.438
MHrms	0.013	0.032	0.152	0.068	0.018	0.067	0.063	0.039	0.033	0.052	0.172	0.047

North Branch of the South Nation River near Heckston 02LB0017Q
LOW FLOW FREQUENCY ANALYSIS RESULTS
From Years 1978-1996
STAT SUMMARY

STAT	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
MEAN	0.21	0.212	0.518	0.968	0.262	0.082	0.021	0.027	0.039	0.159	0.381	0.309
STD.	0.176	0.155	0.5	0.556	0.162	0.082	0.033	0.066	0.064	0.281	0.25	0.213
SKEW	2.664	0.636	1.368	0.892	1.279	1.286	3.298	3.923	2.003	2.953	0.032	0.678

FREQUENCY ANALYSIS RESULTS

METHOD OF MOMENTS

T(YEARS	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
2	0.149	0.194	0.398	0.876	0.225	0.063	0.009	0.003	0.02	0.059	0.381	0.282
10	0.074	0.025	-0.001	0.326	0.09	-0.004	-0.001	-0.014	-0.018	-0.046	0.053	0.052
20	0.069	-0.008	-0.052	0.234	0.072	-0.014	-0.002	-0.014	-0.021	-0.053	-0.033	0.009
RMS	0.292	0.156	0.261	0.048	0.084	0.198	0.869	1.531	1.842	0.789	0.231	0.397

METHOD OF LOWEST OBSERVED FLOW

T(YEARS	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
2	0.165	0.184	0.375	0.861	0.216	0.055	0.009	0.006	0.016	0.057	0.352	0.258
10	0.033	0.037	0.044	0.348	0.107	0.012	0.001	0	0.001	0.002	0.077	0.087
20	0.017	0.013	0.009	0.27	0.096	0.009	0.001	0	0	0	0.023	0.065
MHrms	0.155	0.246	0.251	0.03	0.018	0.158	0.035	0.157	1.29	0.339	0.331	0.105

South Castor River at Kenmore 02LB020
LOW FLOW FREQUENCY ANALYSIS RESULTS
From Years 1979-1996
STAT SUMMARY

STAT	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
MEAN	0.345	0.423	1.017	2.013	0.672	0.277	0.18	0.156	0.163	0.471	0.849	0.673
STD.	0.167	0.347	1.437	1.338	0.359	0.128	0.096	0.125	0.109	0.548	0.506	0.444
SKEW	0.399	1.151	3.101	2.157	1.765	1.282	1.852	2.746	2.307	2.357	0.343	0.433

FREQUENCY ANALYSIS RESULTS

METHOD OF MOMENTS

T(YEARS	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
2	0.333	0.351	0.503	1.587	0.569	0.248	0.152	0.113	0.127	0.29	0.819	0.639
10	0.134	0.045	-0.005	0.858	0.333	0.141	0.092	0.061	0.072	0.019	0.207	0.116
20	0.091	0	-0.032	0.797	0.308	0.127	0.086	0.058	0.068	-0.002	0.069	0.004
RMS	0.078	0.625	0.947	0.081	0.062	0.037	0.154	0.094	0.046	0.772	0.153	0.58

METHOD OF LOWEST OBSERVED FLOW

T(YEARS	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
2	0.333	0.338	0.493	1.643	0.554	0.243	0.166	0.115	0.125	0.271	0.783	0.586
10	0.134	0.065	0.125	0.723	0.372	0.15	0.068	0.051	0.081	0.116	0.243	0.18
20	0.09	0.031	0.11	0.623	0.357	0.139	0.05	0.046	0.079	0.109	0.143	0.119
MHrms	0.076	0.098	0.018	0.102	0.017	0.016	0.128	0.129	0.016	0.016	0.121	0.046

Payne River near Berwick 02LB022
LOW FLOW FREQUENCY ANALYSIS RESULTS
From Years 1977-1996
STAT SUMMARY

STAT	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
MEAN	0.251	0.295	0.64	1.263	0.368	0.128	0.083	0.059	0.056	0.312	0.689	0.442
STD.	0.283	0.246	0.725	0.737	0.244	0.103	0.114	0.108	0.076	0.358	0.465	0.374
SKEW	3.271	1.102	2.288	1.494	1.358	1.303	3.26	3.461	2.517	1.417	0.025	0.985

FREQUENCY ANALYSIS RESULTS

METHOD OF MOMENTS

T(YEARS	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
2	0.149	0.245	0.403	1.075	0.31	0.105	0.041	0.02	0.03	0.224	0.691	0.374
10	0.057	0.023	0.033	0.522	0.114	0.02	0.004	-0.013	-0.005	-0.054	0.079	0.019
20	0.052	-0.01	0.003	0.455	0.088	0.009	0.002	-0.015	-0.007	-0.089	-0.081	-0.037
RMS	0.121	0.172	0.353	0.062	0.049	0.154	0.423	0.29	0.462	0.242	0.325	0.357

METHOD OF LOWEST OBSERVED FLOW

T(YEARS	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
2	0.153	0.234	0.4	1.115	0.309	0.102	0.041	0.02	0.029	0.192	0.623	0.341
10	0.036	0.043	0.043	0.454	0.115	0.025	0.008	0.003	0.004	0.023	0.139	0.077
20	0.028	0.02	0.015	0.356	0.09	0.015	0.007	0.003	0.003	0.011	0.053	0.047
MHrms	0.111	0.107	0.087	0.059	0.048	0.145	0.144	0.157	0.192	0.209	0.275	0.113

Raisin River near Williamstown 02MC001
LOW FLOW FREQUENCY ANALYSIS RESULTS
From Years 1961-1998
STAT SUMMARY

STAT	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
MEAN	1.125	1.141	2.56	5.59	1.568	0.595	0.242	0.158	0.186	0.817	1.855	1.507
STD.	0.802	0.782	2.353	3.19	1.094	0.571	0.45	0.306	0.218	0.942	1.523	1.085
SKEW	1.695	0.998	1.728	1.164	1.325	1.501	4.548	4.29	2.188	1.544	0.595	0.842

FREQUENCY ANALYSIS RESULTS

METHOD OF MOMENTS

T(YEARS	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
2	0.902	0.998	1.896	4.922	1.313	0.449	0.078	0.046	0.116	0.571	1.689	1.339
10	0.355	0.26	0.318	2.125	0.423	0.022	-0.012	-0.021	-0.001	-0.12	-0.004	0.244
20	0.296	0.144	0.151	1.721	0.306	-0.029	-0.014	-0.023	-0.011	-0.201	-0.337	0.055
RMS	0.163	0.039	0.17	0.035	0.077	0.464	0.267	0.191	0.294	0.498	0.487	0.378

METHOD OF LOWEST OBSERVED FLOW

T(YEARS	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
2	0.958	0.991	1.871	4.908	1.273	0.422	0.08	0.049	0.111	0.499	1.486	1.274
10	0.254	0.271	0.371	2.146	0.496	0.078	0.011	0.005	0.014	0.056	0.282	0.339
20	0.152	0.161	0.22	1.754	0.408	0.046	0.009	0.004	0.008	0.024	0.128	0.206
MHrms	0.093	0.04	0.085	0.034	0.018	0.095	0.202	0.243	0.105	0.155	0.308	0.04

Riviere Beaudette near Glen Nevis 02MC026
LOW FLOW FREQUENCY ANALYSIS RESULTS
From Years 1984-1998
STAT SUMMARY

STAT	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
MEAN	0.493	0.554	1.103	2.037	0.62	0.189	0.086	0.102	0.097	0.346	0.75	0.608
STD.	0.231	0.352	1.272	1.161	0.432	0.11	0.073	0.226	0.12	0.396	0.528	0.434
SKEW	0.529	0.886	2.539	0.801	1.681	0.809	1.295	3.682	2.028	1.537	0.084	0.733

FREQUENCY ANALYSIS RESULTS

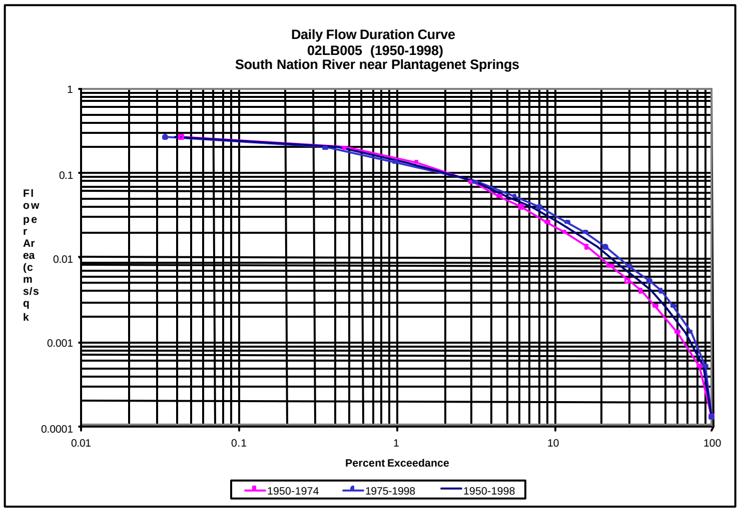
METHOD OF MOMENTS

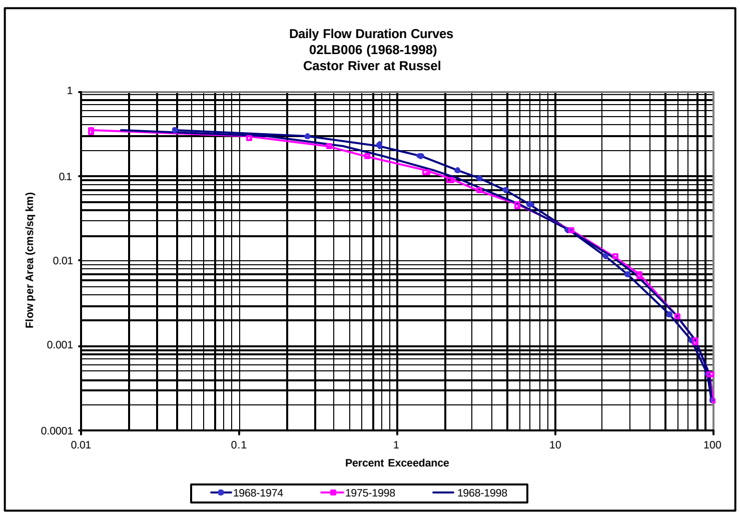
T(YEARS	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
2	0.471	0.497	0.671	1.865	0.5	0.173	0.069	0.02	0.06	0.243	0.746	0.549
10	0.208	0.148	0.093	0.674	0.204	0.06	0.009	-0.043	-0.01	-0.048	0.06	0.092
20	0.155	0.09	0.052	0.464	0.172	0.04	0.001	-0.046	-0.016	-0.083	-0.114	0.008
RMS	0.052	0.044	0.154	0.087	0.043	0.183	0.423	0.303	0.544	0.45	0.431	0.332

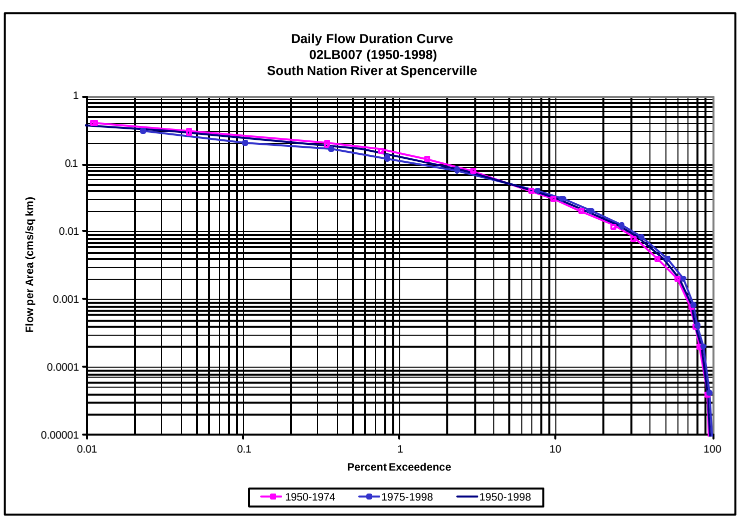
METHOD OF LOWEST OBSERVED FLOW

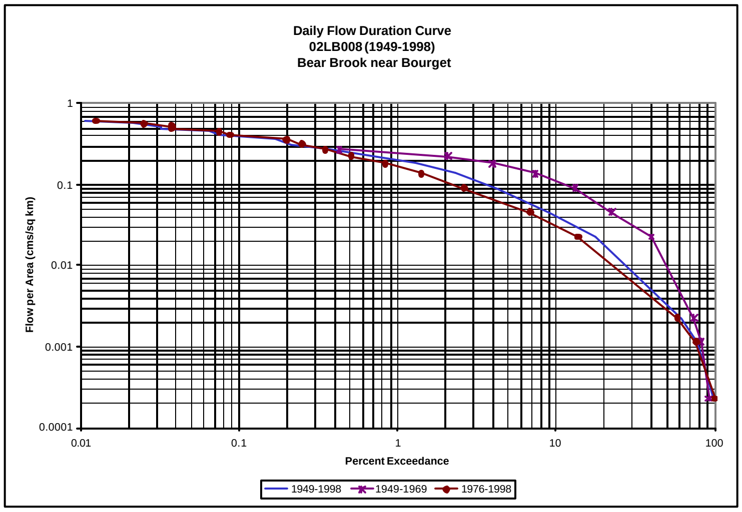
T(YEARS	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
2	0.476	0.498	0.671	1.798	0.502	0.178	0.064	0.026	0.055	0.217	0.67	0.506
10	0.204	0.147	0.1	0.772	0.2	0.053	0.018	0.003	0.004	0.014	0.133	0.155
20	0.145	0.087	0.06	0.623	0.166	0.028	0.014	0.003	0.001	-0.002	0.04	0.109
MHrms	0.044	0.041	0.134	0.087	0.047	0.133	0.092	0.295	0.094	0.221	0.291	0.075

FLOW DURATION CURVES

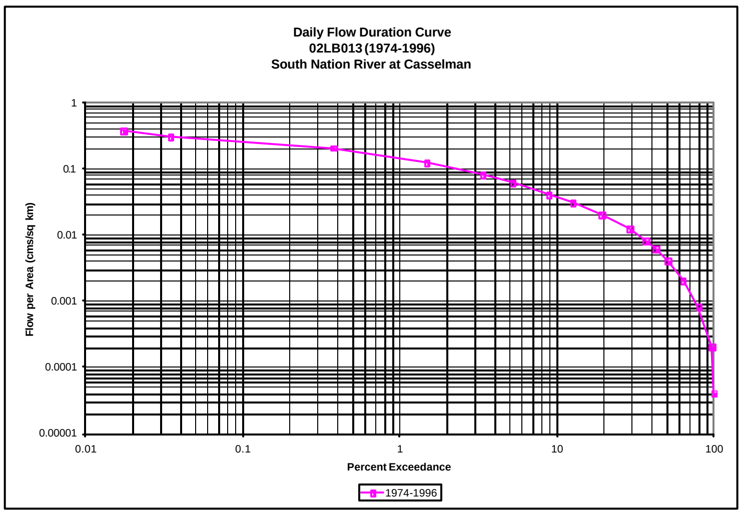




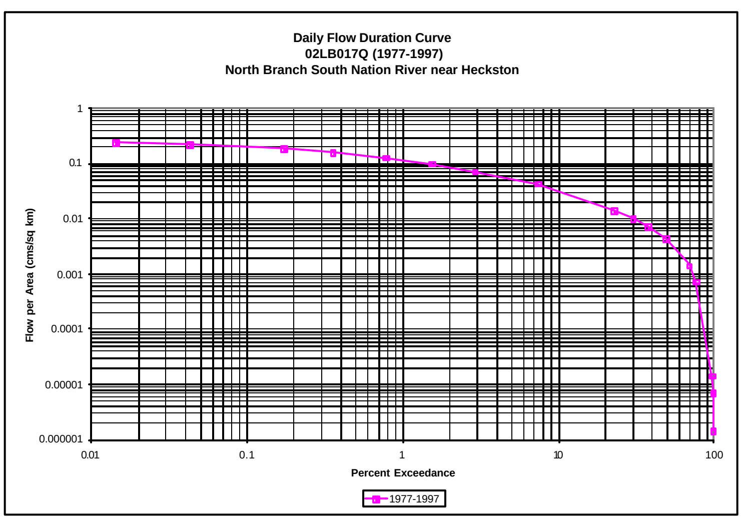




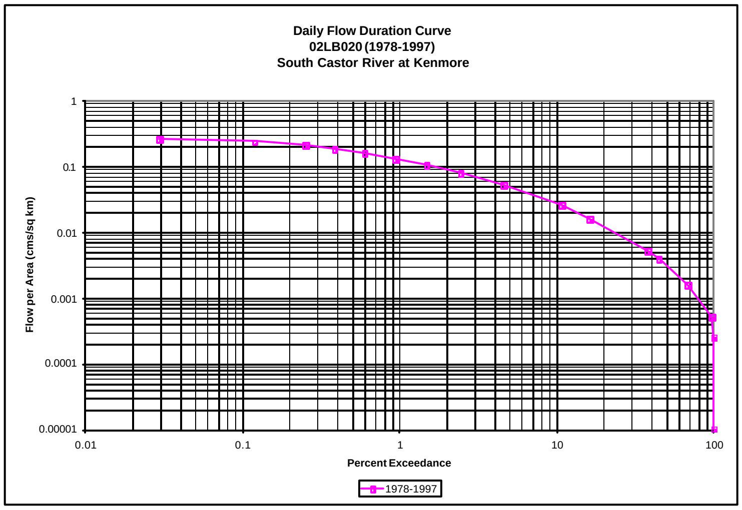
Daily Flow Duration Curve
02LB008 (1949-1998)
Bear Brook near Bourget
Flow per Area (cms/sq km)
1
0.1
0.01
0.001
0.0001
0.01
0.1
1
10
100
Percent Exceedance
1949-1998
1949-1969
1976-1998

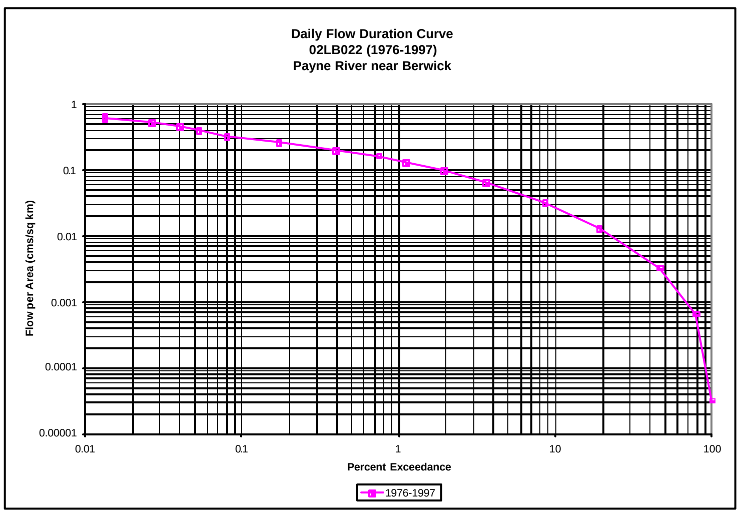


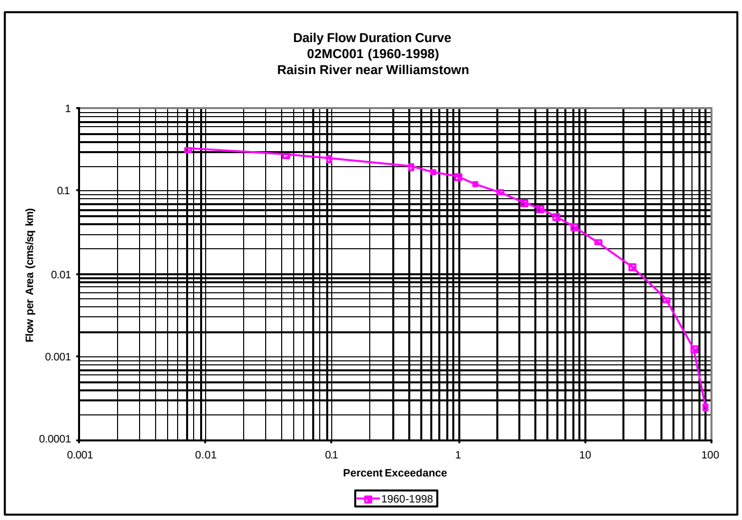
Daily Flow Duration Curve
02LB013 (1974-1996)
South Nation River at Casselman
Flow per Area (cms/sq km)
1
0.1
0.01
0.001
0.0001
0.00001
0.01
0.1
1
10
100
Percent Exceedance
1974-1996

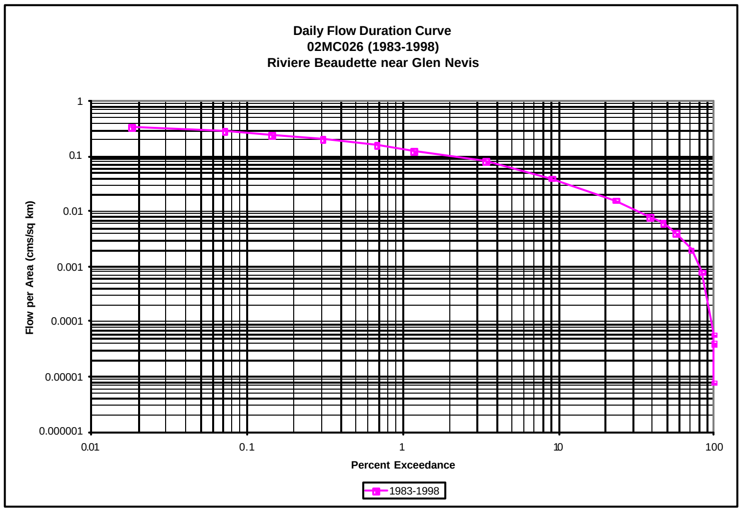


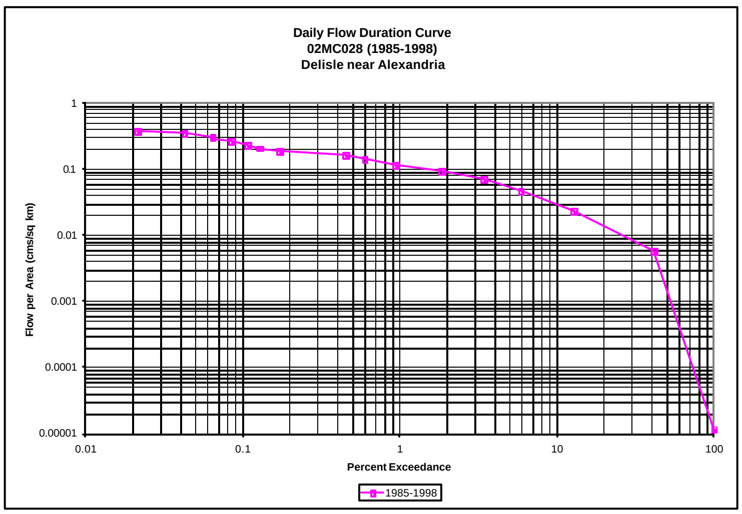
Daily Flow Duration Curve
02LB017Q (1977-1997)
North Branch South Nation River near Heckston
Flow per Area (cms/sq km)
1
0.1
0.01
0.001
0.0001
0.00001
0.000001
0.01
0.1
1
10
100
Percent Exceedance
1977-1997


Daily Flow Duration Curve
02LB020 (1978-1997)
South Castor River at Kenmore
Flow per Area (cms/sq km)
Percent Exceedance
1978-1997


Daily Flow Duration Curve
02LB022 (1976-1997)
Payne River near Berwick
Flow per Area (cms/sq km)
Percent Exceedance
1976-1997

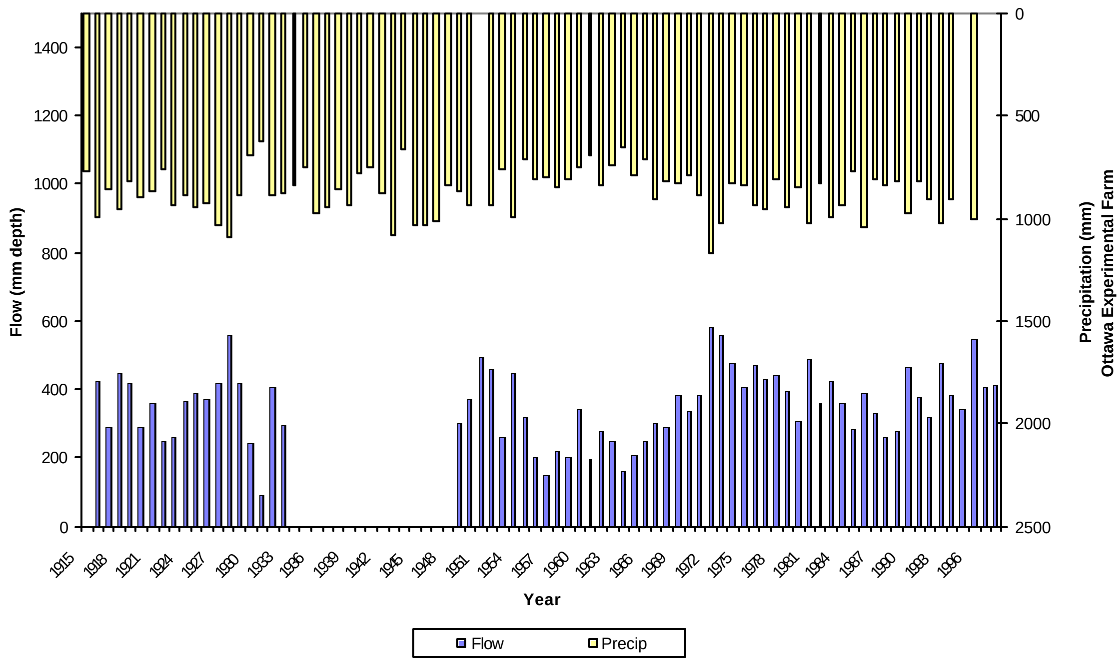

Daily Flow Duration Curve
02MC001 (1960-1998)
Raisin River near Williamstown
Flow per Area (cms/sq km)
Percent Exceedance
1960-1998


Daily Flow Duration Curve
02MC026 (1983-1998)
Riviere Beaudette near Glen Nevis
Flow per Area (cms/sq km)
1
0.1
0.01
0.001
0.0001
0.00001
0.000001
0.01
0.1
1
10
100
Percent Exceedance
1983-1998

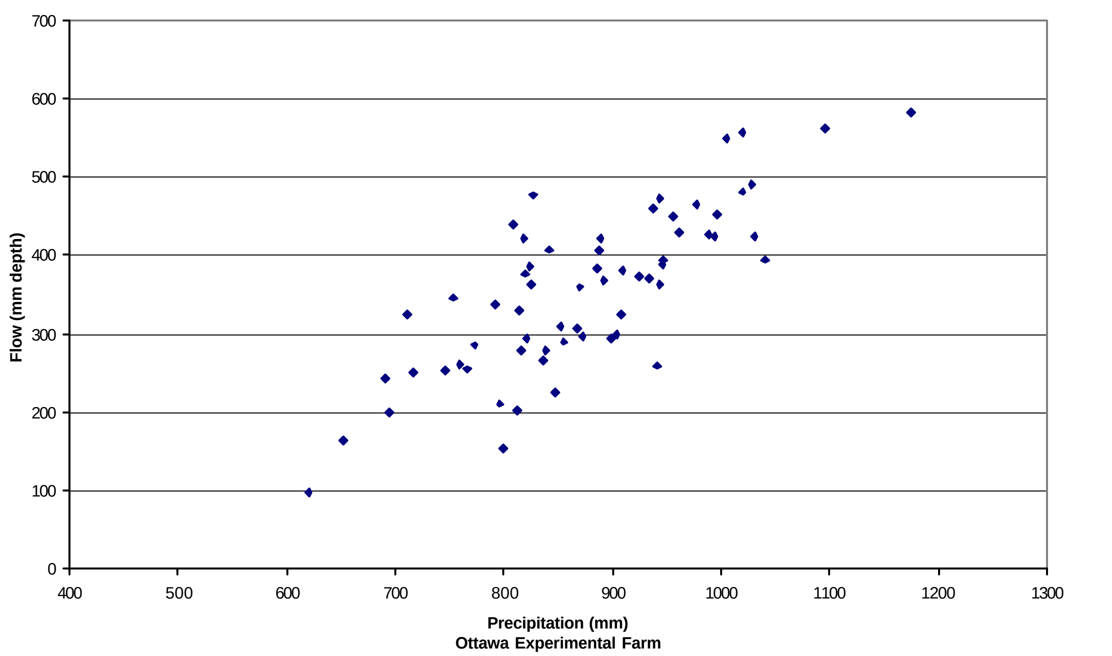

Daily Flow Duration Curve
02MC028 (1985-1998)
Delisle near Alexandria
Flow per Area (cms/sq km)
1
0.1
0.01
0.001
0.0001
0.00001
0.01
0.1
1
10
100
Percent Exceedance
1985-1998

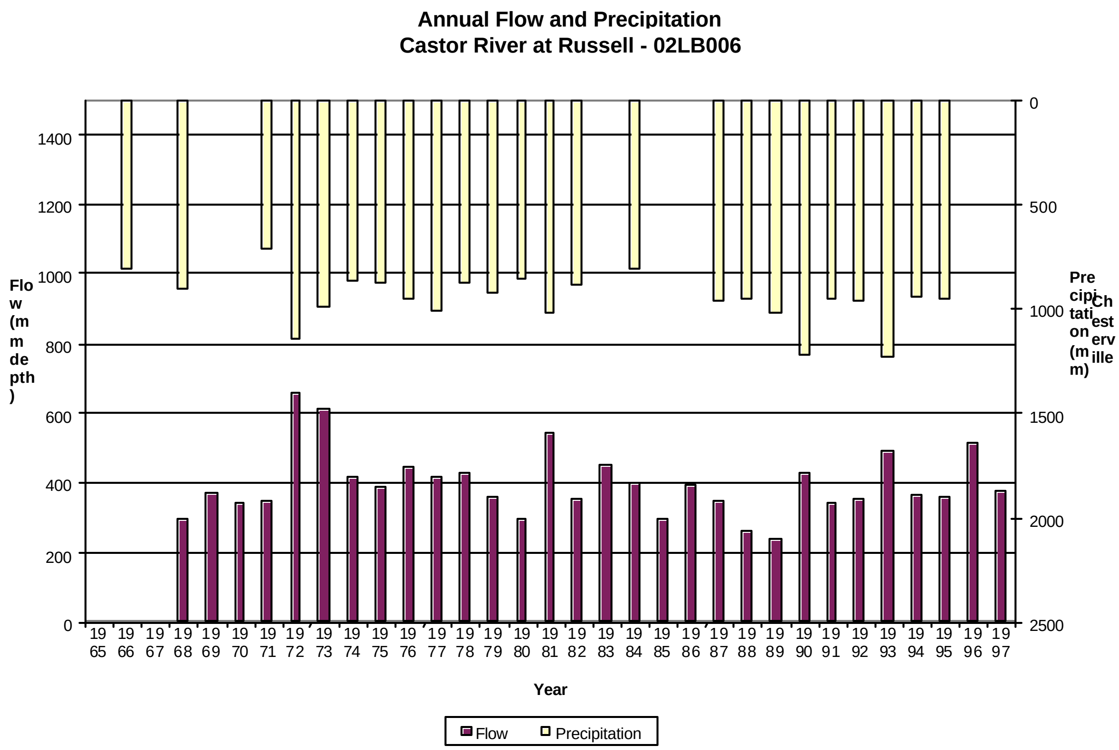
ANNUAL STREAMFLOW VERSUS ANNUAL PRECIPITATION

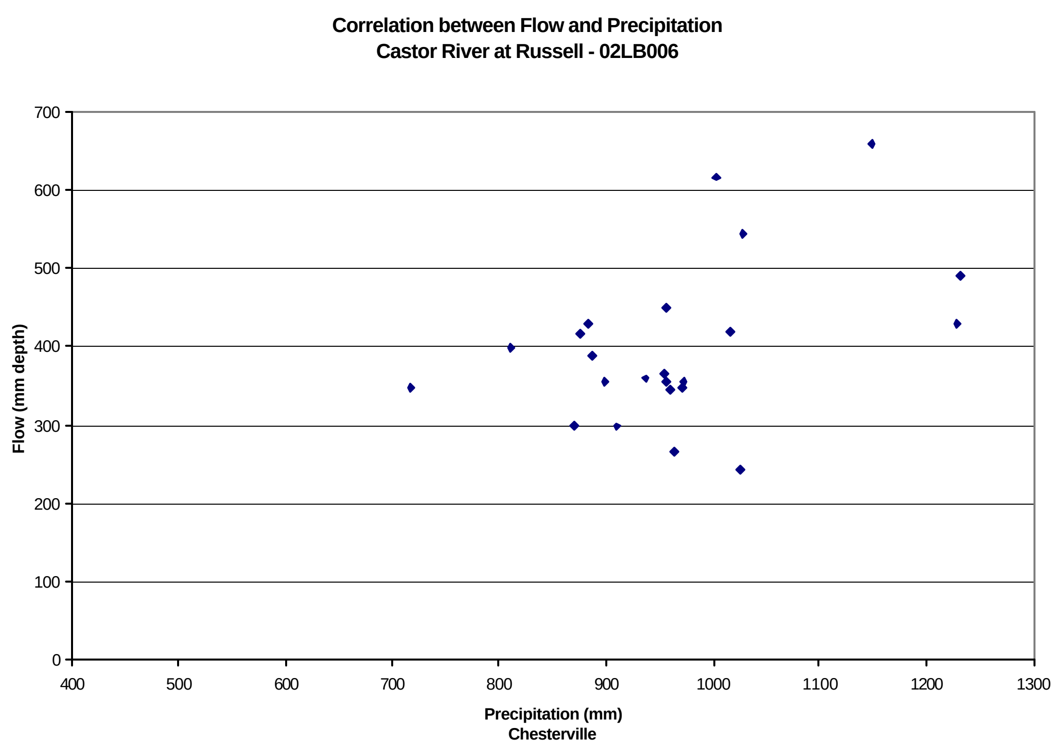
Annual Flow and Precipitation
South Nation River at Plantagenet Springs - 02LB005

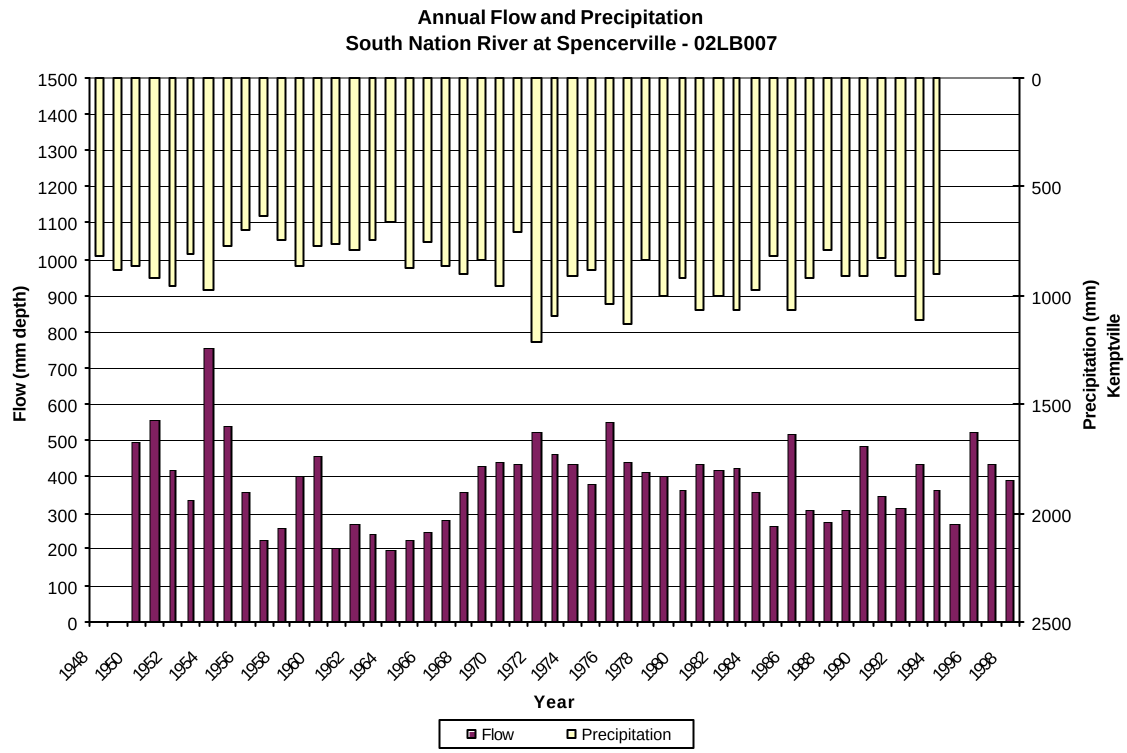


Correlation between Flow and Precipitation
South Nation River at Plantagenet Springs - 02LB005

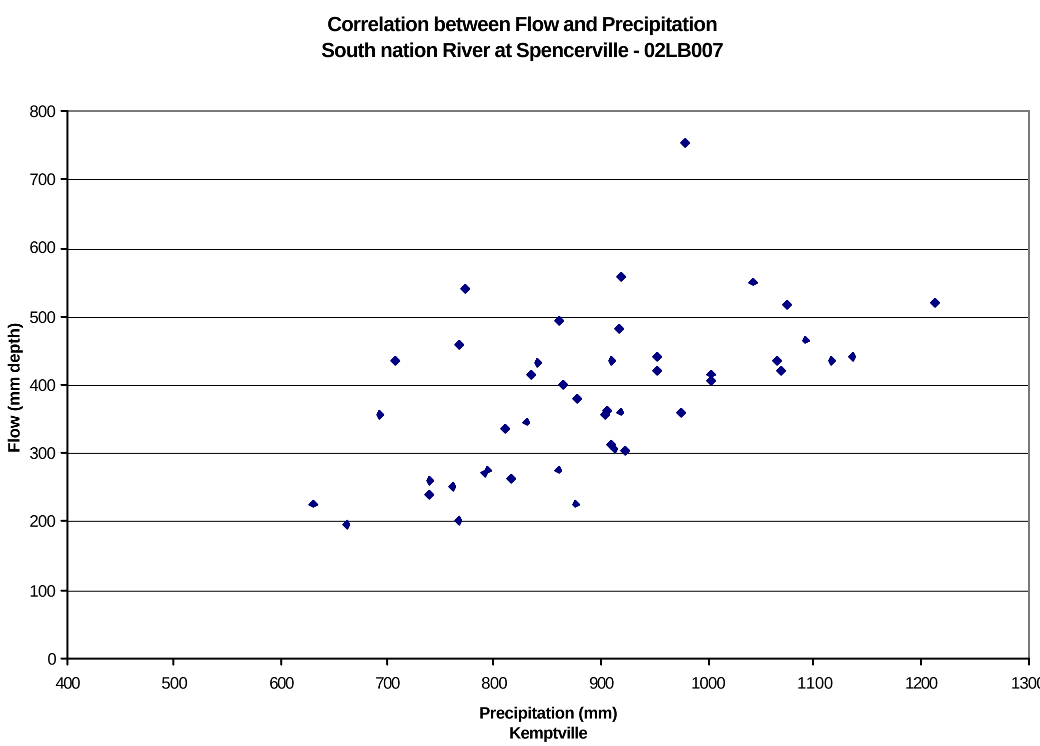



Annual Flow and Precipitation
Castor River at Russell - 02LB006
Flow (mm depth)
Precipitation Chesterville (mm)
Year
Flow
Precipitation

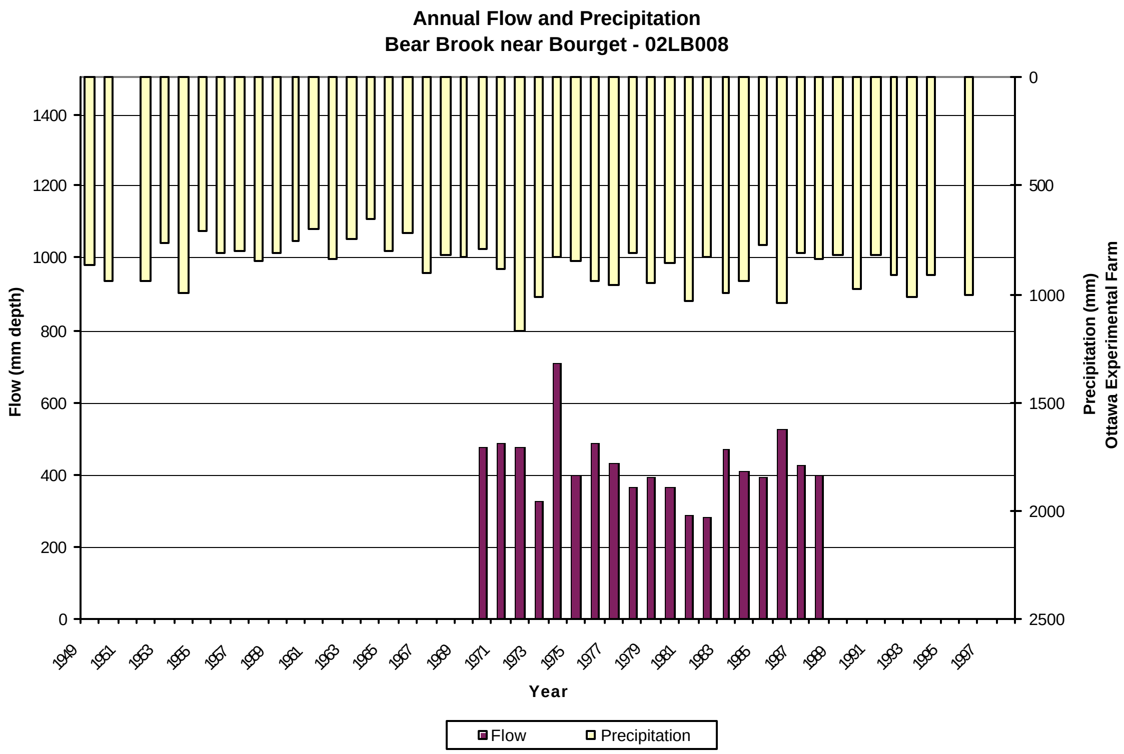

Correlation between Flow and Precipitation
Castor River at Russell - 02LB006
Flow (mm depth)
Precipitation (mm)
Chesterville

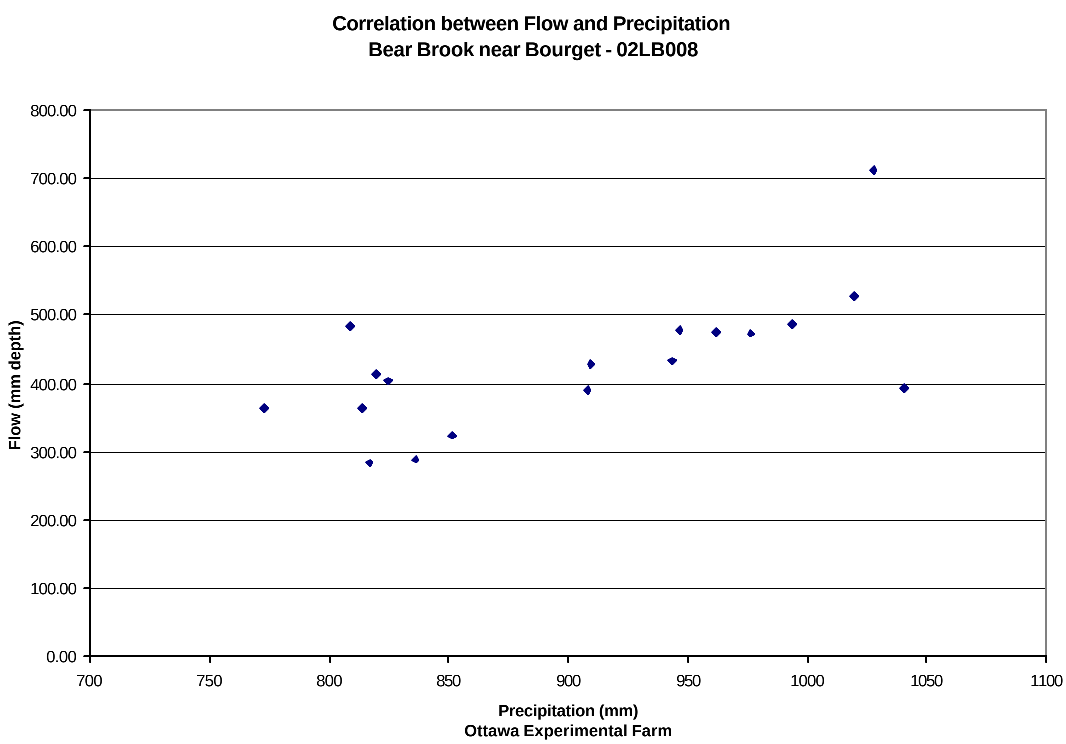


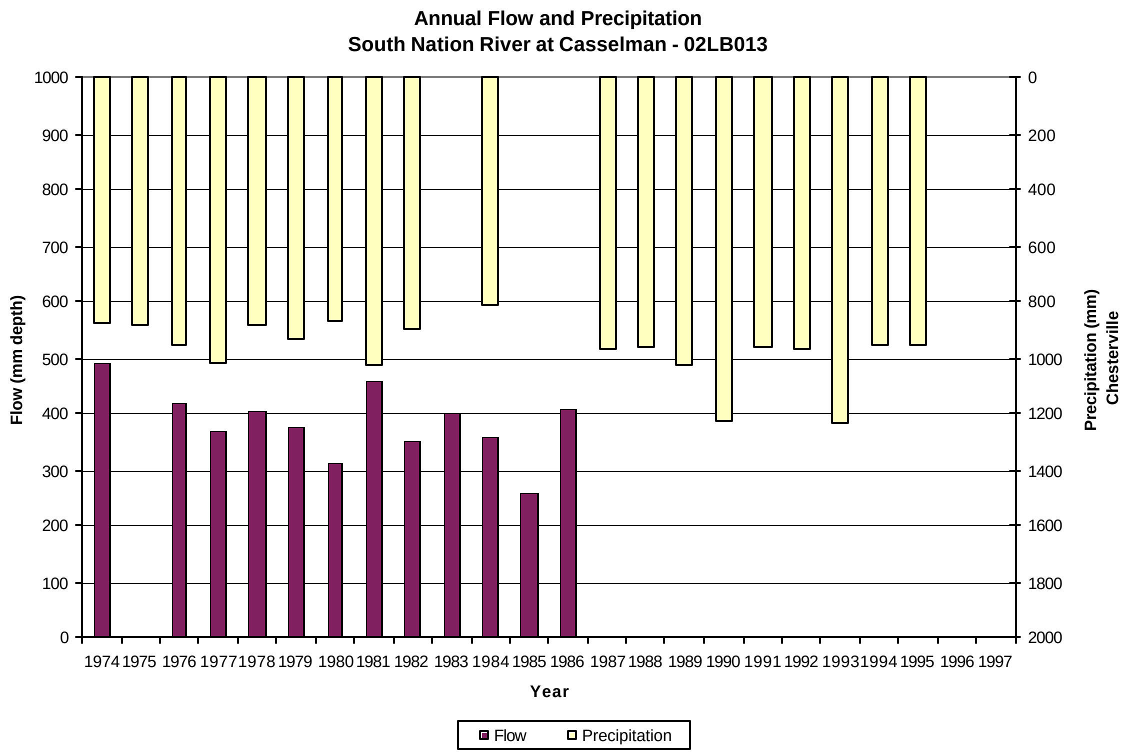
Annual Flow and Precipitation
South Nation River at Spencerville - 02LB007
Flow (mm depth)
Precipitation (mm)
Kemptville
1500
1400
1300
1200
1100
1000
900
800
700
600
500
400
300
200
100
0
500
1000
1500
2000
2500
1948
1950
1952
1954
1956
1958
1960
1962
1964
1966
1968
1970
1972
1974
1976
1978
1980
1982
1984
1986
1988
1990
1992
1994
1996
1998
Year
Flow
Precipitation



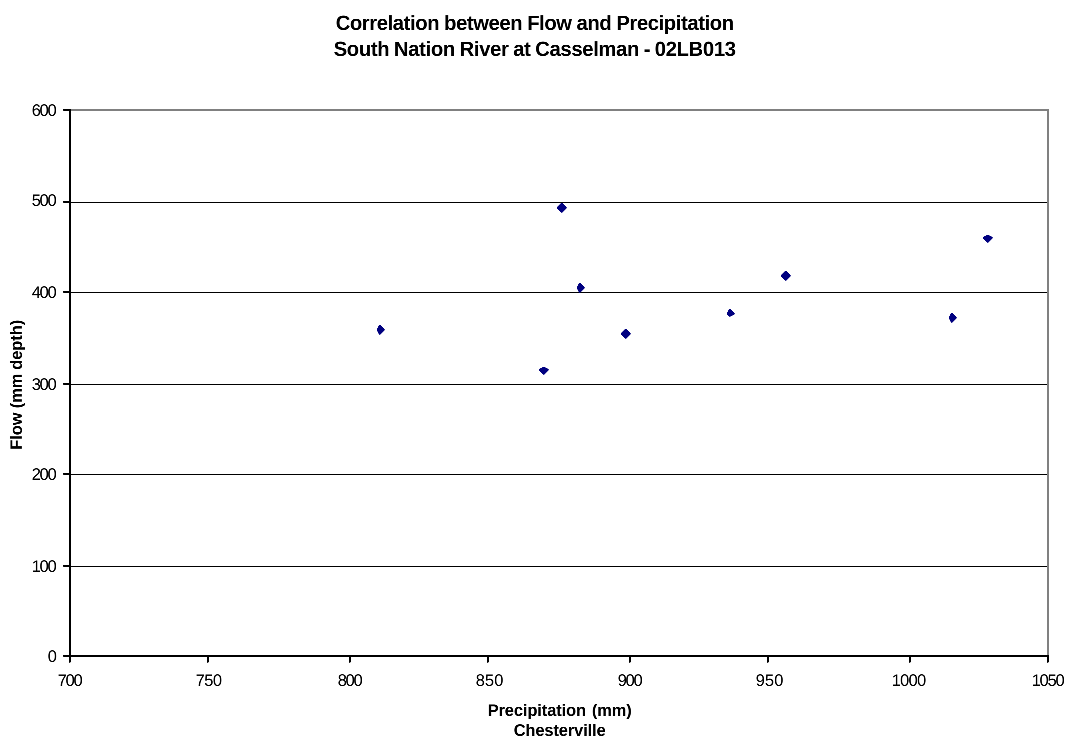
Correlation between Flow and Precipitation
South nation River at Spencerville - 02LB007
Flow (mm depth)
800
700
600
500
400
300
200
100
0
400
500
600
700
800
900
1000
1100
1200
1300
Precipitation (mm)
Kemptville


Annual Flow and Precipitation
Bear Brook near Bourget - 02LB008
Flow (mm depth)
Precipitation (mm)
Ottawa Experimental Farm
Year
Flow
Precipitation

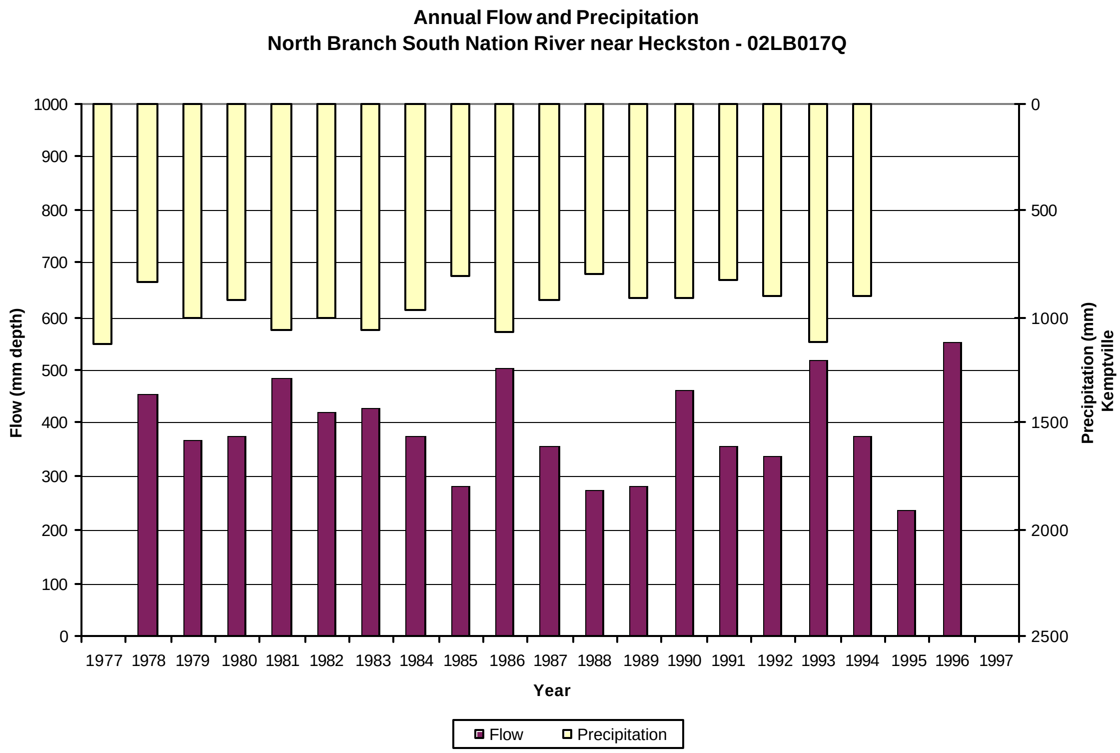

Correlation between Flow and Precipitation
Bear Brook near Bourget - 02LB008
Flow (mm depth)
Precipitation (mm)
Ottawa Experimental Farm

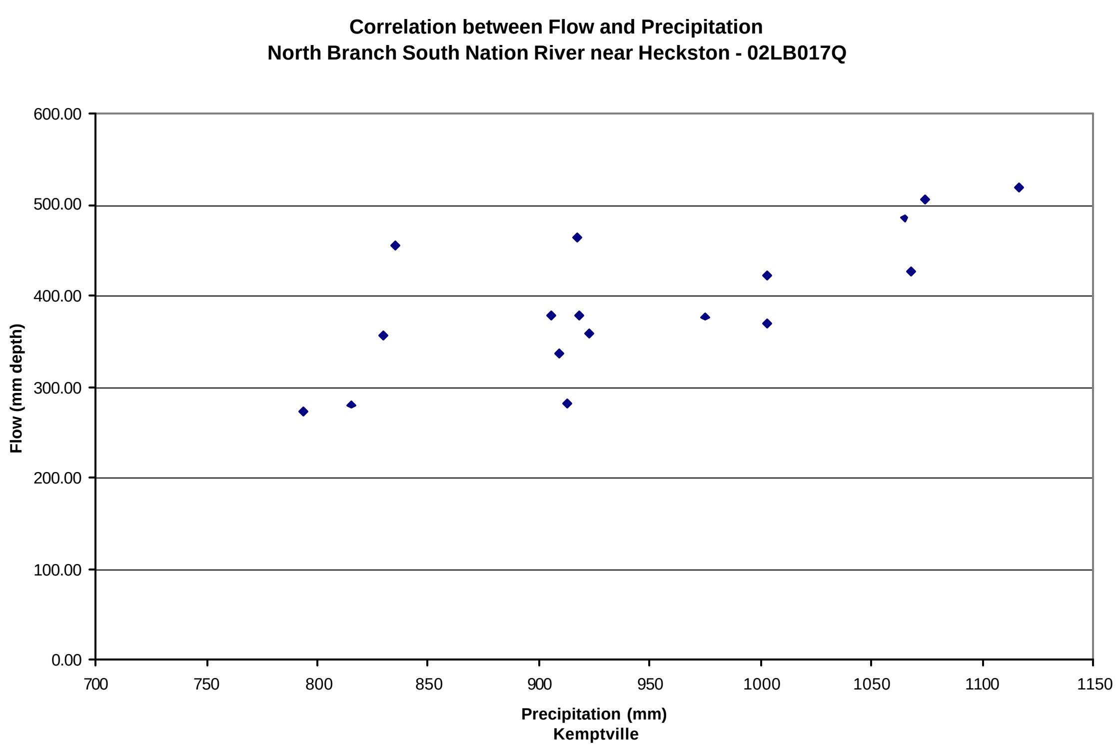


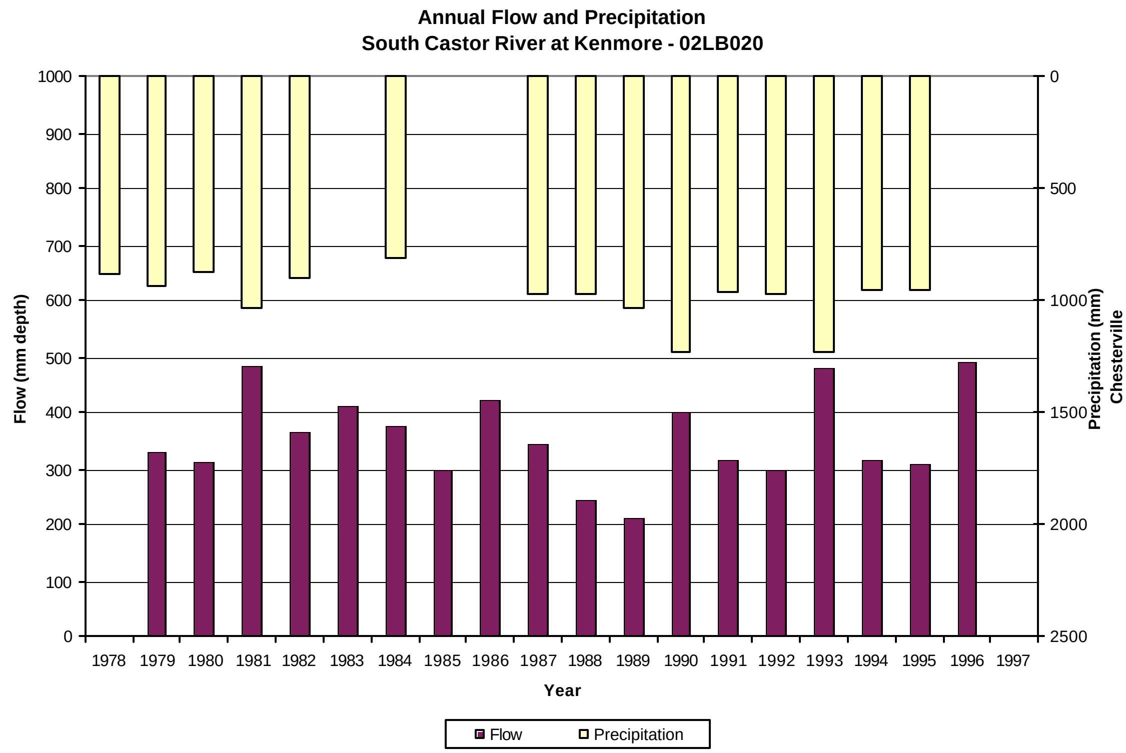
Annual Flow and Precipitation
South Nation River at Casselman - 02LB013
Flow (mm depth)
Precipitation (mm)
Chesterville
Year
Flow
Precipitation

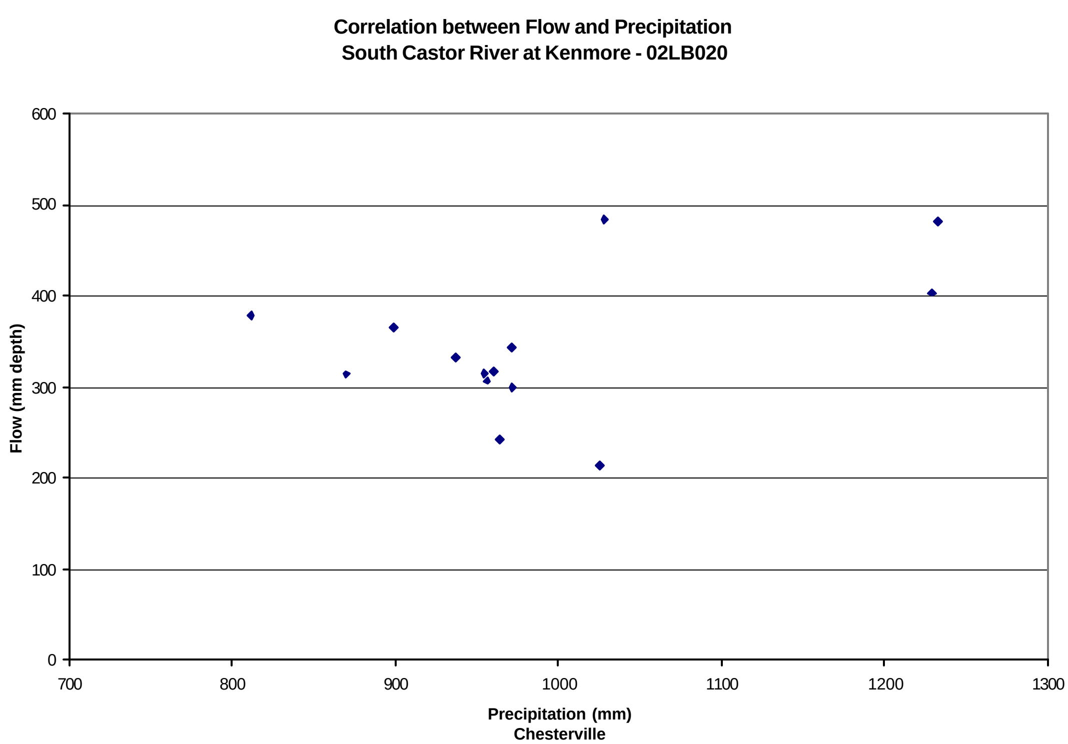


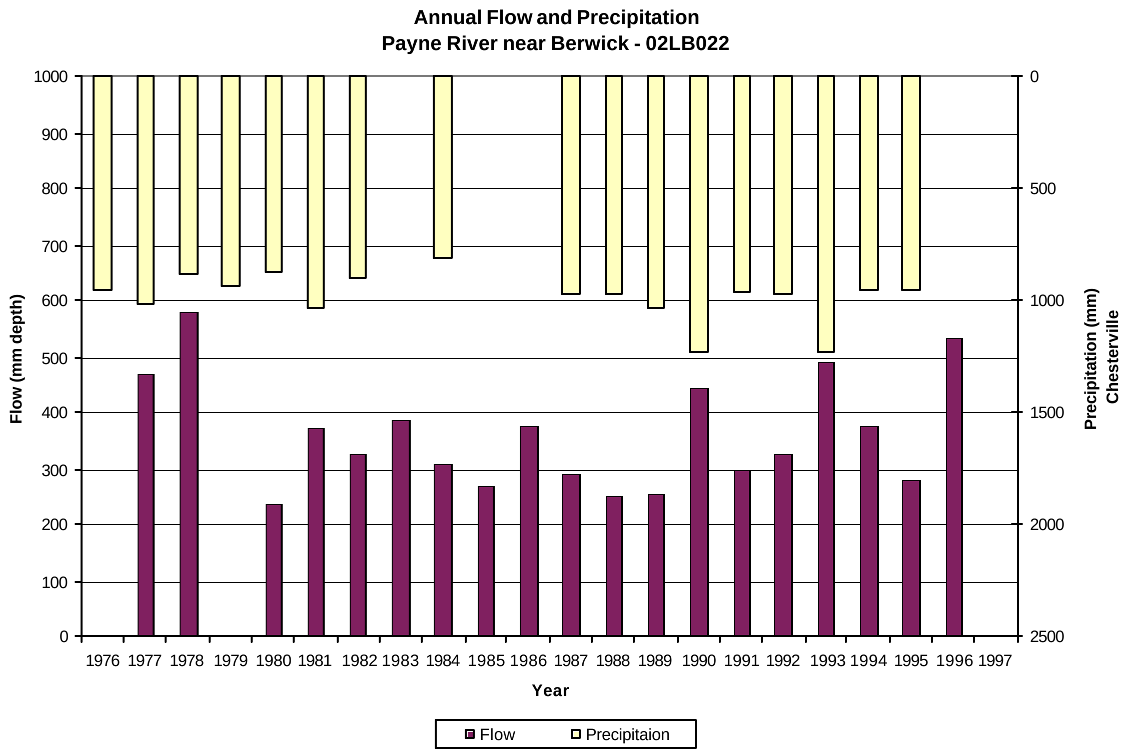
Correlation between Flow and Precipitation
South Nation River at Casselman - 02LB013
Flow (mm depth)
Precipitation (mm)
Chesterville


Annual Flow and Precipitation
North Branch South Nation River near Heckston - 02LB017Q
Flow (mm depth)
Precipitation (mm)
Kemptville
Year
0
100
200
300
400
500
600
700
800
900
1000
500
1000
1500
2000
2500
1977 1978 1979 1980 1981 1982 1983 1984 1985 1986 1987 1988 1989 1990 1991 1992 1993 1994 1995 1996 1997
Flow
Precipitation

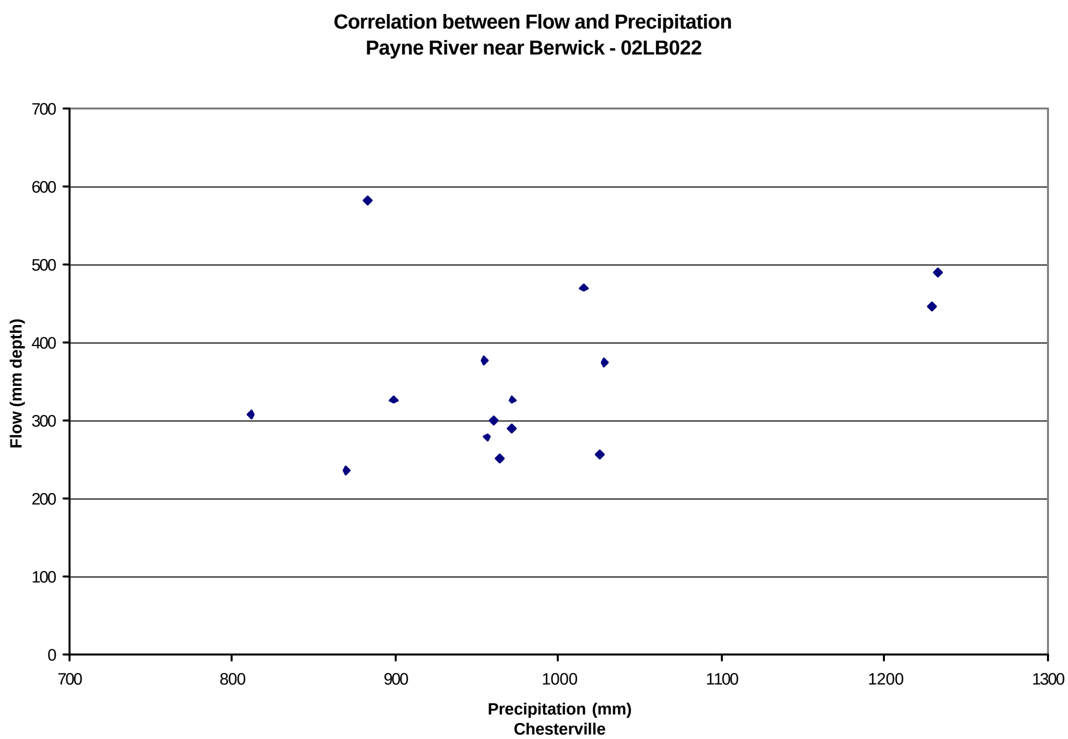

Correlation between Flow and Precipitation
North Branch South Nation River near Heckston - 02LB017Q
Flow (mm depth)
0.00
100.00
200.00
300.00
400.00
500.00
600.00
700
750
800
850
900
950
1000
1050
1100
1150
Precipitation (mm)
Kemptville


Annual Flow and Precipitation
South Castor River at Kenmore - 02LB020
Flow (mm depth)
Precipitation (mm)
Chesterville
Year
1978
1979
1980
1981
1982
1983
1984
1985
1986
1987
1988
1989
1990
1991
1992
1993
1994
1995
1996
1997
Flow
Precipitation


Correlation between Flow and Precipitation
South Castor River at Kenmore - 02LB020
Flow (mm depth)
Precipitation (mm)
Chesterville

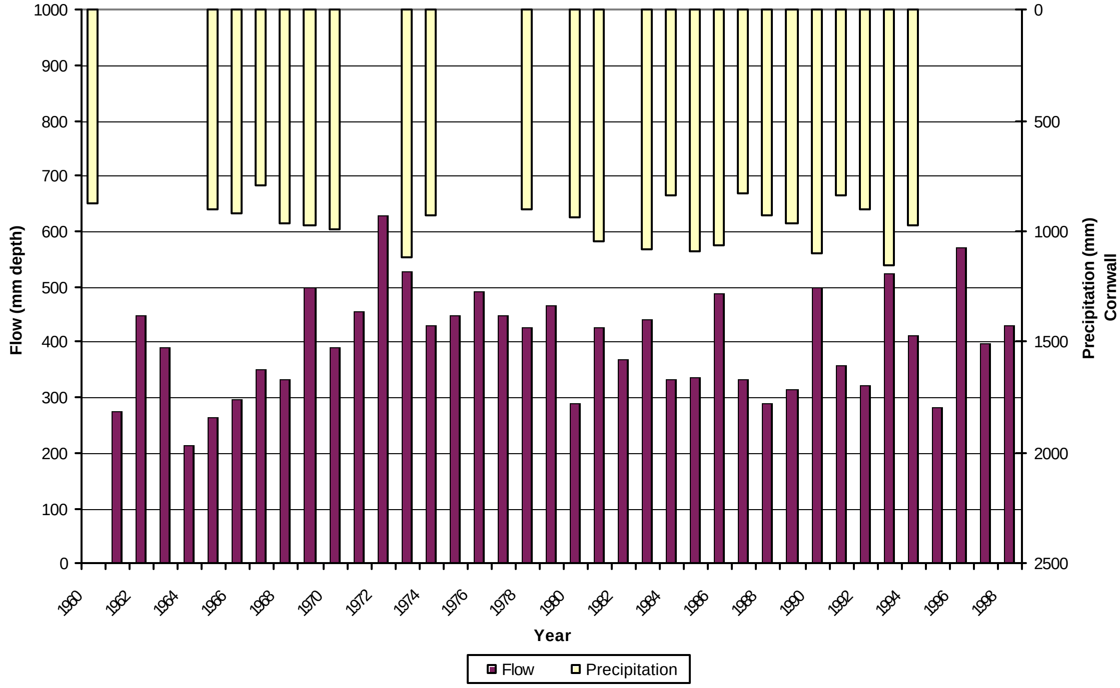


Annual Flow and Precipitation
Payne River near Berwick - 02LB022
Flow (mm depth)
Precipitation (mm)
Chesterville
1000
900
800
700
600
500
400
300
200
100
0
2500
2000
1500
1000
500
1976 1977 1978 1979 1980 1981 1982 1983 1984 1985 1986 1987 1988 1989 1990 1991 1992 1993 1994 1995 1996 1997
Year
Flow
Precipitaion

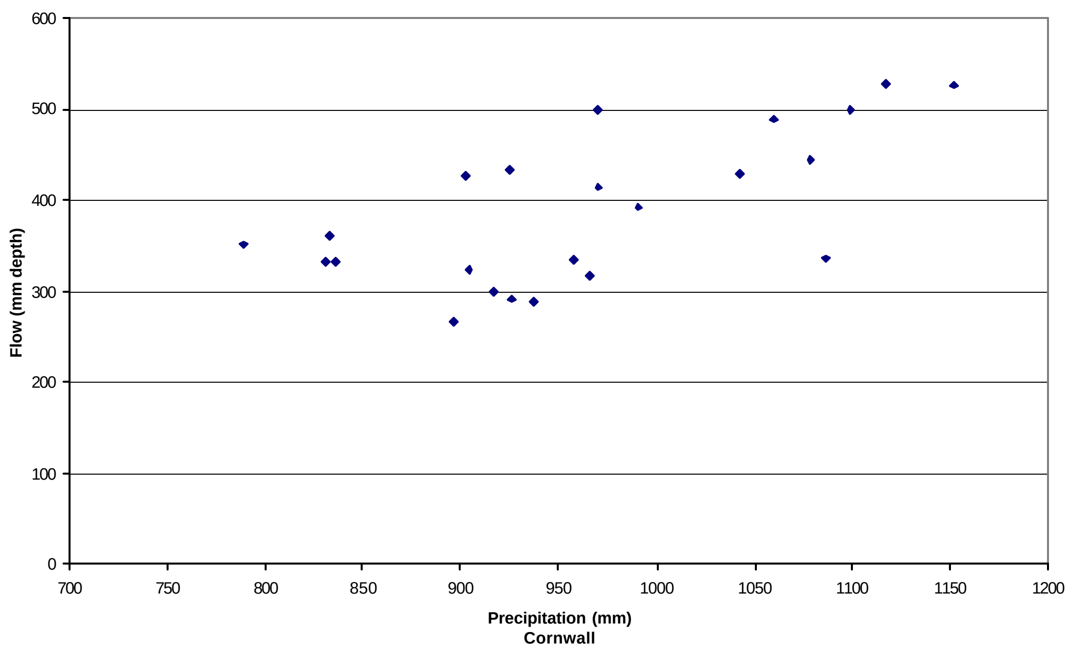


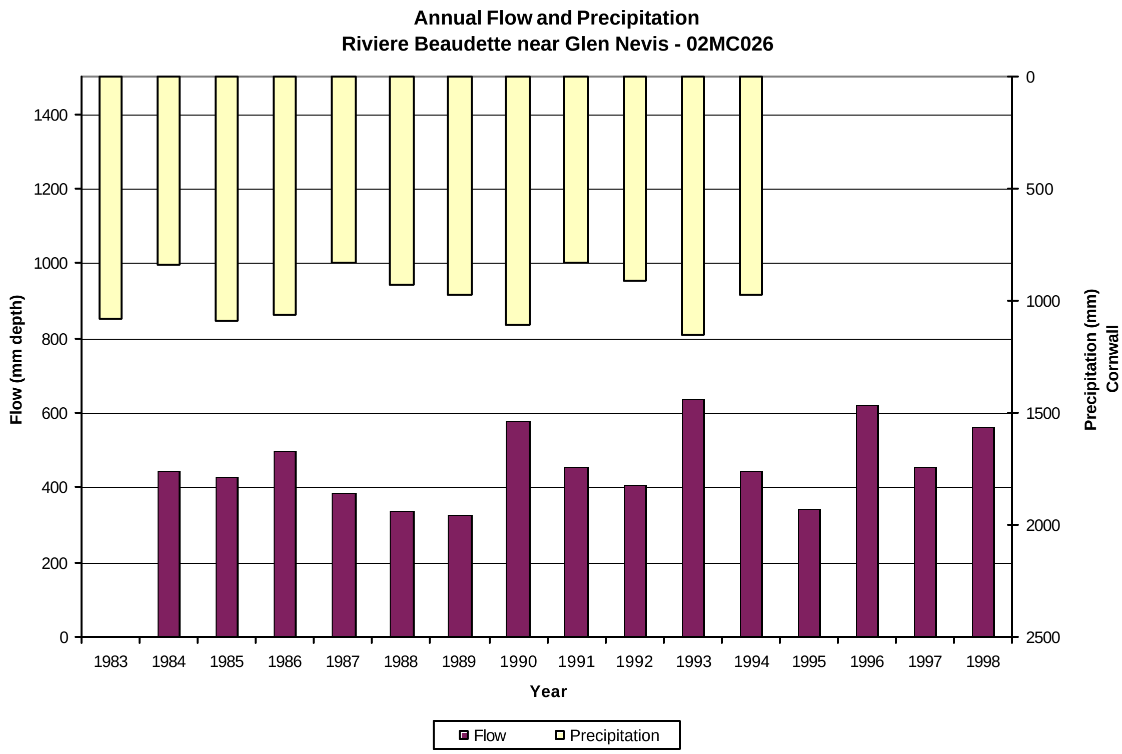
Correlation between Flow and Precipitation
Payne River near Berwick - 02LB022
Flow (mm depth)
700
600
500
400
300
200
100
0
700
800
900
1000
1100
1200
1300
Precipitation (mm)
Chesterville

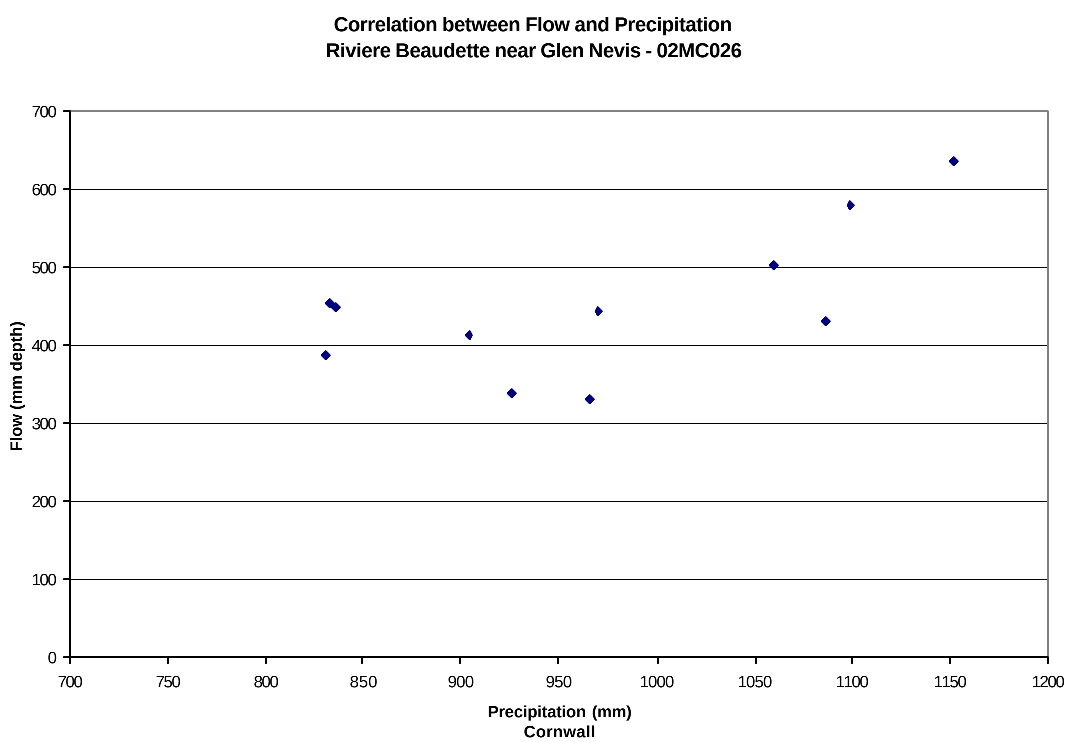
Annual Flow and Precipitation
Raisin River near Williamstown - 02MC001

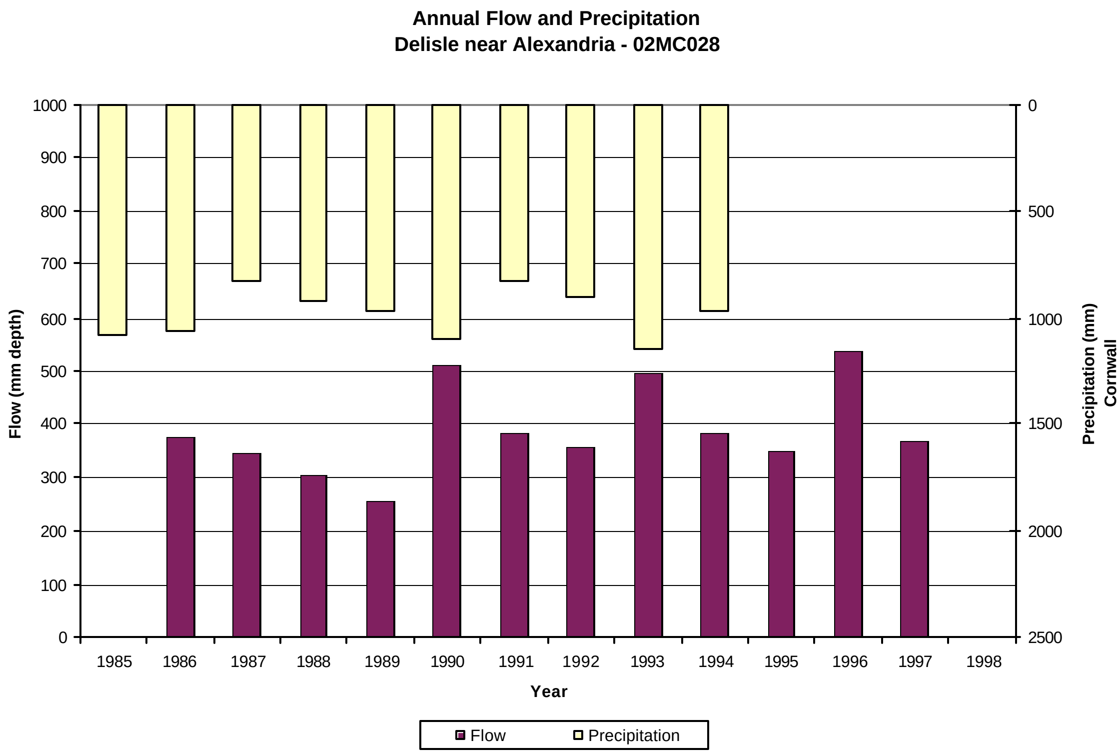


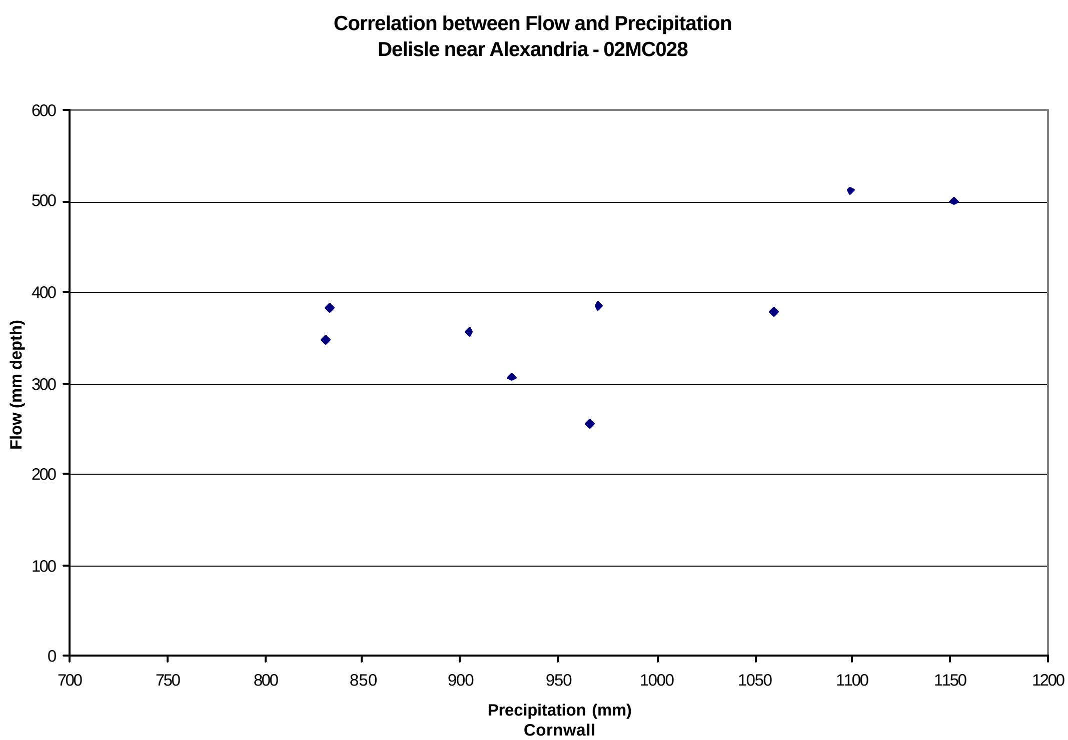
Correlation between Flow and Precipitation
Raison River near Williamstown - 02MC001




Annual Flow and Precipitation
Riviere Beaudette near Glen Nevis - 02MC026
Flow (mm depth)
0
200
400
600
800
1000
1200
1400
Precipitation (mm)
Cornwall
0
500
1000
1500
2000
2500
1983
1984
1985
1986
1987
1988
1989
1990
1991
1992
1993
1994
1995
1996
1997
1998
Year
Flow
Precipitation


Correlation between Flow and Precipitation
Riviere Beaudette near Glen Nevis - 02MC026
Flow (mm depth)
0
100
200
300
400
500
600
700
700
750
800
850
900
950
1000
1050
1100
1150
1200
Precipitation (mm)
Cornwall


Annual Flow and Precipitation
Delisle near Alexandria - 02MC028
Flow (mm depth)
1000
900
800
700
600
500
400
300
200
100
0
Precipitation (mm)
Cornwall
0
500
1000
1500
2000
2500
1985
1986
1987
1988
1989
1990
1991
1992
1993
1994
1995
1996
1997
1998
Year
Flow
Precipitation


Correlation between Flow and Precipitation
Delisle near Alexandria - 02MC028
Flow (mm depth)
600
500
400
300
200
100
0
700
750
800
850
900
950
1000
1050
1100
1150
1200
Precipitation (mm)
Cornwall

APPENDIX D

Detailed Calculations of Soil Loss by Erosion RUSLEFAC (Revised Universal Soil Loss Equation for Application in Canada) Livestock Intensity and Phosphorus Balance

The Revised Universal Soil Loss Equation Approach to Estimating Soil Loss

This is a theoretical model that attempts to represent a phenomenon that is highly episodic in an average fashion. This approach to estimating soil loss due to water erosion is documented in Wall et al (1997). The general conditions, unique to any site, which effect erosion by water are climate, soil, topography, vegetation or crop and land use practices. Each condition is represented by a different factor in the RUSL equation as follows:

$$A = R \times K \times L \times S \times C \times P \quad \text{.....(A1-1)}$$

Where:

A	=	represents the potential long-term average annual soil loss in tonnes per ha per year
R	=	the rainfall factor (MJ mm / ha / h)
K	=	the soil erodibility factor (t h / MJ / mm)
L & S	=	dimensionless slope length and steepness factors, respectively
C	=	the cropping management factor (dimensionless)
P	=	the supporting management practice factor (dimensionless)

Estimates are made for all factors on the right hand side of the equation (except 'P') for the entire EOWRMS study area. The 'P' factor includes practices such as strip-cropping, reduced tillage etc and there is no way to estimate these at the regional level. Values for this factor could be included in a more detailed assessment of soil loss.

"Erosion causes both on-farm and off-farm problems for Canadian agriculture. The off-farm impacts of sediment, bacteria from organic matter, nutrients and pesticides on the environmental quality and economic capability of surface water ecosystems are substantial and well-documented. The Revised Universal Soil Loss Equation For Application in Canada (RUSLEFAC) is intended to provide simple and reasonably accurate methods to estimate soil loss from water erosion. RUSLE only predicts the amount of soil loss that results from sheet or rill erosion on a single slope and does not account for additional soil losses that might occur from gully, wind or tillage erosion, nor does it calculate sediment yield to streams (Wall et al., 1997).

Procedures to Estimate RUSLE Factors for EOWRMS

The 'R' Factor was estimated directly from figure R-1 (Wall et al., 1997), which showed that the EOWRMS study area fell within the range 1250 to 1375. An average figure was used for the calculations.

The RUSLE-K factor is derived from soil surface textures. For the soils of Prescott and Russell (P&R) and Stormont, Dundas and Glengarry (SD&G), many of the soils were characterized within the Ontario soil name and layer files. The estimate of K was based on table K3 of Wall et al (1997) – the cutoff between <2 percent organic matter and >2 percent is 1.2 percent organic carbon. For soils not characterized in the attribute files, textures were estimated from either the mapunitnom – which designated a texture for the older maps or from the soil report and soil description for the Region of Ottawa-Carleton (ROC).

For the soils of Gloucester, surface texture is defined in the denominator of the map symbol by textural groups; the interpretation for the RUSLE-K factor is summarized in Table D.1.

TABLE D.1
RUSLE-K FACTORS ASSIGNED TO SOILS IN GLOUCESTER

Texture Group as Indicated in Map Symbol	Range of Textures Included		RUSLE-K Factor Assigned
O	Organic		None
1	Sand	.001	.010
	LS	.005	
	LfS	.015	
2	SL	.017	.023
	fSL	.024	
3	vfSL	.046	.045
	SL	.017	
	L	.040	
	SiL	.050	
	Si		
4	CL	.040	.040
	Si CL	.042	
	SCL	.026	
5	SC		.032
	C	.029	
	SiC	.034	
6	Hv C		Not found

Estimating LS factor: Initially, the OSLAP (Ontario Soil Landscape Attribute Project) field sheets were used because they reported both slope percent and both natural and effective length. The field sheet points cover was intersected with the soil map and the LS was generalized to describe the soil unit which had been sampled – using mapunitnom; coverage was quite incomplete. However, the weighted average of slope length values for the study area was 2.67 or approximately 130 m.

A second approach was based on assigning a slope percent estimated based on slope classes from the map symbol according to the following rules summarized in Table D.2.

TABLE D.2
RULES USED TO ASSIGN SLOPE PERCENT BASED ON SLOPE CLASS

Slope	Estimated Slope Percent	Qualifying Comment
B b	2	This encompasses the range 0-2 and the values were taken from the midpoint, i.e. slope of ‘1’
B=C	4	
B>C	3	
B=D	6	
b>d	5	
b>f	2	Presumably agriculture is not on the F slope
C c	5	
C=b	4	
C=d	8	(Values in LS table go by 2’s in this range)
C with slopes e, f	5	Presumably agriculture is not on the steeper slopes
c>b	4	
c>d	8	
d D	8	
d = or > c	7	
D with higher slopes	8	Presumably agriculture is not on the steeper slopes
E and F	10	

From the table LS4 – for slopes within the range of agricultural activities, the most sensitive range is between lengths of 50 and 150 m. Recognizing that most effective slope lengths will be limited to about 200 m, the following rules were used to estimate slope length.

Estimated slope length was assigned as follows:

Complex slope (lowercase letters)	75 m
Simple slope (uppercase letters	150 m
Complex and simple (combined class)	100 m

LS values are taken from table LS-4

For Grenville, where slope percent is not listed, the following assessments were made:

Topography	Estimated steepness and length	RUSLE ‘LS’ factor
Undulating – rolling	5/100	0.79
Level –slightly undulating	2/100	0.36
Depressional	0.2/100	0.04
Gently undulating – depressional	2/100	0.36
Level – slightly undulating	1/100	0.17

Potential soil loss (bare soil) is calculated as (1250 + 1375) / 2* RUSLE-K*estLS

Estimating the Crop Factor 'C': This factor was estimated based on Table C-3 Region 4 (Wall et al, 1997) for land in agriculture based on land use reported in the 1996 Census of Agriculture. The calculation for each EA grouping is as follows:

Average EA grouping 'C' factor = (Grain corn area*0.41 + Silage corn area*0.5 + soybean area * 0.46 + Spring grain area * 0.41 + Winter wheat area * 0.22 + alfalfa area * 0.02 + (Tame hay area + pasture area)*0.01) / (area of crop, hay and pasture).

Livestock Intensity

Livestock unit calculations are taken from Table 1: Minimum Distance Separation II (MDS II) Bulletin, OMAFRA, March 1995.

The calculations were based on livestock categories as defined in Census of Agriculture (CoA) and are thus somewhat approximate. In this section, the actual livestock categories as recorded in the CoA are listed followed by the calculation used to estimate livestock units.

Poultry Livestock Units

1. Laying hens - 19 weeks and over – Number
2. Broilers, roasters, cornish, and turkey production – Kilograms *** this is the only other poultry category with the exception of
3. Total poultry – Number

The poultry livestock units are therefore based on:

(Number of laying hens) / 125 + (Total poultry – Number of laying hens) / 200

Beef Livestock Units

1. Beef cows – Number
2. Steers - 1 year and over – Number

Beef livestock units are based on (Number of Beef cows)*1 + (Number of steers)*0.5

Dairy Livestock Units

1. Milk cows – Number
2. Heifers - 1 year and over – Number
3. Calves - under 1 year – Number

Dairy Livestock Units are based on the approximation as listed in MDS II that "a dairy farm usually has milking cows, dry cows, heifers, and calves. Multiply the number of milking cows by 1.5 to account for the followers when they are all kept on the same farm." Because there was no way in CoA to distinguish milking and dry cows, the dairy livestock units will be slightly over-estimated by using the calculation.

(Number of milk cows)*1.5

Swine Livestock Units

Total pigs – Number

Swine livestock units – 5 sows / boars; 20 weaners; 4 feeders. The approximate swine livestock units were calculated assuming 1 sow / boar produces 20 weaners and results in 16 feeders.

Swine livestock units = # of swine *5.2 / 37)

Phosphorus Balance – Quantity Excreted in Livestock Manure Compared to Quantity Removed Annually in Harvested Crops

Calculation of Manure P generated

Barnett (1997) presents a table that shows, by province, the population by livestock sector, and the P excreted in T / day. Manure P was estimated for each of cattle, poultry, and hogs using the equation:

(total number of livestock category) * P excreted (T / d)*365 / Ontario population of livestock category)

'P' excretions were summed for cattle, poultry, and hogs and converted to kilograms for each EA grouping.

Calculation of Crop Removal of P

Based on section 'L' of Nutrient Management Planning workbook:

Actual yields are the average for the three county areas (P&R, SD&G, ROC) for 1996. The removal for spring cereals is estimated as the average for barley and oats (24 lb / ac). The removal for hay is estimated based on the figure for alfalfa (66 lb / ac).

The calculation used is:

Area of crop (ha)*2.47 (ac / ha)*Removal P2O5 (lb / ac) * Actual Yield (bu / ac) / (Base Yield (bu / ac)*2.2 (lb / kg))

Crop Removal Balance P

In the Nutrient Management Planning workbook Balance P is calculated as available P2O5 * 2 "to account for an 80 percent phosphorus availability for soil buildup (only 40 percent is available to this year's crop)". For the EOWRMS – the phosphorus figures used by Barnett are actual phosphorus content of the manure rather than crop available so the figure was used directly. Strictly speaking it could have been reduced by 20 percent to match the Nutrient Management Planning estimate of 80 percent available.

APPENDIX E

Infrastructure Technology

Wastewater Treatment Alternatives

Types of Systems

Sewage disposal and treatment systems range from small single unit applications to communal systems and finally to large municipal treatment plants. There is a range of technologies and applications for each. In the EOWRMS the applications that have the potential to be used as cost effective servicing alternatives are:

Single Unit Applications for Onsite Sewage Systems

A single unit of either residential or commercial development can be services by a variety of methods. The MOE categories include:

- Class 1 systems that are basically chemical or other such toilet systems or privies
- Class 2 and 3 systems that consist of various ways to employ leaching pits in the management of the waste
- Class 4 systems that are septic systems
- Class 5 systems that are basically tanks relying on haulage of wastes to another treatment facility
- Class 6 systems which are Aeration facilities coupled with solids removal and leaching beds for subsurface disposal
- Class 7 which are hauled waste facilities with various land applications

As part of our evaluation of alternatives that may be best suited to the EOWRMS we have selected septic systems for further discussion.

Septic Tank Systems

Septic tank systems are an MOE Class 4 sewage system and consist of a septic tank connected to a building sewer, a leaching bed and the piping that transports the tank effluent to the leaching bed. A Certificate of Approval is required for the construction or alteration of a Class 4 sewage system.

The basic function of a septic system is to receive the waste from the building and partially treat it before discharging the liquid portion to the leaching bed. Solids are retained in the tank for later removal.

Septic systems can be designed for single family dwellings in residential areas and for non-residential applications such as shopping plazas, clubs, restaurants, and bars. The effectiveness of septic systems can be increased by using water conservation devices in conjunction with the septic system.

Septic systems can be designed for multi-unit applications; however, they are more typically applied to single units.

There is wide application of septic tank technologies in Ontario. Their application in Ontario is governed by appropriate design standards and implementation policy/guideline. Septic systems are considered a cost effective and technically effective treatment alternative.

Small Communal Systems

These types of wastewater treatment systems include many process types and configurations.

The term "Communal Wastewater Treatment Systems" has several connotations. The MOEE (MOEE, 1992, 3) and the MMA (1992, 3) defined communal systems as:

> ...those sewage works, sewage systems and water works to be approved, or approved under Sections 2 & 53, Ontario Water Resources Act RSO 1990 [surface discharges], or those under Part VIII, Environmental Protection Act RSO 1990 [subsurface discharges] for the common use of more than five units of full-time or seasonal residential or industrial/commercial occupancy or other occupancy as determined by MOEE staff.

Taken literally, CWTS can be defined as any sewage works that services more than one unit or residence. Definitions do not generally differentiate between CWTS and centralized municipal wastewater treatment systems from a technical standpoint because, with the exception of scale/size, there is no technical difference. Rather, Ontario's legislation refers to sewage treatment works and sewage systems on the basis of their point of discharge (i.e. surface or subsurface).

A background review was completed to establish the most appropriate technologies for application at a small-scale, taking into consideration Ontario's regulatory environment and the system's expected performance, capital and ongoing operating and maintenance costs.

These selected systems include:

Secondary Processes

- Fixed Film Processes
  - Rotating Biological Contactors (RBC)
  - Trickling Filters (TF)
- Suspended Growth Process
  - Sequencing Batch Reactor (SBR)
  - Extended Aeration (EA)
  - Facultative Lagoons
  - Aerated Lagoons
- Hybrid Systems
  - Biological Aerated Filters (BAF)

Because of the effluent constraints in the EOWRMS area it is assumed that tertiary treatment methods for phosphorus removal, ammonia removal and disinfection would be required in a number of areas. The tertiary treatment possibilities examined include:

Tertiary Processes

- Phosphorus Removal
  - Polishing Ponds
  - Constructed Wetlands
  - Physical / Chemical Treatment
- Nitrogen Removal
  - Biological Nitrification / Denitrification
  - Natural Processes
- Disinfection
  - Chlorination / Dechlorination
  - Ultraviolet Disinfection
  - Ozonation

System Descriptions

Rotating Biological Contactors (RBC)

The Rotating Biological Contactor (RBC) process is a fixed film biological reactor consisting of closely spaced disks mounted on a horizontal shaft, supported in a semicircular or trapezoidal concrete or steel tank. Common media forms for the disks include styrofoam and / or dense lattice material of polyethylene. Wastewater flows through the tank as the media slowly rotates with about 40 percent immersed for contact with the wastewater for removal of organic matter by the biological film that develops on the media. As the shaft rotates, the disk surfaces are alternately exposed to wastewater and the atmosphere. Microorganisms naturally present in the wastewater adhere to and grow on the disk surface. Due to the rotating action, the disks carry a film of wastewater into the air, and oxygen is transferred from the air to the liquid film, and ultimately to the slime layer. As the disk passes through the bulk of the wastewater, mixing at the disk surface is promoted and absorption of organics occurs. As the microbial growth proceeds, a biological film is formed on the disk surface. Excess biomass on the media is stripped off by rotational shear forces, and the stripped solids are maintained in suspension by the mixing action of the rotating media.

Primary treatment is an integral part of the overall process, and may consist of a conventional primary clarifier and fine screening followed by grit removal. In small installations, primary treatment and sludge handling can be accommodated by a septic tank. Primary clarification or a septic tank is preferred over screening devices if large amounts of oil and grease are expected. In addition, an integral primary settling tank beneath the disc unit is recommended.

The RBC process is a simple and reliable biological process which has been used for the treatment of municipal wastewater in the United States since 1969. The modular construction, low hydraulic head loss, and shallow excavation make it adaptable to new and existing treatment facilities, and as such, the use of this technology is growing.

Reported treatment efficiencies of RBCs (USEPA, 1980a) at wastewater temperatures as low as 5°C are reported as:

- BOD_5 - 94%
- TSS - 95%
- 94% nitrification
- 81% total nitrogen removal

Trickling Filters

Trickling filters are an attached growth biological treatment process. The process consists of a filter bed of inert media including rock, plastic or other synthetic material over which wastewater is applied for aerobic biological treatment. Wastewater that has undergone primary clarification is dosed by a distributor system over the top of the bed and trickles down through the media. Organic material and oxygen are adsorbed and utilized by the microorganisms attached to the filter media. The quantity of biological slime produced is controlled by the available food, hydraulic dosage rate, type of media, type of organic matter, amount of essential nutrients present, temperature, and the nature of the biological growth. The biological slime is sloughed off the media either periodically or continuously during filter operation. Trickling filters are usually classified as low or high rate, according to the applied organic or hydraulic loadings.

The trickling filter process has widespread use and has been effectively used for large scale sewage treatment facilities. The process and mechanical reliability is highly dependable in moderate climates; however, colder climates may cause operational and performance problems. The use of after-treatment or multi-staging has frequently been necessary to ensure uniform compliance with effluent criteria in colder climates. Performance problems with trickling filters in New Brunswick were reported to be the result of poor management programs rather than the actual treatment process.

The treatment process is appropriate for domestic wastewaters amenable to aerobic biological treatment in conjunction with suitable pretreatment and tertiary treatment, as required. The process is good for the removal of suspended or colloidal materials, and is somewhat less effective for the removal of soluble organics. The process is effective for nitrification of the wastewater through aerobic activity. Denitrification of the wastewater can be provided by a supplementary anoxic process.

Performance criteria of the trickling filter process using a single stage configuration with primary and secondary clarification was reported by the USEPA (1980a) as follows:

- BOD_5 - 75% to 90%
- Phosphorus - 10% to 30%
- NH_4-N - 20% to 40%
- SS - 75% to 90%

Sequencing Batch Reactors

The Sequencing Batch Reactor (SBR) is a form of activated sludge treatment process typically consisting of parallel reactor tanks. The SBR process features the fill and draw principle in which all phases of the treatment cycle occur sequentially in one basin.

Conventional activated sludge systems require separate tanks for the unit processes of aeration and sedimentation/clarification and also require the recycling of portions of the influent flow to maintain a proper mixed liquor suspended solid (MLSS) concentration and control of the sludge volume index (SVI).

Limited use of the SBR process began in the 1960s; however, the fill and draw concept is not new. During the development of the activated sludge process, the fill and draw concept was used in original experiments in the development of suspended growth by aerating wastewater. The key to the process is effective control that can be implemented through the use of programmable logical controllers (PLC) to allow for effective sequencing of the various phases of the process.

There are numerous SBR facilities in the United States and Manitoba and more recently a number of installations have been occurring in Ontario. The design of SBR facilities, however, varies greatly from site to site. SBR facilities appear to be somewhat lower in cost than activated sludge facilities and exhibit much greater flexibility in terms of performance capabilities. SBR facilities are capable of high levels of carbon oxidation, nitrification, denitrification and biological phosphorus removal. In addition, SBR technology is advantageous when space is limited.

The SBR process depends upon the reliability of automatic controllers for valves, pumps, aeration systems and decanter systems. Chemicals can be used if biological conditions for phosphorus removal are not suitable. SBRs are capable of biological nitrogen and phosphorus removal, which is accomplished by proper reactor sizing and selection of stage lengths and aeration times to achieve the desired distribution of aerobic, anoxic and anaerobic conditions.

Modifications to the SBR process can be implemented to changing influent characteristics, with minimal effluent degradation. In addition, the operator can adjust the computerized controls to vary the operation. Nitrification occurs within the aeration phase of the cycle and involves the biochemical conversion of ammonia to nitrate, whereby the ammonia is oxidized to nitrite and the nitrite is oxidized to nitrate. Substantial denitrification occurs in both the BOD and nitrification processes. Where a higher degree of denitrification is required, anoxic sequencing is incorporated into the treatment cycle.

Biological phosphorus removal can provide an alternative to chemical treatment; however, control of such a system for inexperienced operators is difficult. Alternative methods of phosphorus removal using physical-chemical processes (filtration) may be more appropriate.

There are a limited number of SBR installations in Ontario. As such, information on operational treatment efficiencies for small-scale applications is also limited. Based upon larger-scale applications, SBR treatment performance has been reported as follows (USEPA, 1980):

- BOD_5 - 85% to 98%
- TSS - 85% to 98%
- NH_3-N Oxidation - 90% to 95%
- Total Nitrogen Removal - 85% to 90%
- Total Phosphorus - less than 1 mg/L effluent (biological removal)

Extended Aeration

The extended aeration process is a "low rate" modification of the activated sludge. This process is widely used in Ontario with plant capacities up to 2,200 m^3/d. The process is characterized by low loading rates and long hydraulic and solids retention times. Due to the low BOD loading, the process operates in a phase of the microbial growth cycle, whereby the organisms are starved and forced to undergo partial auto-oxidation.

Primary treatment facilities are minimized through the use of raw sewage shredding as primary treatment components. Primary clarification is rarely used. The amount of sludge to be wasted in an extended aeration process is less than a conventional activated sludge process. The sludge normally contains little putrescible organic material, and can be discharged for direct sludge beds without offensive odours. The extended aeration process has been used extensively to treat a wide range of wastewater flows (9 to 3,525 m^3/d). Prefabricated package plants are frequently used for smaller applications.

The process generally consists of an aeration vessel where oxygen is added to the raw sewage/microorganisms mixture and a clarifier which separates the biomass from the treated effluent. The clarifier must be adequately sized and preferably equipped with mechanical sludge and scum removal systems involving scraper mechanisms and pumping equipment. Sludge is typically wasted to an aerobic holding tank.

Additional processes for denitrification and/or phosphorus removal can be added to the extended aeration process to enhance the quality of the effluent for surface or subsurface disposal. BOD and suspended solids removal from extended aeration plants are approximately the same as for conventional activated sludge plants and, as such, a high level of secondary treatment can be achieved. Because of the extremely low loading used in the extended aeration process, disintegration of the sludge flocs may occur. As a result, effluent suspended solids are usually higher than for a conventional activated sludge system.

Lagoons

Lagoons are in wide use in Ontario, with over three hundred installations ranging from small, single cell systems to large, multiple cell (e.g. 6 cells, 48 ha) systems. Lagoons are typically excavated in the natural soil or built above grade by enclosing an area with impermeable earthen dykes. Large areas are generally required for lagoons, and the shape and layout are frequently chosen to accommodate the available space. The most important feature of a wastewater treatment lagoon is the biological life that is encouraged to grow. The type of lagoon, whether it be facultative or aerated, has a particular type of predominant biological life. When locating a lagoon system, such factors as distance from the nearest development, prevailing winds, surface runoff, and groundwater pollution must be considered. In general, setbacks from lagoons are typically 100 to 300 metres, depending upon the prevailing winds.

Facultative Lagoons

Facultative lagoons are intermediate depth (1 to 2.5 m) ponds in which wastewater is stratified into three zones. The zones generally consist of an anaerobic bottom layer, aerobic surface layer, and an intermediate zone. Stratification results from solids settling and water temperature/density variations. Oxygen in the surface stabilization zone is provided by natural re-aeration and photosynthesis. This is in contrast to the aerated lagoons in which mechanical aeration is used to create aerobic surface conditions. In general, the aerobic surface layer serves to reduce odours, while providing treatment of soluble organic byproducts of the anaerobic processes operating at the bottom.

Sludge at the bottom of facultative lagoons will undergo anaerobic digestion in producing carbon dioxide and methane. Photosynthetic activity at the lagoon surface produces oxygen diurnally, increasing the dissolved oxygen during daylight hours, while surface oxygen is depleted at night. Facultative lagoons should be operated in series for optimum performance. Recirculation rates of 0.5 to 2 times the influent have been used to improve overall performance. Most suspended solids settle out rapidly near the inlet of a primary cell, reducing the actual BOD loading on the pond. Additional depth should be provided in the primary cells for anaerobic digestion and storage of settled solids. In colder climates, additional depth should be considered for wastewater storage when ice cover, ice break-up, or thermal conditions may prevent the effluent from meeting stipulated quality standards.

Typically, lagoons are limited to seasonal discharge (once or twice per year), either through spray irrigation or discharge to a watercourse. These discharge requirements result in large volumes of storage required and the need for large areas of land.

Because of the low organic loadings in the partially aerobic conditions during the summer, BOD removal ranging from 75 to 90 percent can be accomplished in lagoons. Effluent may be high in suspended solids, and this is normally attributable to the presence of high algae populations in the ponds. The algal growth rate is slow in the late spring, and in the fall, pond contents become stable and distinct thermal layers prevent mixing. These times of year are usually optimum for pond drawdown. Periods during the spring and fall may be satisfactory for discharge of lagoon contents depending on the requirements of the regulatory agency and the characteristics of the receiving stream. Odours may be noticeable in the spring due to anaerobic activity during the winter and spring turnover.

Estimated effluent characteristics for facultative lagoons with seasonal discharge are approximately 20 to 60 mg/L BOD_5, and 20 to 100 mg/L suspended solids. Effluent from most facultative lagoons does not consistently meet a secondary level of treatment, and is not compatible with the effluent requirements likely to be required by the MOEE without additional effluent treatment/polishing. Single-cell ponds are not as efficient as multi-cell series ponds in reducing algal and bacteria concentrations, colour and turbidity. Phosphorus removal will generally occur through settling and can be enhanced through chemical addition, polishing ponds or effluent filtration.

Aerated Lagoons

Aerated lagoons are medium-depth basins designed for the biological treatment of wastewater on a continuous basis. In contrast to facultative lagoons, which obtain oxygen from photosynthesis and surface re-aeration, aerated lagoons employ mechanical or diffused aeration devices to supply supplemental oxygen to the system. Two general types of aerated lagoons may be considered for use in small systems: completely-mixed and partially-mixed.

Completely-mixed aerated lagoons (aerobic) maintain all of the incoming solids and biological solids from waste conversion in suspension. The essential function of this type of aerated lagoon is waste conversion. Depending on the detention time, the effluent will contain about one-third to one-half of the value of the incoming BOD in the form of solids. Before the effluent can be discharged to a receiving stream, however, the solids must be removed by settling.

Partially-mixed aerated lagoons (aerobic-anaerobic) employ aeration devices to maintain aerobic conditions in the upper zone of the pond. A large portion of the incoming solids and the biological solids from waste conversion settle to the bottom of the lagoon, where they eventually undergo anaerobic decomposition. The partially-mixed aerated pond is particularly useful in Canada because aerobic oxidation can be continued under ice cover. Partially-mixed ponds are designed to maintain a minimum of 2 to 3 mg/L of dissolved oxygen in the upper zone of liquid. An important consideration in the design of aerated lagoons is the aeration equipment. Aeration devices may be mechanical (surface aerators) or diffused air systems. Surface aerators are divided into two types: cage aerators and the more common turbine and vertical shaft aerators. The many diffused air systems utilized in lagoons consist of plastic pipe supported near the bottom of the cells with regularly spaced holes drilled in the tops of the pipes. Aeration devices may be adjusted to maintain the dissolved oxygen level at greater than 2 mg/L, and turbulence may have to be controlled to meet varying operating conditions. Increasing the air supply may correct persistent odour problems in lagoons.

Aerated lagoons normally discharge continuously. The biological solids produced in aerated ponds do not settle readily, thus inhibiting the production of a consistently high quality effluent.

As with facultative lagoons, aerated lagoons are typically constructed by excavating the insitu materials and/or constructing berms/dykes of impervious material. Lining of the lagoons with clay or synthetic material may be required if porous materials are present.

Effluent from most aerated lagoons does not consistently meet a secondary level of treatment, and is thus not compatible with the effluent requirements likely to be required by the MOEE without additional effluent treatment/polishing. The following represents a summary of typical effluent criteria for aerated lagoon systems:

- BOD - 60% to 90%
- COD - 70% to 90%
- TSS - 70% to 90%

Within an aerated lagoon, phosphorus removal is accomplished by settling and is generally poor. However, this performance can be enhanced through chemical addition and polishing ponds.

Biological Aerated Filters (BAF)

The biological aerated filter process is a recent innovation in the biological treatment field. The BAF process generally consists of an aerobic tower with packing material, process aeration equipment, a secondary clarifier, and a backwash system. The filter tower consists of an activated biological zone and a solids removal zone. In the active biological zone, wastewater is passed downward and air is introduced horizontally and vertically. Once the filtering capability of the solids removal zones becomes ineffective, backwashing is initiated. Backwash water is recycled to the influent end of the treatment plant.

In general, the BAF process consists of a granular media filter, generally of expanded shale through which pre-treated wastewater is passed. A number of nozzles pass through an underdrain plate that supports the media. Process air is supplied to the bed above the underdrain plate. The process provides for the removal of organic carbon, suspended solids, and nitrification. Solids and biomass are stored in the media and removed by backwashing (USEPA, 1980a).

In Canada, there are currently three units in operation in Quebec, and at least two more are in the design phase. The largest plant in Quebec began operation in May 1991, and is located in Sherbrooke. The plant is rated at 80,000 m^3/d, servicing a population of approximately 120,000 people.

Tertiary Treatment

Phosphorus Removal

In view of the potential accumulative effects of phosphorus on the receiving watercourse, phosphorus effluent criteria for CWTS must conform to the criteria established by the MOEE on a site-specific basis.

Supplemental processes must therefore be incorporated to reduce phosphorus to levels compatible with the effluent criteria for surface water discharge. Biological phosphorus removal mechanisms are likely sufficient to meet effluent standards of 1 mg/L or greater (Sedlak 1991). However, removal to 0.18 mg/L may be difficult to consistently achieve in small-scale applications without a tertiary level of treatment. Tertiary processes which have been reviewed include the following:

- Polishing Ponds
- Constructed Wetlands
- Physical/Chemical Treatment

Polishing Ponds

Polishing ponds can be used to increase the level of phosphorus removal through advanced settling, either through retention and/or the addition of coagulants. Polishing ponds are typically used with lagoon systems, but can also be utilized as a supplementary process to any mechanical sewage treatment facility. The land requirement for polishing ponds is extensive and open ponds are typically susceptible to such problems as odour and breeding of mosquitoes and other insects. Natural biological processes occur within the polishing ponds, however, the level of treatment is unpredictable and difficult to control. Polishing ponds are not considered to be an appropriate tertiary treatment technology for CWTS in view of the land requirement and lack of certainty with respect to effluent quality. They do, however, complement mechanical primary/secondary treatment processes because they are passive systems with few operational difficulties.

Constructed Wetlands

Constructed wetland systems for wastewater treatment include natural and artificial wetlands. Natural wetlands, both marine and freshwater, have inadvertently served as natural wastewater treatment systems for many years. However, in recent years, constructed wetland systems have been successfully utilized as managed natural systems for polishing pre-treated municipal wastewater effluent under controlled conditions. Constructed wetlands can be designed to meet specific project conditions, while providing new wetland areas that also improve available wildlife, wetland habitats and other benefits. Managed plant communities in constructed wetland as well as natural and constructed marshes, swamps and bogs have been demonstrated to reliably provide pH neutralization and reduction in nutrients, heavy metals, organics, BOD_5, COD, suspended solids, Fecal coliforms and other pathogenic bacteria.

Contaminant removal in wetland systems is accomplished through bacterial transformations and physiochemical processes including adsorption, precipitation and sedimentation. Nitrogen is removed through the nitrification / denitrification process.

In demonstration facilities, the following levels of removal have been reported for secondary effluent treatment (10 day retention) (USEPA, 1980a):

- BOD_5 - 80% to 95%
- TSS - 29% to 87%
- COD - 43% to 87%
- Nitrogen - 42% to 94% (temperature dependent)
- Total P – 60% to 94%

Wetland facilities have been demonstrated to be a viable method of treatment for municipal wastewater. There are several applications in Ontario that are currently monitored for the MOE.

Physical/Chemical Treatment

Independent physical / chemical processes utilize methods other than biological treatment to obtain necessary total phosphorus effluent qualities. Typically, these systems use combinations of chemical addition with clarification or filtration. A chemical coagulant such as alum or ferric chloride is used to react with the alkalinity and phosphate in wastewater to form insoluble salts. The colloidal particle sizes produce well flocculated precipitate, which carries the phosphate precipitate, and can be removed through settling or filtration. Chemical coagulation using alum or ferric chloride is commonly used in water treatment and has more recently been used to assist wastewater treatment. Alum or ferric chloride coagulation may be incorporated into independent physical / chemical treatment, as part of tertiary treatment schemes. Either solids contact clarifiers or gravity / pressure filtration may be used. Solids contact clarifiers or separate flocculation vessels are used for the treatment of either raw wastewater or secondary effluent. Alum or ferric chloride feed can be introduced at a number of points within the process to allow for effective coagulation and settling.

The reduction of total phosphorus through physical / chemical processes is relatively reliable for small- and large-scale applications (Sedlak, 1991). Compared to biological treatment, physical / chemical processes have a number of advantages:

- Quickly brought into operation and easy to restart, where biological processes require seeding and acclimatization

- Less subject to upset from temperature changes

- Very high effluent quality can be attained

Primary disadvantages of physical/chemical processes for phosphorus removal include the increase in sludge mass and volumes produced and related sludge handling/management issues, and the increased capital and operating and maintenance costs related to this level of tertiary treatment.

Nitrate Nitrogen Removal

Nitrogen is an element in the life processes of all plants and animals. Its chemistry is complex because of forms it can take, and the fact that changes can be brought about by living organisms. Nitrogen compounds are released from human and animal bodies through urine and faeces. Urine contains nitrogen resulting from the metabolic breakdown of protein. This nitrogen exists primarily as urea, which hydrologizes rapidly to ammonium carbonate. Faeces contain unassimilated protein matter of organic nitrogen. The organic nitrogen is converted to ammonia by bacteria in the receiving environment. If ammonia is not taken up by plants, it is oxidized by autotrophic bacteria to nitrite. In return, nitrites are rapidly oxidized by the nitrobactor group into nitrate. This biochemical process takes place under aerobic conditions called biological nitrification, and is typical of most secondary treatment processes. Under anaerobic conditions, bacterial activity can reduce the nitrates back to nitrite, and in some cases, back to ammonia. Commonly, the nitrites are reduced to nitrogen gas, which is released to the atmosphere to complete the nitrogen cycle. This process is called biological denitrification.

Various methods of nitrogen removal are available, including the following:

- Biological Nitrification/Denitrification
- Natural Processes (e.g. absorption beds/peat filters)
- Constructed wetlands

Biological Nitrification/Denitrification

Various methods of physical/chemical treatment have been used for the removal of nitrogen from municipal wastewater, including ion exchange, reverse osmosis, chlorine oxidation and air stripping. Each of these processes can remove up to 90 percent or more of the nitrogen in wastewater, but have high power requirements and a need for intensive operator control. As such, these methods are not practical for small-scale applications.

Nitrogen can be removed biologically through plant uptake, bacterial assimilation, or denitrification. Biological denitrification appears to be the most practical method of removing nitrogen at a small-scale, as conditions can be closely controlled. Biological denitrification is a complex yet relatively passive process that can be easily adapted to CWTS applications.

Denitrification occurs through the biochemical reduction of nitrate nitrogen to nitrogen gas under anaerobic conditions. Wastewater must first be nitrified, which typically takes place in the secondary aerobic process. However, the aerobic biological treatment process removes the carbohydrates necessary for cell synthesis by denitrifying microorganisms. Therefore, a carbon source must be added to the nitrified water by either using untreated wastewater or providing a chemical source ahead of the denitrifying stage.

Biological nitrification and denitrification are extremely temperature dependent, which will reduce the nitrogen removal efficiency in cold climates unless measures are taken to control the environment. The denitrification reactor can be easily added as a process stage to a CWTS to allow nitrified wastewater from the secondary process to pass through a mixed anaerobic vessel containing denitrifying bacteria. The denitrification reactor should be covered but not airtight to ensure anaerobic conditions and minimizing surface re-aeration.

This technology is well developed at a full-scale, but is not in widespread use at a small-scale, with little reliable data. A number of experimental denitrification reactors have recently been installed, including modifications to the RBC process through the use of submerged reactors or the expansion of the SBR process to allow for an anoxic phase. Large-scale applications have proved that the anoxic reaction is capable of reducing 80 percent to 98 percent of the nitrate and nitrite entering the system to gaseous nitrogen (Sedlak, 1991).

Natural Processes (Absorption Beds/Sphagnum Peat Filters)

Natural processes can occur in the soil system of conventional absorption trenches which remove nitrogen from the waste stream, however the degree of removal is difficult to predict and monitor. The range of nitrogen removal that can be expected is 10 percent to 40 percent. Because of the uncertainty, it is usually assumed that all of the nitrogen entering a conventional onsite absorption trench will eventually reach the groundwater. A high degree of nitrification occurs in the aerobic zone in the upper level of the tile field, however, denitrification is wholly dependent upon the concentration of carbon in the effluent and a suitable anaerobic environment. For these reasons, it is felt that conventional absorption trenches cannot be relied upon to achieve the required reductions in nitrates.

The sphagnum peat sewage disposal system (as discussed above) was developed and tested in Maine primarily for use with single-family residential units. Recently, an experimental installation in Ontario has shown good promise in reducing nitrate nitrogen levels.

Conventional absorption trenches and / or experimental beds, including sphagnum peat disposal systems, are not considered to be practical for the proactive reduction of nitrates in the groundwater at a communal scale at this time. Although the sphagnum peat bed system shows much promise, sufficient information is not yet available with respect to long-term effluent performance and use at a communal-scale level.

Constructed Wetlands

(see previous discussion on wetland systems)

Disinfection

Disinfection should be provided for all CWTS facilities discharging to a surface water environment. A number of disinfection methods are available; however, chlorination / dechlorination, ultraviolet disinfection and ozonation are considered to be the three most practical methods for small-scale applications.

Chlorination/Dechlorination

Chlorination is the most commonly used wastewater disinfection process. The process involves the addition of elemental chlorine or hypochlorite to the wastewater, which combines with the wastewater to form hypochlorous and hydrochloric acids. The difference between the amount of chlorine added to the wastewater and the amount of chlorine residual remaining at the end of a specified contact period is the chlorine demand. Chlorine demand for any given water or wastewater varies with the amount of chlorine applied, the desired residual, the time of contact, the temperature, pH, and the amount of chemical and organic contaminants in the wastewater. Chlorine compounds used most commonly in wastewater treatment are calcium and sodium hypochlorite and chlorine gas. Calcium and sodium hypochlorite are usually used in small treatment plants because of the relative safety in handling.

Since the early 1970s, attention has focused on the effects of chlorinated effluent, as free chlorine and chloramine residuals are toxic to fish and other aquatic organisms. As such, dechlorination of the wastewater must now be considered. Dechlorination processes applicable to CWTS include holding ponds and the addition of sulphur compounds.

Chlorination is considered to be a practical disinfection method for CWTS if dechlorination is not required. Should a non-toxic effluent be required, ultraviolet disinfection may be more appropriate. The MOEE has recently been discouraging the use of chlorination with dechlorination.

Ultraviolet Disinfection

Ultraviolet disinfection relies on the absorbance of UV energy by the genetic material of the cells within wastewater. The damage it causes results in the inability of the cells to replicate. Ultraviolet equipment is designed to have a number of lamps arranged in a reactor at centreline spacings ranging from 5 to 10 cm. The lamps are inserted into quartz sleeves, and are submerged in the wastewater to provide maximum contact with the liquid. The size of the system is defined by the number of lamps. An open channel modular system design is the most common configuration for treated wastewater applications, and generally comprises several gravity flow, open channels which hold one or more banks of lamps in series. Lamps are placed in the channel as modules that can be easily removed for maintenance and repair. UV disinfection is applicable to effluent treated to secondary levels or better.

UV disinfection is particularly applicable as an alternative to chlorination in cases where dechlorination is required or where there are overriding concerns with safety. Coliforms associated with suspended solids can limit the lower levels attainable by the process. Careful consideration should be given to the methods of cleaning the quartz sleeves. Ultraviolet disinfection has been extensively used with small-scale applications.

Ozonation

Ozone is an extremely reactive oxidant and a very effective bactericide and virucide. In order to efficiently transfer ozone from its gas to liquid phase, it should be generated onsite from air or oxygen gas carriers. Unlike chlorination, ozonation can have beneficial impacts on the environment through its elevation of dissolved oxygen levels in treated wastewater. In most cases, any need for effluent re-aeration can be eliminated through ozonation. Ozone residuals can be extremely toxic to aquatic life, however, they are rarely found in effluent discharging to surface waters as ozone dissipates rapidly.

Ozonation systems are relatively complex to operate and maintain compared to chlorination / dechlorination. The capital costs for ozonation equipment are also relatively expensive compared to chlorination and UV disinfection. Operating costs can also be quite high as ozonation is a power-intensive process.

Municipal Waste Water Treatment Plants

Large scale treatment facilities are in use in the EOWRMS area as described in the infrastructure summary. The goal of the EOWRMS study is, however, to examine cost effective treatment alternatives that allow the deferral of capital expenditures. Large-scale wastewater treatment plants are therefore examined from an optimization perspective.

WWTP Plant Analysis Technologies (PAT)

PAT is essentially an optimization study. WWTP process optimization involves the on-site testing of the plant treatment components. The testing is carried out to determine if the process is operating as efficiently as possible. Typical Pat applications are in clarifier flow pattern investigations, aeration equipment oxygen transfer efficiency testing and unit process stress testing.

A successful PAT investigation would determine which plant treatment component was limiting the capacity of the plant to treat wastewater flow and make recommendations in regard to what options were available to increase the capacity at the plant. This normally results in the re-definition of the plants rated capacity in the Certificate of Approval issued by the MOE.

Water Treatment Technologies

Conventional Clarification Processes / Technologies

Conventional Clarification

Conventional clarification normally consists of rapid mix, flocculation, and sedimentation for particle / turbidity removal.

The rapid mix step uses a flash or static mixer to mix the raw water with coagulant that is then injected into the water. Coagulants typically used include ferric chloride, alum, and polyaluminum chloride (PACl). The purpose of the flash mixer is to achieve the initial contact between the water and chemical and to begin the destabilization of particles to form a floc in the next steps in the process.

The flocculation step uses a series of basins with different speed mixers along the process to promote interparticle collisions and production of a large floc for sedimentation. The mixers decrease in speed (energy input) from the inlet to the outlet of the tank. Minimum detention time for this process is typically 20 minutes.

Sedimentation has been used at WTPs for many years as an effective means of treatment to produce a clarified effluent for further treatment by filtration. Conventional sedimentation tanks have long detention time (3 to 4 hours for gravity settling), which can help plant operators adjust to rapid changes in raw water quality in order to maintain an acceptable finished water product. Sedimentation basins also allow a treatment train to accommodate large solids loading as a result of raw water quality changes or algal blooms.

Powdered activated carbon (PAC) can be added at the rapid mix or flocculation steps of the process to control Taste and Odour, but since the PAC tends to settle, the contact time is sometimes less than adequate to achieve good control. PAC can also assist in the removal of organics from the water by adsorption on the carbon, which then settles in the clarifier.

Enhanced coagulation (EC) is typically performed in the conventional coagulation process to increase TOC removal in the process. EC consists of decreasing the rapid mix pH and optimizing the coagulant dose to achieve the best removal possible. All conventional treatment plants will be required to implement EC under the Stage 1 D/DBPR and to meet the required TOC removal standards based on the raw water organic content (TOC) and alkalinity, unless one of the alternative compliance criteria is met.

Conventional coagulation requires a large area to build tanks for flocculation and sedimentation. Surface loading rates must be low (0.5 to 1.0 gpm/ft^2) to achieve proper operation and an acceptable effluent. A considerable amount of mechanical equipment is required to accomplish flocculation and sludge withdrawal from the sedimentation basin.

Dissolved Air Flotation (DAF)

DAF was first used as a pretreatment for conventional granular media in South Africa and Scandinavia in the 1960s and became more widely used worldwide in the 1980s and 1990s. DAF is becoming more common in the US because it provides a cost-effective alternative to conventional sedimentation. DAF has also been successfully used to remove algae.

In DAF, the solids are separated out by floating the floc to the water surface, as opposed to settling to the bottom of the basin. The process introduces air bubbles at the bottom of the contactor to float the floc. The air bubbles are produced by reducing to ambient pressure a pressurized recycle water stream saturated with air. The “float” is scraped from the top of the reactor, and the clear water is removed from a location well beneath the surface.

DAF is particularly effective in removing solids, such as algae, which are close in density to that of water and, thus, are resistant to removal by sedimentation. DAF has been shown to be as effective as conventional processes at removing turbidity and TOC. It is also able to have an effect on some T&O compounds that can be readily stripped by the dissolved air in the water. DAF provides enhanced particle contact that can increase the removal of small particles and small diameter pathogens, such as *Giardia* and *Cryptosporidium*.

DAF is less costly than conventional flocculation-sedimentation for two reasons—the flocculation section is half the size (or less) of a conventional process, and the surface loading of the solids separation part of the process can be as high as 8 gpm/ft^2. Detention times required for both flocculation and clarification are less than in conventional treatment. This results in a much smaller reactor than is possible for a conventional process. DAF also produces a more concentrated sludge than conventional treatment, although the sludge may contain entrapped air and need to be deaerated.

DAF requires much more energy input than conventional treatment, and requires considerably more mechanical equipment to run the system. Adding PAC to the DAF process would result in even less contact time than in conventional treatment, and presumably less effective T&O control.

Upflow Solids Contact Units

Solids contact units are frequently known as upflow clarifiers. They combine rapid mixing, flocculation, and sedimentation in one unit. Solid contact units are designed to maintain a large volume of flocculated solids within the unit, which enhances flocculation by encouraging interparticle collisions. The flocculated solids (solids blanket) are usually maintained at a set volume in the contactor and cohesion of the blanket is achieved through the use of a polymer in addition to the coagulant.

Upflow clarifiers are popular because of their reduced size. Consequently, they are more compact and occupy less land space. Higher surface loading rates than in conventional treatment can be used to produce more water per unit area. One such unit is the Superpulsator®, manufactured by Infilco Degremont, Inc.

Rapid mixing occurs upstream of the unit where coagulant is added to begin the formation of floc. After rapid mixing, a polymer is added to promote sludge blanket cohesion. The coagulated water then enters the unit. The Superpulsator uses a vacuum pump and vacuum chamber to produce a “pulsing” effect within the flocculation zone. The pulsing of the solids blanket expands the blanket and increases the rate of interparticle collisions. Clarification occurs with the use of inclined plates above the sludge blanket that settle the remaining floc. The clarified effluent is discharged at the top of the unit. Solids are maintained in the unit at a set height by use of a solids overflow weir. Solids are overflowed into a hopper and can be removed at a set interval. The sludge hopper is sloped to act as a sludge thickener as well, depending upon the solids residence time. Typical solids concentrations range from 0.5 to 2 percent in the concentrated sludge.

These units do not tolerate rapid changes in raw water quality and hydraulic loading well. Detention time is lower in this unit (1 hour or less at typical loading rates) than in a conventional process, therefore requiring more operator attention during changing raw water quality conditions. A polymer is required at doses between 0.1 to 0.4 mg/L for cohesion of the sludge blanket.

These units have no submerged moving parts or mechanisms, and the sludge blanket is self-leveling. Typical surface loading rates for the Superpulsator can range from 1.5 to 4 gpm/ft^2 for water treatment, requiring much less surface area for equivalent treatment as compared to conventional processes. These units have been shown to be effective at removing turbidity and TOC. Because of the long retention time of solids, use of PAC is particularly effective at removing T&O-causing compounds in these units. Along with T&O-causing compounds, DBP precursors can also be adsorbed in the solids blanket.

Filtration Technologies

Granular Media Filters

Most conventional surface water treatment plants use granular media filters after the coagulation/sedimentation process to produce finished water. Most granular media designs are monomedia or dual media designs using sand, anthracite, GAC, or combinations of two of the media types.

Monomedia filters are usually deep bed GAC or anthracite filters that are run at high loading rates with coarse media (1.2 to 1.5 mm effective size). Typical depths range from

5 to 8 feet. The high loading rates and increased media depth over conventional dual media filters increases the headloss through the filter. However, this increase in headloss is offset by increased media size. Advantages to this type of filter design include reduced headloss and production of more water per unit area versus dual media designs. Deep bed filters also provide flexibility to use different media designs in the future as new regulations and tighter controls are put in place for finished water. Deep bed monomedia filters can produce similar finished water quality as dual media designs. Disadvantages to this type of filter include the increased possibility of particle and turbidity breakthrough in long filter runs, and the additional capital costs for deep filter boxes.

Dual media filters are the most common filters found at water treatment plants today. Most designs are anthracite/sand or GAC/sand. The dual media design is typically a shallow bed with 18 to 24 inches of anthracite or GAC followed by 12 inches of sand. Media sizes can vary to balance the particle removal and headloss, but the most common media size for the sand part of the filter is 0.5 mm (effective size), while the anthracite and GAC can range from 0.8 to 1.2 mm (effective size). Dual media filters exhibit additional headloss as compared to deep bed monomedia designs, but they provide equal finished water quality. The smaller sand media provides a barrier to particle breakthrough at higher loading rates or long filter run times. The finer the media, the greater the protection; however headloss increases with the finer media, thereby reducing filter productivity. A dual media filter is usually less productive than a monomedia filter, but depending upon the filter influent water quality, this may not be an important factor to consider.

Biologically Active GAC Filters (BAC)

BAC filters are used usually with ozonation to provide DBP control and produce a biologically stable filter effluent. Either a deep bed monomedia GAC or dual media sand/GAC filter is used. The filters are usually preceded by ozonation to convert many of the large organic molecules into smaller organic molecule that are readily assimilable by microbiological activity in the filter. Ozone also introduces large amounts of oxygen to the water, creating excellent conditions for microbial growth on the filter media. Biological growth can be supported on GAC, sand, and anthracite because of the surface area available to the bacteria. GAC is most amenable to biological growth because of the rougher surface characteristics than the other granular media types.

Advantages to BAC include:

- Production of a biologically stable filter effluent that reduces regrowth in the distribution system
- Removal of organic precursors to DBPs
- Reduction in the disinfectant demand of the filter effluent, thereby reducing the amount of disinfectant required in the finished water and possibly reducing DBPs
- Removal of ozonated DBPs (bromates)
- GAC Packed Bed Adsorption

GAC is used in packed bed columns downstream of non-GAC filtration or membrane processes to adsorb organic DBP precursors. It is also used in other installations without upstream filtration as an adsorptive media for organics. GAC packed bed columns can provide adsorptive capacity for T&O-causing compounds. Since these type of GAC columns are used in an adsorptive mode, the GAC must be periodically regenerated or replaced (perhaps every 6 months) to retain the adsorptive capacity of the process.

Many factors influence the performance of GAC contactors to adsorb both DBP precursors and T&O-causing compounds. These include:

GAC Particle Size—The smaller the particle size, the greater the surface area and therefore the greater the adsorptive capacity. Smaller particle size means more headloss.

Empty Bed Contact Time (EBCT)—The amount of contact time and bed depth must be big enough to ensure a given level of treatment, and is usually based on bench-scale testing. The typical required EBCT is 10 minutes.

GAC Hardness—The hardness of the carbon media can minimize the losses of GAC during handling and regeneration, as well as during the minimal backwashing that may be required.

GAC contactors can be very expensive in terms of both capital costs and operational costs to regenerate or replace the GAC. The headlosses associated with the contactors can have a significant effect on treatment plant hydraulics. The use of GAC contactors precludes the use of chlorine or chloramines upstream of the process for oxidation or disinfection credit.

Membranes

With the increasingly stringent requirements for better drinking water quality and reduction in use of disinfectants because of health concerns, the drinking water industry has looked into alternative processes to conventional treatment. Membrane treatment is gaining popularity in the U.S. The long-term experience with membranes is limited at this time, but installed capacity is now over 200 mgd worldwide.

Membrane processes can be separated into four basic categories—reverse osmosis, nano-filtration, ultrafiltration, and microfiltration. Reverse osmosis (RO) and nanofiltration (NF) are used to remove dissolved inorganic compounds such as sodium, calcium, and magnesium ions, or dissolved organic compounds such as humic and fulvic acids that make up the primary source of DBP precursors. They operate at transmembrane pressures of about 80 to 1,200 psi, depending upon the source water quality and degree of separation required. Some uses for RO and NF include desalination of seawater and membrane softening, respectively. Ultrafiltration (UF) and microfiltration (MF), on the other hand, cannot remove dissolved materials, and are limited to removal of particles. UF membranes have a nominal pore size of between 0.003 and 0.03 µm, whereas MF membranes have a nominal pore size of between 0.05 and 0.5 µm.

MF membranes, because of the pore size, are limited to removal of *Giardia* and *Cryptosporidium*, while UF membranes have the added feature of removing not only *Giardia* and *Cryptosporidium* but also viruses. NF membranes remove particles but also can remove most DBP precursors and some dissolved salts. RO membranes remove everything the other membranes do, plus almost all dissolved salts.

The cost of installing and operating RO or NF systems make this process cost-prohibitive in many cases. To operate NF or RO on a surface supply, the feed water must be pretreated with conventional treatment or MF/UF to be effective. Typically, NF is used on surface water and groundwater supplies, while RO is usually limited to use in desalinization applications because of costs and operational issues.

The major disadvantage to a RO or NF system is the concentrated wastes produced by the process. The high concentration of the waste stream usually requires some advanced treatment prior to disposal and is not usually able to be recycled into the head of the process. The recovery of the feed water on the RO or NF units is less than in MF or UF units. Typical product water recovery is between 80 and 90 percent.

The earliest commercially available UF and MF membrane systems designed to filter-sterilize liquids are known as pressure-driven, hollow-fiber membranes. The liquid is passed either from the outside to the inside (lumen) of the hollow fiber (outside-in) or from the lumen to the outside of the fiber (inside-out). The hollow fibers are installed in vessels, which provide support for the pressure necessary to drive the liquid through the membrane pores. This type of filter is commercially available from Aquasource (UF) and Memcor (MF). Other suppliers active in the U.S. include Pall (UF or MF), Hydranautics (UF), PCI (UF), and Smith and Loveless (UF). These units use water, air, or air/water backwash systems.

Immersed membranes are a relatively recent development in membrane process configuration. In this process, hollow fiber membranes are installed (immersed) in a raw water vessel and a small vacuum is applied to their downstream side. This process is much more energy efficient and can result in a smaller footprint than pressure-driven configurations. Immersed membranes are available from Zenon (UF) and Memcor (MF). With the Zenon ZeeWeed® Process, air is introduced at the bottom of the membrane feed vessel, which creates turbulence in the tank effectively scrubbing the solids from the membrane surface. Memcor uses air only in the backwash of its immersed membranes.

MF and UF units are now in use or in planning at many locations for potable water treatment. MF and UF costs have significantly decreased in the past few years with the development of the technology. The advantage of a solids separation barrier with a known diameter makes MF or UF a feasible technology for control of microbes and provides effective filtration while achieving reasonable recovery of the product water. Product water recovery for MF and UF membranes ranges from 85 to 95 percent and can be even higher in some cases.

Advantages and disadvantages of membrane treatment compared to conventional treatment are:

Advantages	Disadvantages
Increased particle and turbidity removal	Pretreatment of raw water is necessary to maintain treatment capacity
Reliability of consistent effluent quality	Need to clean membranes using acids or surfactants (new waste stream)
Removal of pathogens (protozoa and bacteria [MF], protozoa, bacteria, viruses [UF])	Production of a more concentrated backwash stream (particles and pathogens)
Ease of automation of the treatment system	Capital costs are still high as compared to most other processes
More flexibility in being able to meet future finished water quality goals	

Oxidants/Disinfectants

The following table lists all the oxidants / disinfectants that should be considered, and their application in water treatment processes. Each section below goes into greater detail about each process.

Applicability of Oxidants/Disinfectants in Water Treatment

Oxidants/Disinfectants	Oxidant	Primary Disinfectant	Secondary Disinfectant
Chlorine	Yes	Yes	Yes
Chloramines	No	No	Yes
Chlorine Dioxide	Yes	Yes	Yes
Ozone	Yes	Yes	No
Ultraviolet Light (UV)	No	Yes	No
Potassium Permanganate	Yes	No	No

Chlorine

Chlorination has been practiced in water treatment since the early 1900s as an effective disinfectant for protection against waterborne diseases. It is relatively inexpensive and provides a residual concentration in a distribution system. Today it is the most commonly used disinfectant in water treatment. For many plants, the source of chlorine is liquefied chlorine gas or a sodium hypochlorite solution. When chlorine gas is applied to water it forms hypochlorous acid (HOCl) and hydrochloric acid (HCl). When sodium hypochlorite is added to water it forms HOCl and a hydroxyl ion (OH^-). Therefore, when chlorine gas is added to water the pH decreases, and when sodium hypochlorite is added to water the pH increases.

Hypochlorous acid can be deprotonated to form hypochlorite ions (OCl^-) depending upon the pH of the solution. At lower pH values (less than 7.6) HOCl dominates, while above 7.6 OCl^- dominates. HOCl is a much more effective disinfectant than hypochlorite ion; therefore optimum disinfection occurs at lower pH values (below 7). Temperature also is a factor in the disinfection efficacy of chlorine. As the temperature of the water increases, so does the inactivation efficacy for all microorganisms.

Chlorine is highly effective as a disinfectant for bacteria and viruses, with limited effectiveness on *Giardia* and virtually no effect on *Cryptosporidium* at the doses typically used in water treatment (less than 5 mg/L). CT requirements for surface waters were promulgated under the SWTR (1989) to ensure inactivation of at least 0.5-log of *Giardia* and 2-log for viruses for conventional treatment plants, and increased log inactivation for other types of plants (i.e., direct filtration).

Chlorine is also highly effective as an oxidant in water treatment for use in iron and manganese oxidation, hydrogen sulfide reduction, colour removal, and T&O control. Chlorine can also increase particle removal and decrease turbidity through filtration by its oxidation power. It is also employed to control algal and biofilm growth in treatment plant basins. Depending upon its intended use, chlorine can be applied at multiple locations in a treatment train. Most commonly, chlorine is applied upstream of the coagulation process, prior to the filters, and as a final disinfectant for the distribution system.

Chlorine has many disadvantages to its use as a disinfectant/oxidant in water treatment. Its oxidation power also acts upon natural organic matter (NOM) to form DBPs in the plant and the distribution system. The most common DBPs formed are trihalomethanes (TTHMs) and haloacetic acids (HAAs). These are currently regulated under the Stage 1 D/DBPR and will be further regulated under the Stage 2 rule. In most cases, the higher the dose of chlorine used, and the further upstream in the process it is used, the greater the formation of DBPs. DBPs can also be influenced by the pH of the water.

High doses of chlorine can cause new taste and odour concerns from the chlorine itself. Chlorine is also a dangerous chemical to handle as both sodium hypochlorite and gaseous chlorine. Gaseous chlorine is also a poisonous gas that is highly regulated by EPA and DOT, as well as local fire and building codes. Many gaseous chlorine facilities require a scrubber system in addition to the process components. Sodium hypochlorite is a corrosive liquid that requires handling similar to other types of corrosive liquids, as well as spill containment and corrosion resistant materials of construction.

Advantages and disadvantages associated with the use of chlorine are:

Advantages	Disadvantages
Best known and most widely used method of disinfection	Chlorine gas or hypochlorite are hazardous substances and corrosive
Least expensive method of disinfection	Produces halogenated DBPs (TTHMs and HAAs), controlled by the dose and application points.
Provides a stable residual for the distribution system	Special materials and containment needed for storage and handling
Effective in inactivation of bacteria and viruses	Taste and odour problems at high concentrations
Can enhance coagulation and filtration processes	Cannot be used as a prefilter oxidant or disinfectant with GAC filter media
Effective oxidant for iron, manganese, colour, and taste and odours	
Effective to control biological fouling in the treatment plant	

Chloramines

Chloramines are formed by the reaction between hypochlorous acid and ammonia to form a more stable disinfectant than free chlorine. Three different species can be formed from the reaction, those being monochloramine (NH_2Cl), dichloramine ($NHCl_2$), and trichloramine (NCl_3). The speciation is competitive between the reactions and depends upon two factors: pH of the water, and the chlorine to nitrogen (Cl_2:N) ratio. As the Cl_2:N ratio increases, the reaction is driven more towards trichloramines. As the pH decreases below 6, dichloramine is favored over monochloramine. Above pH 6, monochloramine is dominant. Monochloramine is the desired form because the other two forms are sometimes associated with T&O issues. Therefore, chloramine formation conditions are best when the pH is at or above neutral with a 3:1 Cl_2:N ratio.

Chloramines are effective as a secondary disinfectant for establishing and maintaining a distribution system residual. The dose requirements for maintaining a measurable residual in the distribution system are less since chloramines are much more stable than free chlorine. Monochloramine has also been found to be effective in controlling biofilms in distribution systems. Chloramines have been found to be adequate disinfectants for bacteria, however they are not effective for inactivation of viruses and protozoa.

Chloramines are much less reactive than free chlorine, thereby reducing the formation of halogenated disinfection by products as well as taste and odour compounds in the distribution system. Normal dosages are in the range of 1.0 to 4.0 mg/L. High CTs are required when using chloramines if any inactivation credit is desired from the process.

Chloramines can also be used upstream of non-biological filters to reduce biological growth in the treatment plant basins. Chloramines are not effective as oxidants for iron and manganese, colour, or primary taste and odour control. Problems can also occur in the distribution system due to excess nitrogen in the finished water that can cause nitrification. Nitrification can cause a loss of chlorine residual and an increase in bacterial counts in the distribution system. This can be controlled via ammonia addition controls and seasonal free chlorine flushing of the distribution system.

The ammonia added for chloramine formation can be either from aqueous ammonia or anhydrous ammonia. This then requires capital costs associated with storage and process equipment to use chloramines in the process. However, the production of chloramines is relatively inexpensive as compared to other disinfectants.

Advantages and disadvantages associated with the use of chloramines are:

Advantages	Disadvantages
Little reaction with NOM to form DBPs, easier to meet D/DBPR	Less efficacy against microorganisms than other disinfectants (need a primary disinfectant)
Maintain a stable residual for a longer period of time than other disinfectants in the distribution system	No oxidation power
Less taste and odour formation in distribution system with monochloramines	Nitrification problems in distribution system
Inexpensive disinfectant	Taste and odours associated with di- and trichloramine
Effective for reducing biological growth in the treatment plant	High CT required for any disinfection credit
	Must be produced onsite (operational and maintenance concerns)

Chlorine Dioxide

Chlorine dioxide has uses as both an oxidant and a primary disinfectant in water treatment. Currently 700 to 900 public water systems world-wide use chlorine dioxide to treat potable water. It is generated by the reaction of sodium chlorite with gaseous chlorine or sodium hypochlorite in a generator located onsite at the treatment plant.

Chlorine dioxide generation and addition to water produces byproducts of chlorite and chlorate, both of which can be harmful to human health. The new Stage 1 D/DBPR regulates both chlorine dioxide and chlorite. The maximum residual disinfectant level (MRDL) for chlorine dioxide is 1.0 mg/L, and the MCL for chlorite is 0.8 mg/L. The formation of chlorite greatly limits the dose that can be applied to surface water. If the oxidant demand of the water to be treated with chlorine dioxide is greater than 1.4 mg/L, the formation of chlorite in the water may exceed the MCL. Chlorine dioxide can also produce taste and odour concerns at residual levels above 0.4 mg/L. Typical doses used in water treatment vary between 0.07 to 2.0 mg/L (USEPA, 1999).

Chlorine dioxide is usually applied at the head of the plant prior to coagulation in a liquid solution. However, it can also be applied in the clarifiers or at an intermediate point following clarification. As with all oxidants and disinfectants, the oxidant demand of the water plays an important role in the application point and potential use of the chemical.

Chlorine dioxide produces chlorite and chlorate as by-products in water that are regulated by the Stage 1 D/DBPR. Chlorine dioxide does not produce halogenated DBPs, and can be used as one mechanism for the reduction of DBP precursors (by oxidation of organic material) in water. However, the possibility does exist for the production of nonhalogenated DBPs that are not currently regulated but may be regulated in the future.

In disinfection, credit based on the "CT" for chlorine dioxide is currently given for *Giardia* and viruses. Chlorine dioxide has been found in many studies to be more effective than chlorine in disinfection of bacteria, *Giardia*, and *Cryptosporidium* over a wide range of pH. For viruses, chlorine dioxide has been found to be equally effective as chlorine in inactivation. With a CT of approximately 4 mg/min/L at 15 degrees Celsius, 1-log *Giardia* inactivation can be achieved (AWWA, 1991). Studies that have been completed have shown chlorine dioxide inactivation of *Cryptosporidium* is possible at reasonable doses, but the dose and residual needed to achieve the CT required for 0.5 to 1-log inactivation may not be possible due to the Stage 1 D/DBPR limits.

As an oxidant, chlorine dioxide can be used to treat taste and odours and oxidize dissolved iron and manganese. Chlorine dioxide can destroy phenolic compounds that cause taste and odours, as well as compounds associated with decaying vegetation and algae. Chlorine dioxide reacts with soluble forms of iron and manganese to form precipitates that can be removed through coagulation. The use of chlorine dioxide for iron and manganese is limited due to the new Stage 1 D/DBPR. In some locations, chlorine dioxide can be used ahead of the clarification process to replace chlorine as a chemical for the control of algae growth in the water treatment plant.

Major equipment that would be required for a chlorine dioxide system includes stock chemical storage and feed systems, chlorine dioxide generators, and feed piping and injection equipment. If a plant uses sodium hypochlorite or gaseous chlorine for chlorine or chloramine disinfection of the finished water, the storage from this system can be used to feed the chlorine dioxide system. A separate storage and feed system must be provided for the sodium chlorite stock solution. Chlorine dioxide generators require careful monitoring of the chemical feed rates and mixture to ensure the most efficient production of chlorine dioxide. If not carefully monitored, chlorine dioxide generation can produce excess chlorine, as well as excessive concentrations of chlorites that cannot be easily removed from the process stream.

Advantages and disadvantages associated with the use of chlorine dioxide are:

Advantages	Disadvantages
More effective inactivation of most pathogens than chlorine or chloramines	Formation of chlorite and chlorate as by-products
Control of taste and odours in water	High doses cannot be used based on Stage 1 D/DBPR for chlorite MCL and chlorine dioxide MRDL
Provides a residual under most conditions	Higher residuals (above 0.4 mg/L) can cause taste and odour concerns
Oxidation of iron and manganese	Chemical costs for sodium chlorite are high
Provides plant control over algae growth	Training, laboratory equipment, and sampling is expensive
Does not produce halogenated DBPs	Must be produced on-site (more maintenance and operational concerns)
Oxidation of DBP precursor material	Usually has higher operational costs than other disinfectants

Ozone

Ozone is one of the most powerful disinfectants and oxidants available for use in water treatment and has been used in Europe since the early 1900s. It has more recently found acceptance in the U.S., with more than 260 plants using ozone. Most of those plants have a capacity of less than 1 mgd and use ozone as an oxidant as opposed to a disinfectant. However, with new and anticipated future regulations, ozone has become more widely accepted as a disinfectant for the inactivation of *Cryptosporidium*.

Ozone is produced by the passing of dry air or oxygen between two electrodes. A high potential (10,000 to 30,000 volts) is applied across the electrodes, which converts some of the oxygen to ozone. Ozone must be generated onsite and used immediately. It has a very short half-life (less than 30 minutes) under normal conditions encountered in water treatment.

Ozone generators can be fed either from dried air from the atmosphere or from a liquid oxygen (LOX) system. Ozone can be generated at a higher percent weight concentration with LOX systems versus dry air systems. Most manufacturers of ozone equipment prefer the use of LOX system for feed gas. However, with the use of a LOX system comes the need for storage facilities and the periodic purchase of LOX. With a dry air feed system, all that is needed is an air dryer and compressor to supply the feed to the ozone generator.

Ozone is used as a disinfectant because of its efficacy against bacteria, viruses, and protozoa at low doses. Typical doses for inactivation range from 1.0 to 4.0 mg/L. Ozone can be applied at various points in the treatment train, although it is usually applied prior to coagulation or filtration. Disinfection is not significantly affected by temperatures or pH found in water treatment. The CT requirements for ozonation are significantly lower than for any other disinfectant.

Since ozone is such a powerful oxidant, it has been found to have many other uses than just for disinfection, such as iron and manganese reduction, taste and odour removal, removal of colour, improvement of downstream processes (coagulation and filtration), reduction of DBP precursors, and increasing the biodegradable dissolved organic carbon (BDOC) in the water. In conjunction with BAC, ozone can provide a significant reduction in DBP precursors. Ozone also does not lead to the formation chlorinated DBPs when applied.

Ozone does form DBPs, most notably brominated species. If bromide is detected in the water, the potential for bromate formation should be measured. Bromates are regulated under the Stage 1 D/DBPR and will be further regulated in the Stage 2 rule. A higher level of bromide in the raw water may preclude many water treatment plants from using ozone. Other DBPs are also formed by ozonation include aldehydes and ketones.

Due to its short half-life, ozone decays quickly and does not maintain a residual for downstream processes. Therefore, ozonation can be used as a primary disinfectant but must be followed by a secondary disinfectant (chlorine or chloramines) for effective control of the distribution system.

Ozone is expensive to install and dangerous to handle, similar to chlorine. Therefore separate facilities are required for the production of ozone and also for storage of LOX if used as the feed gas. The production and feed systems for ozone, as well as the monitoring systems are expensive to install and require extensive training for operators to effectively operate the system.

Advantages and disadvantages associated with the use of ozone are:

Advantages	Disadvantages
Effective inactivation of *Cryptosporidium* at low ozone dosages	Formation of brominated DBPs in waters with bromide
Control of taste and odours	Initial capital costs for equipment are high
Oxidation of iron, manganese, colour	Expensive to generate, and must be produced onsite
Very short contact time required for disinfection, thereby reducing capital costs associated with contactors	Provides no residual (need a secondary disinfectant)
No chlorinated by-products	Need BAC filters to remove BDOC or can cause regrowth problems in distribution system
Disinfection efficacy is not significantly affected by temperature or pH	Ozone is highly toxic
Enhancement of treatment processes to increase particle removal and decrease turbidity	Ozone systems require a high amount of training and skill to operate
Reduction of DBP precursors (more reduction when used with BAC filters)	
Production of a stable effluent (when used with BAC filters)	

Ultraviolet Light (UV)

UV disinfection is a physical disinfection process, as opposed to a chemical disinfection process. It uses electromagnetic energy in the 200 to 300 nm wavelength range to inactivate microorganisms. For many years, UV has been used in wastewater for final disinfection in place of chlorine. Recently, changes in technologies and studies on the effects of UV on protozoa and other emerging pathogens has made UV become an option for primary disinfection for drinking water.

The inactivation of microorganisms is based on the UV dose (mWs/cm^2), which is a product of the light intensity (mW/cm^2) and the exposure time (seconds). The UV dose is analogous to the CT product used for inactivation credit for chemical oxidants. Since the UV dose is primarily based on the light intensity, water quality parameters that have the most effect on UV dose are turbidity and suspended solids that can shield microorganisms from the UV light, and some organic and inorganic compounds that can absorb UV energy.

UV light is generated by a flow of electrons from an electrical source through ionized mercury vapor. UV lamps are classified as low-pressure and medium-pressure lamps. Low-pressure lamps are more efficient than medium-pressure lamps, but the total UV radiation is weaker. Medium-pressure lamps produce 10 to 20 times more UV radiation output than low-pressure lamps. Some low-pressure lamps are classified as low-pressure, high-intensity and have special design features that allow for a higher transmittance of UV radiation at a low pressure in the lamp. The systems currently being tested for use in water treatment are the low-pressure, high-intensity lamps and the medium-pressure, high-intensity lamps.

There are no known installations of UV disinfection systems for primary disinfection of surface water greater than 1 mgd in the U.S. Most of the existing UV systems are for disinfection of small to mid-size groundwater systems and for advanced drinking water treatment. In all cases, there has been no disinfection credit granted by regulatory agencies for these installations for *Giardia* or *Cryptosporidium*. However, based on the results of a USEPA UV Technical Work Group (TWG) in April 1999, UV has been identified as a feasible alternative for primary disinfection in drinking water for inactivation of all types of pathogens of concern. More work is ongoing within the framework of this TWG to further define the regulatory implications of UV disinfection in drinking water.

Previous work has shown that UV disinfection is a feasible disinfectant for bacteria and viruses, thus its use in other industries. Recent research has shown that at reasonable doses of UV (less than 30 mWs/cm^2) 3-log inactivation of *Cryptosporidium* and 2-log inactivation of *Giardia* can be achieved. Recently, there has been concern that UV cannot inactivate *Giardia* as well as previous research indicated. Research has also indicated that higher UV dosages are required for viruses than for *Cryptosporidium*. Further work is needed and ongoing to gain more dose-response data for these two protozoa of concern.

UV disinfection has a major advantage of little or no production of DBPs. Studies have shown that there is no appreciable increase in TTHM or HAA concentrations as a result of UV disinfection at doses that would be applicable in water treatment. However, low levels of formaldehydes and assimilable organic carbon were produced from UV treatment of finished water.

The advantages and disadvantages associated with UV disinfection are:

Advantages	Disadvantages
Effective disinfectant against viruses and *Cryptosporidium*	Little full-scale experience in surface water treatment
Little to no production of DBPs	High equipment and operational costs
Efficacy not as dependant upon typical water quality parameters (pH, temperature) as chemical disinfectants	Does not hold a residual, must be followed by a secondary disinfectant for the distribution system (i.e. chlorine, chloramines)
Identified by USEPA as a viable mechanism for primary disinfection	Effectiveness can be compromised by particle clumping
	Technology is still evolving
	Measurement of transmitted dose (analogous to chemical residual) to measure effectiveness is difficult
	Not yet acceptable to most state regulatory agencies for surface water

Potassium Permanganate

Potassium permanganate is used primarily as an oxidant and is a very poor disinfectant. It can be used in place of chlorine as a means to control some problems such as taste, odour, iron, manganese, algae, colour, and regrowth in the treatment plant. Potassium perman-anate is a very strong oxidizer and is effective in this role as opposed to other oxidants.

Potassium permanganate is made up in a batch or continuous feed using a dry crystalline solid from which a 1 to 4 percent solution is made. The stock solution is usually applied at the head of the treatment train or in the clarifiers to achieve oxidation. No residual is main-ained from the application of potassium permanganate. Any residual can affect downstream processes, especially ozonation, so care should be taken in choosing the appropriate appli-ation point.

In addition to its oxidizing capabilities, potassium permanganate can effectively reduce DBP formation by eliminating the use of chlorine for oxidation and plant maintenance needs.

Potassium permanganate should not be added concurrently with PAC in the front of the plant, because PAC will consume permanganate and make it unavailable for adsorption of organics and for taste and odour control. Potassium permanganate may also increase finished water manganese levels, which may be a concern in some waters. It is a very toxic chemical that is dangerous to handle, and therefore requires special storage and handling procedures.

The advantages and disadvantages associated with the use of potassium permanganate are:

Advantages	Disadvantages
Effective oxidation of iron, manganese, colour, taste and odour	Not an effective disinfectant
Control of treatment plant regrowth	Can be toxic and dangerous to handle
Can reduce some DBP precursors	Requires tight control over dosing to prevent downstream problems
Allows for the removal of chlorination as a preoxidant, thereby reducing the DBP formation potential	
No negative effects on downstream processes if no residual is maintained	

Backwash Treatment and Recycle

Filter Backwash Treatment

Filter backwash water typically represents 2 to 5 percent of the total water processed at a plant. The most applicable technology for any treatment option that uses granular media filters is the treatment of spent backwash water through the use of an equalization basin followed by a clarifier with plate settlers. This is common practice at many water treatment facilities and is currently practiced at the Lake Gaillard WTP.

This process can produce a clarified effluent that can be discharged to a sanitary sewer or local stream (with an NPDES permit), or it can be recycled to the head of the plant. In places where sewers may not exist, such as at the West River WTP, a recycle stream is a feasible option and allows for greater recovery of spent water from the process. It is important to note that by discharging overflow from the washwater lagoons to a supply reservoir, the West River WTP is actually practicing a partial recycle process.

At the Lake Gaillard and Lake Saltonstall WTPs, where permitting, capacity, and cost of discharge to the local sewer is an issue, backwash treatment and recycle is a practical alternative. Recycle becomes more important if the Authority plans to practice filter-to-waste on its granular media filters. The water is easily treated and recycled and can provide significant cost savings as compared to disposal of the filter-to-waste flow.

Recycle of treated backwash water must be handled carefully to ensure that proper treatent is still achieved by the water treatment processes. The recycle stream should be added at the head of the plant prior to flash mixing and should be returned at an equalized rate so that the flow is less than 10 percent of the total influent flow. The solids stream from the equalization and clarification processes needs to be handled by some measure of solids treatment, as discussed later in this section.

Membrane Concentrate Treatment

Membranes form a semipermeable barrier in the water treatment process and produce a concentrated waste of the rejected constituents in the raw water. Treatment of the membrane concentrate from MF or UF systems is usually much easier than that from an NF system because of the smaller pore size and removal of some molecular size compounds that can significantly affect the pH of the concentrate. Membranes are also periodically backwashed, which produces washwater that must also be treated.

Typical concentrate from an MF or UF system can be treated by the same treatment methods as those used for backwash recycle, or another membrane can be used with a high recovery rate to produce high quality filtrate that can be recycled or disposed of. The small volume of remaining concentrate can be combined with other solids residuals (if present) for further processing or possibly disposed of in a sanitary sewer.

The concentrate from an NF system can contain high levels of some ions, organics, and salts, and so the possibility of advanced levels of treatment for the concentrate may need to be considered. Concentrate from NF can be treated again by an NF membrane to further concentrate the waste stream prior to discharge. Disposal options for the NF concentrate include softening and thickening, evaporation ponds, or a sanitary sewer.

A secondary waste stream that must be dealt with is that produced during cleaning of the membranes. Membranes are typically cleaned with low or high pH solutions on a periodic basis to maintain adequate production through the membrane. Treatment of this waste stream is usually done by using a tank for pH neutralization of the spent cleaning solution followed by a sanitary sewer discharge.

Solids Handling Processes

Solid residuals produced by treatment of backwash water or membrane concentrate, as well as residuals produced from coagulation processes, must be treated before final disposal. The treatment methods employed are designed to increase the percent solids of the residuals to decrease the total volume and therefore reduce the final disposal costs. State and local regulations may also influence the level of residuals treatment or the options available to the Authority for treatment of these solids. The solids concentration of metal hydroxide residuals treated by each process type is:

- Thickening less than 8 percent solids
- Dewatering between 8 to 35 percent solids
- Drying greater than 35 percent solids

Thickening

Thickening is usually the first step in reducing the quantity of solid residuals. The effectiveness of the thickening process has a large impact on any downstream solids treatment. The water removed from the solids during the thickening process can be disposed of or recycled to the head of the plant, allowing for additional recovery of source water in addition to any backwash water recycle.

Gravity Thickeners

The most common method of thickening is the use of gravity thickeners. This method is used in most locations as the initial step in treatment. It is currently used at the Lake Saltonstall and Lake Gaillard WTPs. The thickener is usually a circular shaped settling basin that is operated in either a batch or continuous flow mode. The bottom of the thickener has either a hopper or scraper mechanism to remove the thickened solids. Solids can typically be thickened to 1 to 2 percent solids without polymer addition. Addition of polymer can increase the solids concentration as well as the quality of the supernatant for recycle. The thickener can also be used to provide some residuals storage and equalization to achieve constant solids flow to downstream processes.

Gravity Belt Thickening

A gravity belt thickener concentrates residuals by means of gravity drainage on a continuously moving belt. This has been used extensively in the wastewater industry to thicken primary and waste activated sludge to 5 to 11 percent solids. The Authority previously evaluated a gravity belt thickener in comparison to a gravity thickener and found that a maximum of 5.5 percent solids could be achieved by the belt thickener as compared to 2 percent by the gravity thickener. Loading rates on the unit are higher than most other thickening processes, and the space requirements are low. However, there are more O&M requirements and the need for equalization and storage facilities to provide a constant flow to the belts.

Dissolved Air Flotation

DAF can be used not only as a clarification process but also as a method of thickening of solids from the water treatment process. DAF is used in municipal wastewater treatment for solids handling. Some experience in water treatment has shown that DAF can produce thickened hydroxide sludge of 3 to 4 percent solids. DAF units accommodate a higher hydraulic and solids loading rate than gravity thickeners. They also provide better solid/liquid separation than gravity thickeners. DAF units do have high operational costs and have a limited storage capacity. If DAF is included as a clarification process at any plant, the DAF unit will thicken sludge sufficiently to reduce any need to thicken the DAF overflow.

Dewatering

Centrifuges

Centrifugal dewatering of solids is a process that uses the force generated by a fast rotation of a cylindrical bowl to separate solids from liquids. The main types of centrifuges used to dewater WTP residuals are basket and solid bowl. The solid bowl is the most common type used. It is a rotating cylindrical conical bowl with a scroll that rotates with the bowl at a different speed and carries the dewatered sludge to a discharge point. The centrate from the process is discharged from the shell of the bowl. Centrifuges can operate in either a cocurrent or countercurrent flow, although most centrifuges in the U.S. are of the countercurrent design.

Polymers are usually used to help condition the sludge before centrifugation. Typical solids concentrations can range from 15 to 30 percent with feed solids in the 1 to 3 percent range. The space required for the unit is minimal, and centrifugation is a proven technology. However the energy requirements are high, and the centrate waste stream may have a high concentration of suspended solids that may be difficult to treat or recycle.

Belt Filter Press

The belt filter press has been used in the water industry to mechanically dewater various types of residuals. Dewatering occurs in a continuous process that consists of gravity drainage and then compression. Residuals are first conditioned with a polymer and then distributed onto a porous belt for gravity drainage. Then the partially dewatered residuals enter a compression stage, where they are compressed between two belts that apply pressure to the residuals of up to 20 psi.

This technology is usually solids limiting and requires a high dose and careful selection of polymer for good dewatering. Also a constant flow of solids is required for good dewatering. Solids concentrations can range from 15 to 25 percent, although it is very dependent upon the concentration of the hydroxide in the sludge. In sludge with a significant amount of sands and silts, the resulting cake can have 40 to 50 percent solids.

Drying Beds

The Authority has drying beds or facilities at Lake Gaillard and West River WTP. The use of these facilities is advantageous in increasing the solids concentration of residuals. Currently a thickened sludge is applied to the drying beds at a solids concentration of less than 5 percent. Thus, the drying or freeze-thaw beds are used more for dewatering than for drying, and the underflow from the beds is a significant waste stream that must be treated and recycled or discharged. If the drying beds are used downstream of a mechanical dewatering method, the capacity of the beds will increase and the ultimate volume of sludge to be disposed of will decrease. The Authority should consider this option for further analysis.

Solids Disposal

The Authority currently disposes of solids from its lagoons or drying beds offsite and trucks all thickened solids from Lake Gaillard and West River WTPs to the Hartford MDC WWTP. Some of the liquid streams from treatment are discharged to the sanitary sewer.

Options for the future disposal of residuals will depend upon the level of residuals treatment at the plant as well as future state and federal regulations. The state of Connecticut does not allow the land disposal of WTP residuals. However, this could become an option in the future and should be considered. Options should also be considered to limit the amount of liquid waste generated that must be discharged to the sanitary sewer. Sanitary sewers exist at the Lake Saltonstall and Lake Gaillard WTPs, but West River WTP does not have a sewer available for discharge.

Options for future disposal may include:

- Landfill of solids
- Land application
- Trucking and disposal at WWTP

These options may have requirements as to the level of residuals pretreatment necessary before disposal. They should be weighed by the Authority in conjunction with the level of residuals treatment to determine the most cost-effective and environmentally sound practice for final disposal.

Stormwater Management, Stream Protection/Remediation and Erosion Control

Technologies

Porous Pavement

Porous pavement consists of various surface treatments from concrete pavers to porous asphalt. Concrete pavers rely on the paver joints to provide the pervious area for infiltration. Porous asphalt technology involves installation of a pervious, open-graded asphalt wearing course over a base course with large void spaces. The base course functions as a detention reservoir. Rain passes through the wearing course, collects in the void space of the base course, and ultimately drains away by natural infiltration. Porous pavement is suitable for areas such as parking lots, playgrounds, and lightly travelled roads. The effect is to reduce the amount of stormwater runoff that enters the sewer system.

Generally, porous pavement is superior to conventional pavement in terms of traffic safety due to the increased skid resistance, and it is less susceptible to hydroplaning. The impact of porous pavement on the natural environment is essentially the same as for infiltration trenches and basins. Porous pavement diverts a large fraction of the annual runoff volume into the soil, which helps to maintain baseflows, and can prevent serious erosion immediately downstream. The only negative impact is a slight risk of groundwater contamination, and application is restricted to areas where conditions are favourable in terms of soil type, depth of groundwater, land slope, and proximity to water supply wells. This option is suitable because of the permeable soil type that exists in the watersheds, especially in the upper reaches of the drainage basins.

Erosion/Sediment Control

Erosion / sediment control measures reduce the potential for eroded material to enter the sewer system and receiving waters. Erosion / sediment control measures are required at construction sites and storage areas for salt, sand, and other materials comprised of particulates. At construction sites, control measures should include the maintenance of natural vegetation to the extent possible; the use of hay bales and filter cloths along fencing to filter runoff; the use of crushed rock or riprap in drainage channels to help attenuate runoff; the covering of stockpiled materials; and the use of stormwater sedimentation basins to attenuate runoff and provide solids deposition. At storage areas, all stockpiled material should be covered or located within shelters.

Footing Drain Disconnection (to surface discharge)

Disconnection of footing drains from storm sewers and discharging to surface through the use of sump pumps reduces volumes of flow entering the storm sewer system. Very little impact on potential contaminant discharges would result because of the low contaminant levels found in footing drainage. It is not anticipated that the implementation of a footing drain disconnection to surface program would be very successful because of the high cost of implementation and the inherent drainage problems associated with backyard ponding.

Water Quality Inlets

Water quality inlets have become increasingly popular for use in controlling oil, grit, and hydrocarbon loadings that are generally associated with parking-lot runoff. Inlets are designed to store only a fraction of the design storm; however, they separate some of the coarse sediment, oil/grease, and debris in urban runoff. Fine-grained particulate pollutants, such as silts, clay, and associated trace metals and nutrients, are less likely to be removed.

Water quality inlets can typically serve parking lots 0.4 ha or less and are also appropriate for areas with excessive oil and grease, gas stations, roads, or loading areas. The inlets are unobtrusive, compatible with the storm drain network, easy to access, and allow pre-treatment of runoff before it enters the stormwater system. Disadvantages associated with inlets include their limited stormwater and pollutant removal capabilities, the need for clean-outs, and difficulty in disposal of accumulated sediments.

Increase Pervious Areas (Land Use Control)

An increase in pervious areas in a watershed can help to reduce the amount of runoff produced by that area. Increasing open space and/or providing detention facilities can both reduce the rate and quantity of stormwater runoff and decrease the pollution load contribution. Control of lot densities is a specific means of increasing pervious areas in new or re-development areas.

Control of Road De-icers

The best control of road de-icers is the limitation of their use. In a winter environment, such as experienced in Eastern Ontario, limiting de-icing is not a simple task. Efficient use of de-icers on streets would, however, limit unnecessary impacts of de-icing materials on receiving streams.

Control of Fertilizers and Pesticides

The control of fertilizers and pesticides is one of the most overlooked measures for reducing pollution potential from neighbourhood areas. Controls should be put in place to limit the use of fertilizers, pesticides and herbicides and to regulate their application, distribution, storage and disposal. Public education programs are vital to the implementation of this control and could cover information pertaining to:

- Environmental impacts from fertilizer and pesticide application
- Application, storage, and disposal guidelines
- Lawn maintenance programs that eliminate the use of fertilizer and pesticides

The amount of impact this control will have on the quality of stormwater is difficult to predict, although it would be reasonable to assume that runoff quality would be better maintained.

Industrial Runoff Control

Runoff from industrial areas may contain the residue from chemicals that are spilled during handling and storage. Gasoline and oil spills are typical pollutants often found in service areas. Industrial runoff control requires that the runoff from these areas is intercepted and the pollutants separated from the runoff and disposed of.

There is a range of options for controlling pollutants in stormwater that is discharged from industrial sites, for example, provisions for end-of-pipe treatment. Where the stormwater discharges contain significant amounts of pollutants, the stormwater discharge may be diverted to the sewage treatment plant through connection to the sanitary sewer. Traditional stormwater management practices can also be applied to an industrial setting. However, the most cost-effective way of controlling pollutants is to eliminate the pollution source.

Discharge Bylaw Review/Implementation

Discharge bylaws ensure that adopted stormwater quantity and quality controls are effective and adhered to. The impact of this control is immeasurable and is considered to be an important part of any runoff control strategy.

Enforcement of Oil/Grease or Hazardous Material Disposal Bylaws

There are no published articles on the effectiveness of implementing this control, however, the pollution control benefits are apparent. Enforcement prevents oil / grease or hazardous materials from being discharged to storm sewers or drains. The Town of Strathroy does promote specific drop-off locations for household hazardous materials, but the effectiveness of this control is through public education and enforcement of the bylaws. The effect of this BMP is not currently measurable but will be in the future as public education and compliance increases.

The control has a high benefit to cost ratio and can be very effective for a target pollutant source.

Public Education

Public education overlaps with other BMPs such as water conservation, enforcement of anti-litter bylaws and disposal and discharge bylaws and plays a vital part in their effectiveness. Educating the public promotes good practices and helps the public to keep pollution control in mind on a continuous basis.

Extensive effort over time is necessary to successfully implement this control, a measure of its effect is not readily available but it will be measurable in the future. Public education can be a vital part of any control strategy.

Conveyance Controls

Conveyance controls act to store or treat or infiltrate stormwater through various conveyance system designs. The overall impact is to reduce the quantity and improve the quality of stormwater prior to end-of-pipe discharge. Conveyance controls may include:

Infiltration Trenches and Basins (can also be considered end-of-pipe control)

Infiltration trenches are long, narrow facilities, while basins can take any shape. For in situ infiltration to be effective, the groundwater table must be sufficiently low and soil infiltration rates must be sufficiently high. This method encourages recharge of the groundwater table, removes a significant number of pollutants from the stormwater, and can also assist in reducing peak flows in the system, if constructed as a conveyance control prior to end-of-pipe discharge.

There are a number of different types of infiltration basins. Three of these are:

- Complete exfiltration: Runoff can exit the trench only by exfiltrating through a stone reservoir and into the underlying soils. The stone reservoir must be large enough to accommodate entire expected runoff volume.
- Partial Exfiltration: A perforated pipe is inserted near the top of the trench. Runoff will exit the trench if the level rises to the level of the outlet pipe. Storms of less volume than the design storm will be subject to complete exfiltration.
- Water Quality Exfiltration: The storage volume is set to receive only the first flush runoff volume during a storm. The remaining volume is not treated by the trench and is conveyed to the collection system.

Pervious Exfiltration Pipe

A few municipalities (in Ontario, Nepean and Etobicoke) have implemented pervious pipe for stormwater drainage. The pipes are perforated along their length, allowing exfiltration from the pipe into the surrounding soils. The technology is still not widely used.

Pervious pipe applications have experienced significant maintenance problems, usually associated with clogging. Implementation requires some type of pretreatment to remove solids. The pipe itself does not limit contaminant discharge and may in fact facilitate contamination of groundwater resources.

Pervious Catchbasins or Manholes

Pervious catchbasins or manholes are connected to some form of exfiltration storage media. They can be used for near-source control such as roof discharges, sump discharges, and parking lot runoff.

Pervious catchbasins can be combined with some form of pretreatment to remove solids. They may still experience frequent clogging and are considered experimental (MOE, 1994). The catchbasin itself does not limit contaminant discharge and may in fact facilitate contamination of groundwater resources.

Grassed Swales

Grassed swales are generally associated with rural drainage. Some municipalities in Ontario are promoting the design of grassed swales for more urban areas to replace curbs and gutters. Grassed swales remove contaminants through infiltration and filtration mechanisms.

If properly designed and implemented under the proper soil conditions grassed swales can be an effective method of contaminant removal. During low intensity rains most of the precipitation infiltrates in a grass swale.

Problems associated with grassed swales, such as public acceptance in urban areas and maintenance, may not be significant concerns in Eastern Ontario because of the type of rural and low density urban development.

Culvert Sizing

A number of culverts in the study area are already in place. The adequacy of these culverts plays an important role in providing proper conveyance. Properly designed culverts convey stormwater adequately and still provide for ecological enhancement opportunities.

End-of-Pipe Controls

End-of-pipe controls reduce the rate of discharge or the volume of discharge and/or increase the quality of discharge through various system designs. The overall impact is to reduce the quantity or rate of discharge and improve the quality of stormwater prior to discharge to the receiving stream. End-of-pipe controls may include:

Wetlands

Stormwater volumes generated during a rainfall event may be captured and discharged to a natural or constructed wetland area prior to final discharge to a receiving stream. Constructed or natural wetland areas can be used to treat or remove contaminants in stormwater prior to discharge to the receiving waters or as a polishing step for other control measures.

Wetland treatment can be an effective means of stormwater treatment. The use of wetlands for stormwater management is a feasible solution to the problem of water quality. Wetlands also provide flow control during storm events, thus reducing the impacts of water quantity on receiving streams.

Examples of typical projected long-term removal efficiencies in stormwater wetlands are:

- Total suspended solids – 69% to 75%
- Total phosphorus – 45% to 55%
- Total nitrogen – 25% to 64%
- Organic carbon – 15%
- Lead – 75%
- Zinc – 50%
- Bacteria – 2 log reduction

Wetland performance during colder winter months in Canada has been studied by a number of researchers and the results indicate that very effective treatment occurs even during the winter. There may be significant opportunity to implement wetland controls in Eastern Ontario because of the land availability and the desire to enhance ecological features.

Wet Ponds

Wet-pond effectiveness varies depending on the size and type of facility. Studies carried out by USEPA under its National Urban Runoff Program indicate that detention basins can be effective in reducing urban runoff concentrations if properly designed. Removal efficiencies of up to 90 percent for suspended solids were achieved in some of the ponds studied. However, detention basins designed for pure drainage control are relatively ineffective in reducing pollutant loadings. Studies carried out in Ottawa as part of the Rideau River Stormwater Management Study indicated that detention times of approximately three days are required to reduce bacterial loadings by 90 percent. Potential problems exist with the disposal of basin sediments and / or the contamination of groundwater through infiltration.

Wet ponds have already been employed for management of stormwater in the existing developments in the study area.

Dry Ponds

Dry ponds have very few direct water-quality benefits due to scouring of bottom sediments. They do, however, decrease receiving water velocities and peak flows, thus lowering erosion potential and associated impacts on fisheries habitat.

Dry ponds operated in a batch mode can achieve higher removal efficiencies but are operationally less efficient. In general, dry ponds should be implemented only if conditions prevent construction of a wet pond or a wetland.

Watercourse Stabilization (Environmental BMPs)

Ecological Consideration

Protection of the watercourses in the EOWRMS area from erosive forces and encroachment of development into riparian zones is accomplished primarily through the implementation of biotechnical solutions to erosion control and aggressive planting and protection strategies for riparian buffers. Here we examine alternative BMPs for the protection of the watercourse from local scour and for the protection and enhancement of riparian areas that act as green space linkages and serve to enhance the ecology of stream corridors in general. Stabilization of the watercourse and protection of riparian areas will enhance existing ecological features.

Environmental BMPs

There are a number of categories of watercourse stabilization BMPs that, for the purposes of this discussion, can be divided into:

- Channel modifications
- Habitat enhancement and erosion control

Habitat Enhancement and Erosion Control

Stream protection measures work by reducing the force of flowing water that attacks the bed and /or bank, by increasing the resistance of the bank to erosion, or by some combination of the two processes. Stormwater reduction or retention methods, grade reduction structures, and structural designs that create turbulence and reduce flow velocity fall into the first category. Live booms and dead tree or brush retards are examples. Channels lined with grass or stone and streambanks protected by riprap, cellular concrete, or other revetment designs are of the second type. Most designs that employ brushy vegetation, either alone or in combination with structures, protect from erosion in both ways.

The management of a stream corridor through protection or enhancement of greenspace linkages and riparian zones and the protection of the stream from erosive forces can be assessed together as biotechnical alternatives. The following table identifies stream bed and bank protection measures that are appropriate for different erosion problems as well as those measures that enhance the habitat value of green space linkages and riparian zones.

BIOTECHNICAL ALTERNATIVES

Condition or Process Observed	Alternative
Toe erosion and upper bank failure	1. Vegetated geogrid 2. Live cribwall 3. Rock toe with vegetation 4. Conventional riprap
Local bank scour	1. Branchpacking 2. Vegetated geogrid 3. Live cribwall 4. Live fascine 5. Joint planting 6. Tree revetment 7. Conventional vegetation 8. Conventional riprap
Loss of riparian cover or green space linkages	1. Establish Buffer Zones 2. Replant riparian areas 3. Establish protected habitat areas

Conventional Vegetation

Increasing the density of naturally occurring vegetation through a planting program can decrease the severity of erosion. Native plant species develop a root mass, which acts to stabilize the streambank soils. In Strathroy, where there is also a loss of riparian cover, this alternative also enhances habitat values.

Conventional Riprap

Riprap are large rocks and boulders that can be placed along streambanks for erosion protection. They can be very effective in protecting the sites from erosion, but do not provide a natural appearance to the stream if not vegetated.

Live Staking

Live stakes are living, woody plant cuttings capable of rooting with relative ease. The cuttings are large enough and long enough to be tamped into the ground as stakes. They are intended to root and grow into mature shrubs that, over time, will to stabilize the soils. This is an effective stabilization method for simple and/or small problem sites once the roots and vegetation have become established. The roots reinforce the soil mantle and provide surface protection via the top leaf growth. Live staking can also be used for securing natural geotextiles, such as jute mesh. Once they have become well established, live stakes are effective in camouflaging an open area and usually enhance the development of healthy habitat areas over time.

Joint Planting

Joint planting is a system that installs live stakes in between previously placed riprap rock. It is intended to increase the effectiveness of the conventional riprap by forming a living root mat, and water filtering system in the material base upon which the riprap has been placed. This system assists in dissipating energy, and causing deposition to occur along the banks, thus developing a more natural look and function. It provides additional adjunctive protection for high torrent, steep gradient stream systems and is useful for systems carrying heavy suspended sediment load.

Vegetated Geogrid

Vegetated geogrids are meshes made of a variety of materials that are laid and secured along seeded streambanks or bare streambanks to be used with live stakes. They offer some immediate protection from erosion and protect and allow seedlings to develop. The grids form a permanent structure for stem and root reinforcement.

Branchpacking

Branchpacking is the process of alternating layers of live branches and soil, incorporated into a hole or slumped out area in a slope or streambank. The branches form a living tieback into the slope root to form a permanent reinforced installation, while the tips produce vegetative top growth that is intended to reduce erosion. The branchpacking installation produces an immediate filter barrier, reducing gully erosion and scouring condition. It is one of the most effective and inexpensive methods for repairing holes in earthen embankments along small stream sites and providing bank reinforcement.

Live Cribwall

A live cribwall installation is a rectangular framework of logs or timbers, rock, and woody cuttings that can protect an eroding streambank or prevent the formation of a split channel. These are complicated and expensive systems. They are very effective on the outside bends of main channels where strong currents are present. They cause deposition to occur and therefore create a natural toe protection. Live cribwalls are very useful where space is limited. The log or timber framework provides immediate protection from erosion, while the plants provide long-term durability. It provides effective bank erosion control on fast flowing steep gradient streams and is useful on streams that carry heavy bedload.

Live Fascine

Live fascine structures are bound sausage-like bundles of live cut branches. They are tied together securely and placed into trenches along streambanks. The live fascine bundles are installed with live stakes and dead stout stakes. Their use is usually limited to larger streambank protection projects.

Rock Toe with Vegetation

The protection of the toe area of a streambank is in most cases the key element in an erosion control strategy. The toe of the streambank normally experiences the most extreme erosive force and destabilization of the toe can often cause slope failure. Protecting the toe of the slope with rock (riprap) sized approximately in conjunction with upslope vegetation to stabilize the streambanks can be an effective method of erosion control.

Tree Revetment

Tree planting on streambanks is a long-term strategy for erosion control and is not often effective without the use of more immediate bank stabilization techniques. Tree revetments can serve the dual purpose of providing stabilization and increasing the green spacelinkages required to enhance ecological features.

Buffer Zones

"Buffer zone" refers to an area on both sides of the stream that is allowed to naturalize, thus providing habitat and transportation corridors for wildlife movement or greenspace linkages. Buffer zones vary in width depending on the local significance of the stream and encroaching development but normally a minimum of 15 metres on both sides of the stream is recommended as a minimum. A more functional width for greenspace linkages would be provided by allowing for 30 metres on both sides of the stream.

Water Efficiency

Technologies

Home Conservation Kits. Home conservation kits consist of a number of replacement plumbing fixtures and other items designed to reduce indoor residential water use. Usually older homes and multiple-family residences are targeted for such a program because of the greater amount of savings that can be realized. The conservation kits can include:

- Low volume showerheads
- Toilet tank dams or displacement devices
- Low volume faucet aerators
- Toilet tank leak detection tablets
- Teflon tap
- Installation instructions

Low volume showerheads have a flow rate of approximately 10 litres per minute compared to normal showerheads which are generally over 20 litres per minute (1). Toilet tank dams or displacement devices can reduce the volume per flush in standard toilets by as much as 20 to 40 percent (2). Low volume faucet aerators are most effective in kitchen faucets. Reductions in water usage with aerators can be as much as 50 percent (2). Toilet tank leak detection tablets are placed in the toilet tank to determine if water is leaking into the toilet bowl and thus discharging to the sanitary sewer. The tablets are easy to use and can potentially discover sizeable losses. Teflon tape and instructions are usually included in the kits for do-it-yourself installations.

Several methods have been used to distribute the kits and install the fixtures. These methods range from kit pick-up by the home owner, mailing the kits directly to home owners, home delivery, and home delivery with installation. Various levels of effort have also been extended to ensure proper and continued use of the devices by the home owner.

Home Conservation Retrofit. Home conservation retrofit programs include all the devices and strategies in the Home Conservation Kits with the exception of the toilet devices. In a comprehensive retrofit program, the toilet itself is replaced by a toilet with a substantially lower flush volume. The toilets referred to as low volume, very low volume, ultra low volume, or microflush toilets generally reduce the flush volume from approximately 22 litres per flush to approximately 16 litres per flush or as low as 7 litres per flush in some models (3).

Retrofit programs have been carried out on a pilot program basis in a number of municipalities and generally include some monetary incentive and installation by contractors selected by the municipality.

Water Metering and Water Meter Reading. Water metering is essential to a water conservation strategy. Most rate structures are based on water usage and therefore require metering. Metering and subsequent billings also encourages customers to use less water and makes them more aware of the amounts of water they are using and the potential for conservation (4).

Water meters are generally installed on all new buildings as part of ongoing water conservation strategies. Retrofit programs for installation of new meters on existing buildings is also being carried out in a number of municipalities. Water metering applies system wide to residential, industrial, commercial, institutional, and agricultural sectors. Water metering can also include servicing and / or replacement of old meters which tend to read low.

The frequency of meter reading can also impact the use of water by metered customers. The increased frequency allows more effective and visible monitoring of individual water use and can make customers more aware of the impacts of the municipalities and their own water conservation efforts.

Water Recycling and Water Reuse. Water reuse refers to the secondary us of water without the benefit of any water treatment. Water recycling implies some form of treatment is provided to the water prior to its being reused. Water reuse or “grey water option” for residences can include a number of water reuse strategies that focus on the secondary use of water which has already served some function in the home. Water used in the home is separated into that which is suitable for secondary uses, such as shower water, and that which may be too contaminated with solids or grease, such as dishwater, to serve any secondary function. Typical applications for grey water include toilet flushing or garden irrigation with shower water.

Water quality requirements for each application of grey water must be taken into consideration. Also, the plumbing alterations necessary for grey water reuse in most homes would be extensive.

Water reuse in agriculture is usually considered for irrigation purposes.

Water treatment processes that kill bacteria or reduce solids may be an intermediate step in residential or agricultural water recycling.

An increasing trend in commercial facilities, such as golf courses, car washes, and laundry services is to look to cheaper sources of water or to more fully utilize the water they are currently purchasing. A number of options can be open to such facilities depending on specific water quality requirements.

Municipal Bylaws. Municipal bylaws that restrict the use of water or set limits on landscaping design can also be effective in conservation strategies. Restricting the use of water for lawn irrigation and car washing during periods of water shortage may be particularly effective in reducing peak demands. Bylaws that ban the use of once-through cooling water can substantially reduce the use of water, on an average basis, by many industries.

Bylaws can also address new development within a region. New developments within municipalities can be limited, by the municipality, to those that conform to a NO-LOAD (no net increase in water demand) agreement with the municipality. Another method of ensuring water efficiency is to require that a water conservation strategy document accompany any submissions to the municipality for building permits.

Xeriscaping. Xeriscaping refers to landscaping design using plant species that are adapted to dry growing conditions or landscaping that uses less grassed turf areas and ornamental areas that do not require water. Xeriscaping is most adaptable to residential areas although industrial sites and commercial/institutional facilities can also reduce their outdoor use of water by employing this type of landscape design.

Xeriscaping would have an impact on both the long-term average consumption of water and the peak water use periods that occur in the summer months. Water-efficient landscaping could also be incorporated into municipal bylaws.

Public Education. The objective of a public education program is to raise the awareness of the consumer in regards to the facts of water conservation and to development, within the public, a conservation ethic.

The majority of municipal water consumers do not understand the role the municipality plays in delivering potable water to the consumer from a source, through treatment, to the end user, and final treatment and discharge. An understanding of the steps and costs involved in the process and the benefits that can be accrued from a comprehensive water conservation program can enhance the level of participation of the community in saving water.

Public education can take the form of billing notices and fliers, articles within the local media, open houses, video productions or can be more aggressive in the form of seminars and workshops. Regardless of the format, it is important to target specific audiences with any public education program.

Conservation Rate Structure. Rate structures within a municipality can fall into a number of different categories, including:

- Flat rate
- Declining block rate
- Increasing block rate
- Constant rate

Under a flat rate structure, the consumer pays a fixed charge for water use regardless of the amount of consumption. In a declining rate structure, the consumer pays less per unit of consumption as more water is consumed. With an increasing or inverted block rate structure, the consumer pays more per unit of water as his consumption increases and, in a constant rate structure, the consumer pays a constant rate per unit of consumption. It is important to send the correct message to the water consumer. The flat rate, declining block rate, and the constant rate structure do not promote water conservation. The conservation rate structure equation, however, is not a simple one and must also address revenue aspects.

In some municipalities with water shortage problems, a peak season surcharge is also aplied. This enables municipalities to charge those consumers who create the extreme demand during peak periods.

Outside Use Restrictions. The use of water for outside purposes, such as lawn watering and car washing, can have a great affect on peak water demands during the summer months. The restriction of water use for these and other non-essential purposes can reduce the peak demand.

Typically, water use restriction strategies follow a system that limits the use of water for certain segments of the population at different times. An example of one of these schemes is the odd / even water restriction in which customers with odd or even numbered houses can use outside water only on certain dates. At other times, the use of outside water for non-essential purposes can be banned entirely.

Other examples of water restrictions may include limitations on the watering of new grass, sod, or the use of water for ornamental fountains, etc.

Leak Detection Program. Leaks in municipal water supply systems are an inevitable part of a water distribution system. The amount of leakage that is acceptable can, however, be exceeded. Most municipal distribution systems can experience system losses of approximately 5 to 10 percent or more. The level of water loss at which it becomes cost-effective to detect and repair these leaks is generally in the range of 10 to 15 percent.

Pressure Reduction. Pressure reduction in the distribution system reduces the amount of water use by consumers system-wide. The reduction of pressure can be accomplished system-wide, at a subdivision development level or at individual housing units.

In existing distribution systems that have been designed to function at a particular pressure, there are a number of considerations that must be looked at which limit the effectiveness of this measure including the pressure requirements of essential services such as fire fighting.

Pressure reduction is usually considered an option for new developments or individual buildings that can be designed to operate under the lower pressure conditions.

Water Audits. Water audits are carried out to determine the nature of water use and water conservation / efficiency options that have the potential to reduce the amount of water used at any one particular facility or residence.

Water audits can be conducted at any residence, industrial, commercial, agricultural, or institutional site. Water audit efforts are most often concentrated on large volume water users such as industrial processing facilities and large institutional complexes where the potential water savings are the highest.

Annual Water Conservation Budget. Adequate annual water conservation budgets are important to the continued success of any water conservation / efficiency efforts. The education components provide continued incentive to use water efficiently and affect a gradual change in consumer behaviour patterns. Support for efficiency initiatives must be ongoing if the impact of a water-efficiency strategy is to be maintained.

Annual budgets for water efficiency can be incorporated into the water rate structure.

Inventory of Unmetered Water Use. An inventory of unmetered water use is an important component in the overall water balance analysis for a municipality. A thorough knowledge of the unmetered water uses can assist municipalities in apportioning an appropriate level of effort to leak detection and repair and to more accurately forecast future water demand.

FIGURES

Eastern Ontario
WATER RESOURCES MANAGEMENT STUDY

Final Report

Prepared for

United Counties of Prescott and Russell
United Counties of Stormont, Dundas and Glengarry
City of Ottawa

Prepared by

CH2MHILL

March 2001

"Working with Eastern Ontarians to develop a common understanding of regional water resources issues and a strategy to use comprehensive information and analysis to manage these resources for sustainable development."

Figures – Eastern Ontario Water Resources Management Study Final Report

The Eastern Ontario Water Resources Management Study Final Report is provided under separate cover.

"Working with Eastern Ontarians to develop a common understanding of regional water resources issues and a strategy to use comprehensive information and analysis to manage these resources for sustainable development."

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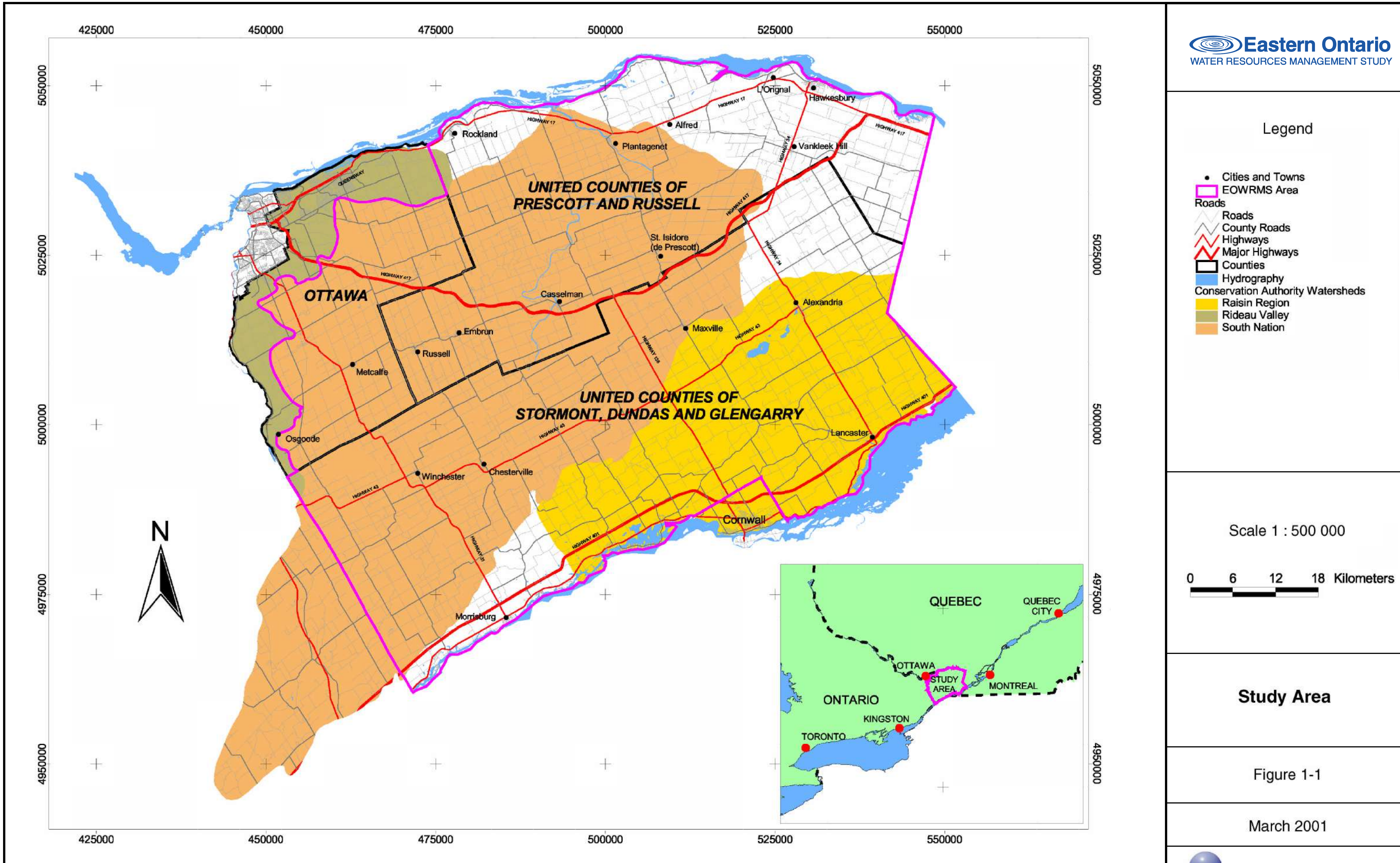

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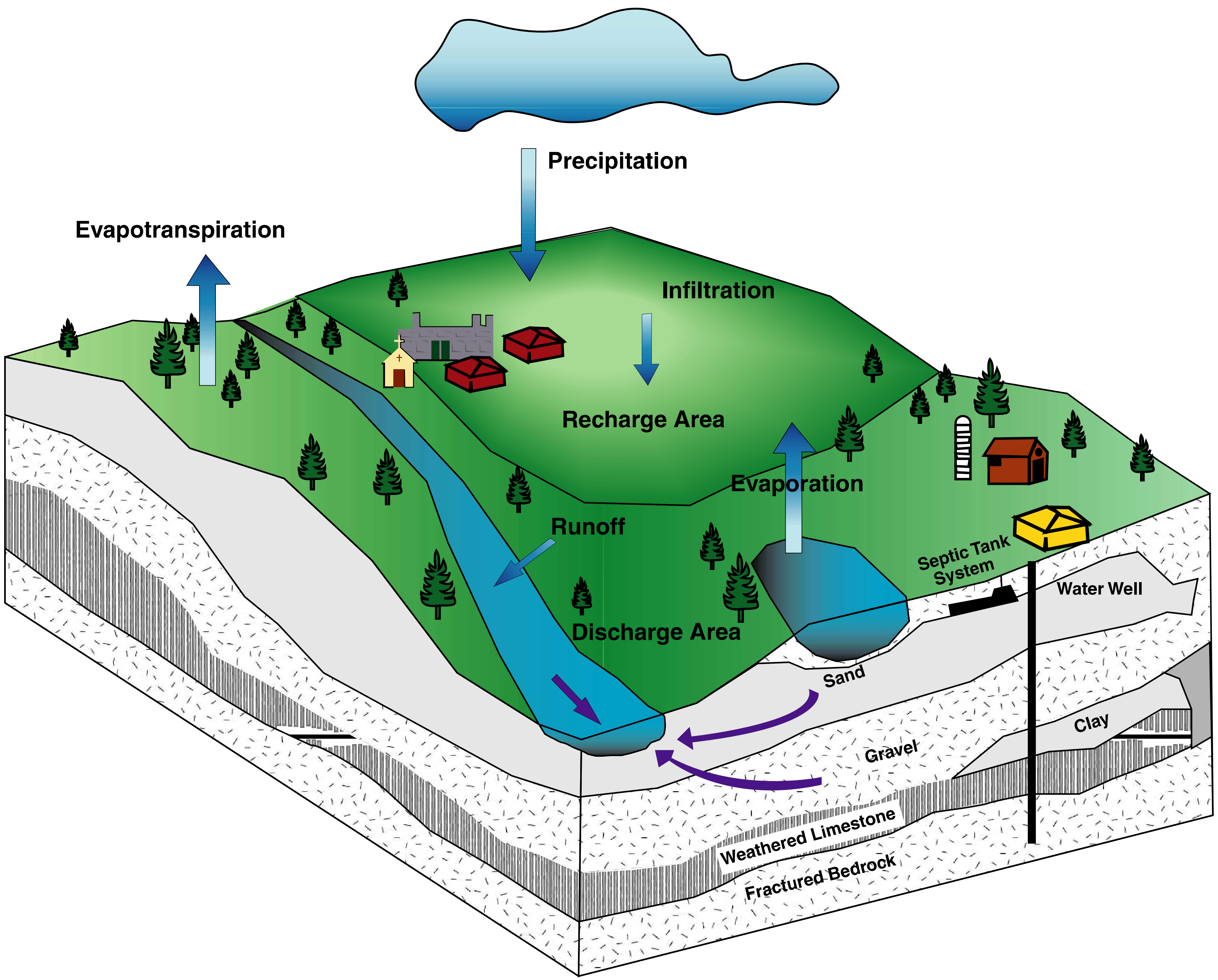

Eastern Ontario
WATER RESOURCES MANAGEMENT STUDY

Hydrologic Cycle

Figure 3-1

March 2001

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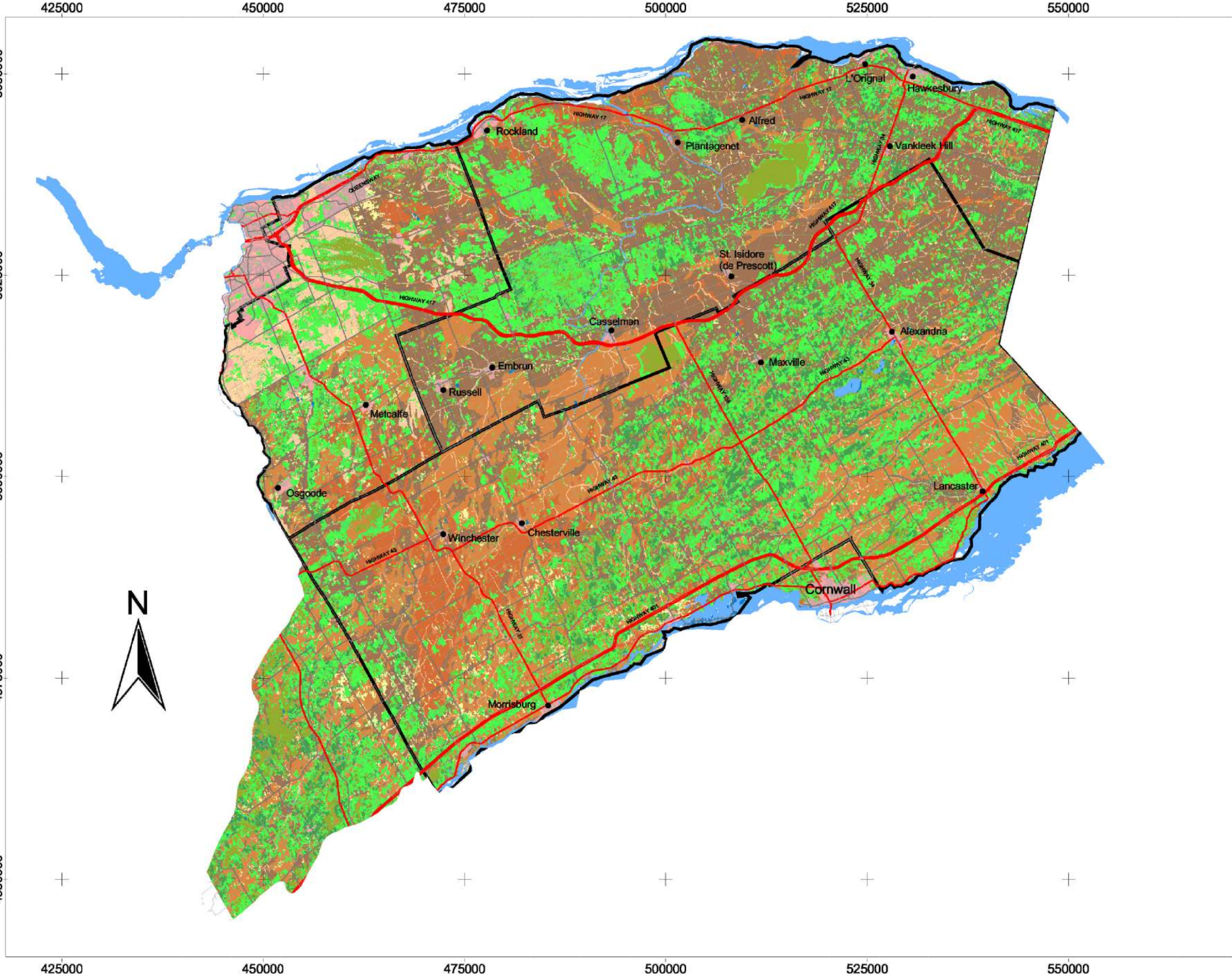

Eastern Ontario
WATER RESOURCES MANAGEMENT STUDY

Legend

- Cities and Towns

Roads
- Roads
- County Roads
- Highways
- Major Highways
- Counties
- Hydrography

Evapotranspiration
- Urban (150 mm/yr)
- Soil - Coarse Texture (270)
- Soil - Medium Texture (390)
- Soil - Fine Texture
- Soil - Unclassed Texture (330)
- Soil - Unclassed Texture - Gloucester (334)
- Open Areas (335)
- Forest - Conifer (445)
- Forest - Mixed (541)
- Forest - Deciduous (638)
- Forest - Unclassed (577)
- Open Water (640)

Scale 1 : 500 000

0 6 12 18 Kilometers

Estimated Evapotranspiration

Figure 3-2

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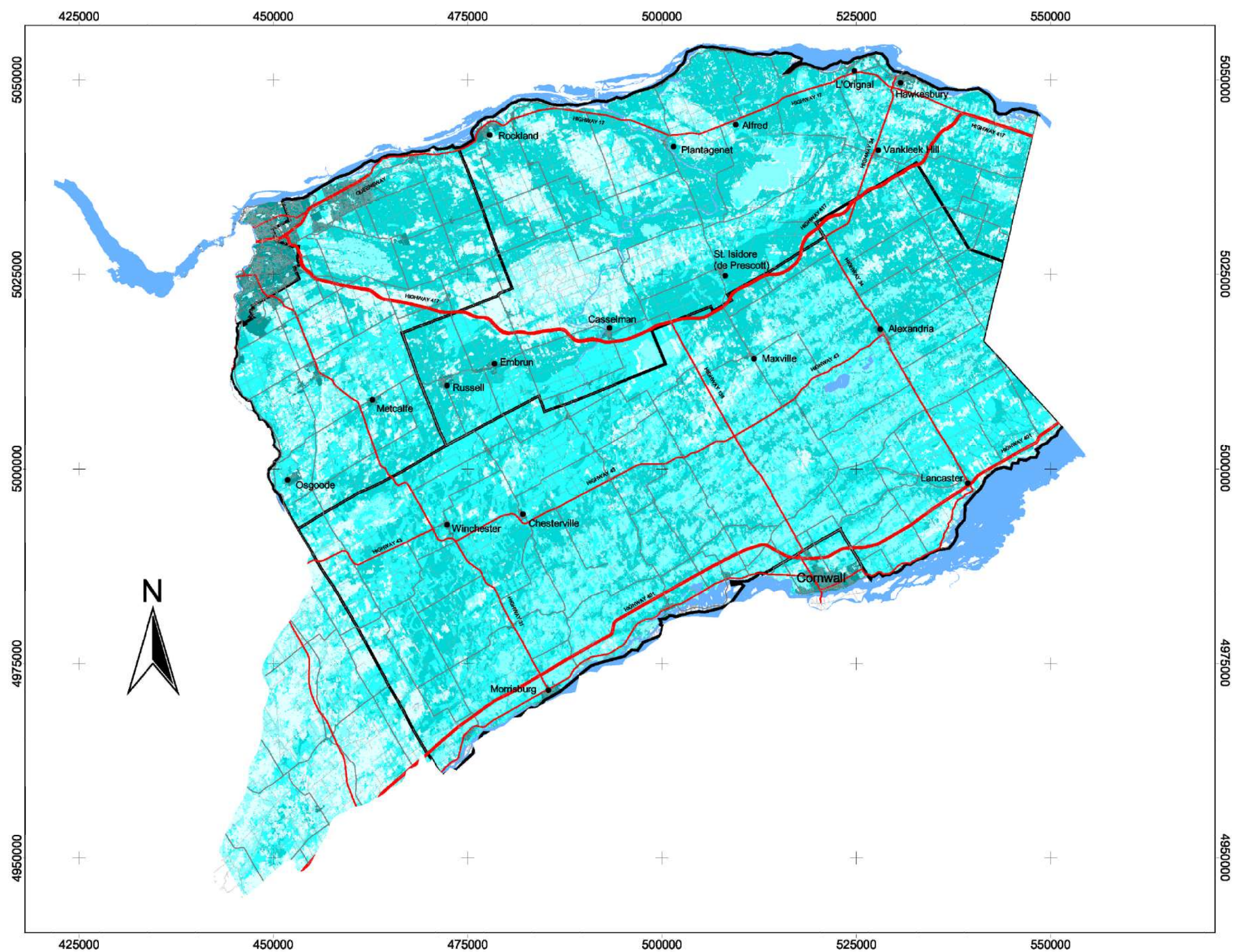

Eastern Ontario
WATER RESOURCES MANAGEMENT STUDY

Legend

- Cities and Towns

Roads
- Roads
- County Roads
- Highways
- Major Highways
- Counties
- Hydrography

Water Budget - Surface Water (mm/yr)
- 170 - 300
- 300 - 430
- 430 - 560
- 560 - 690
- 690 - 820

Scale 1 : 500 000

0 6 12 18 Kilometers

Surface Water Contribution

Figure 3-3

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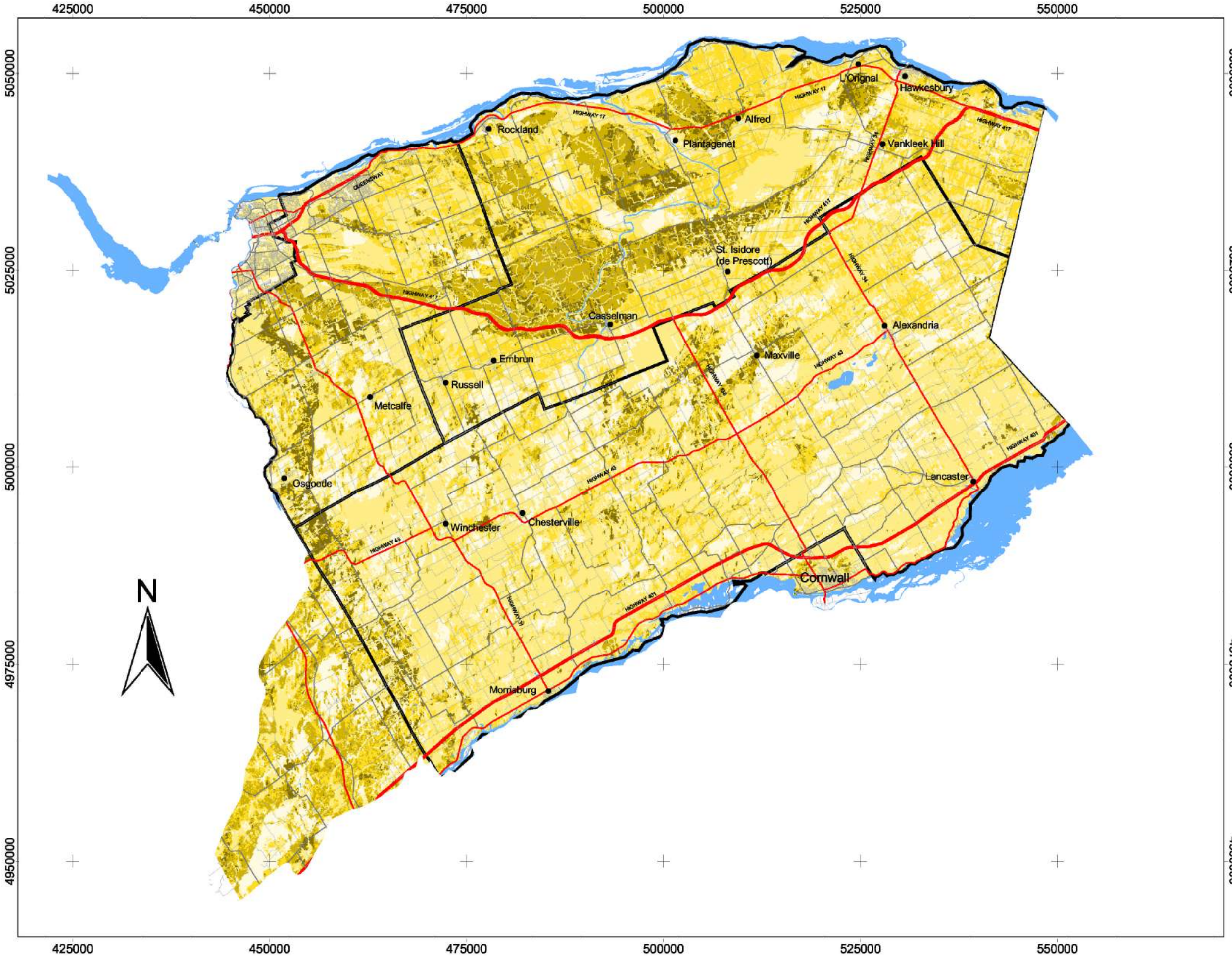

Eastern Ontario
WATER RESOURCES MANAGEMENT STUDY

Legend

- Cities and Towns

Roads
- Roads
- County Roads
- Highways
- Major Highways
- Counties
- Hydrography

Water Budget - Groundwater (mm/yr)
- 0 - 10
- 10 - 30
- 30 - 60
- 60 - 100
- 100 +

Scale 1 : 500 000

0 6 12 18 Kilometers

Groundwater Contribution

Figure 3-4

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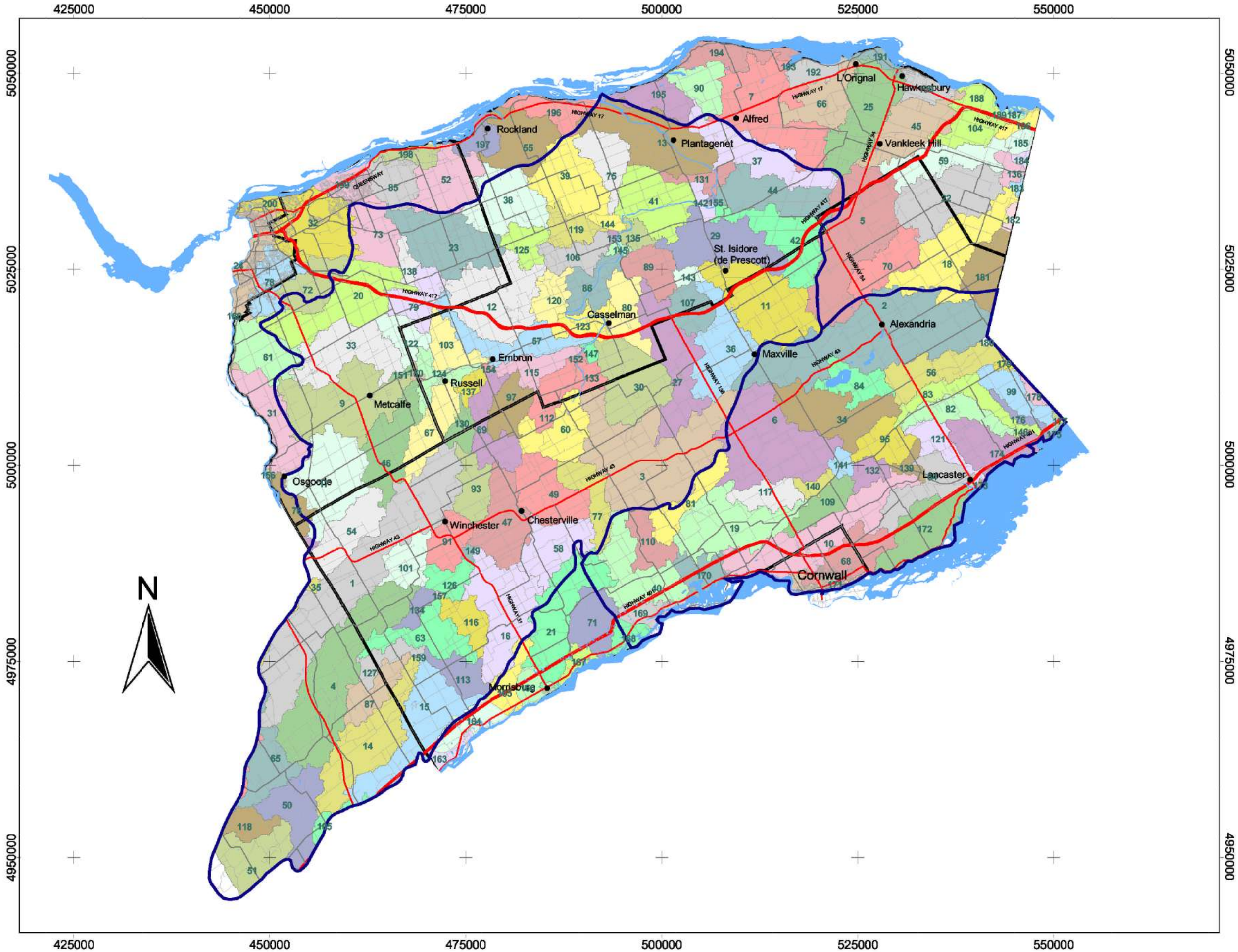

Eastern Ontario
WATER RESOURCES MANAGEMENT STUDY

Legend
- Cities and Towns
- Hydrography

Roads
- Roads
- County Roads
- Highways
- Major Highways
- Counties

Subwatersheds

- Watersheds

Scale 1 : 500 000

0 6 12 18 Kilometers

Subwatersheds

Figure 3-5

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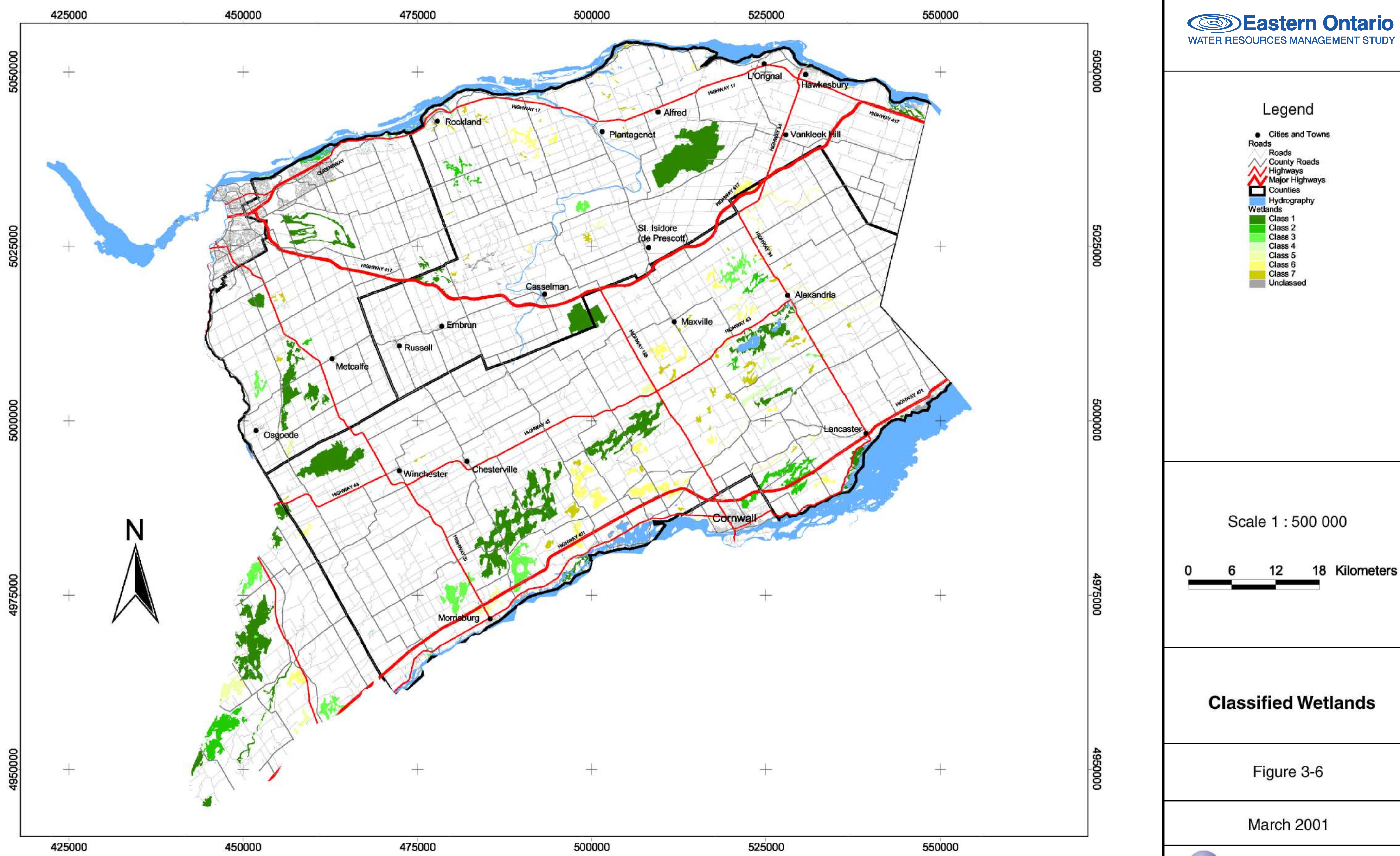

Eastern Ontario
WATER RESOURCES MANAGEMENT STUDY

Classified Wetlands

Figure 3-6

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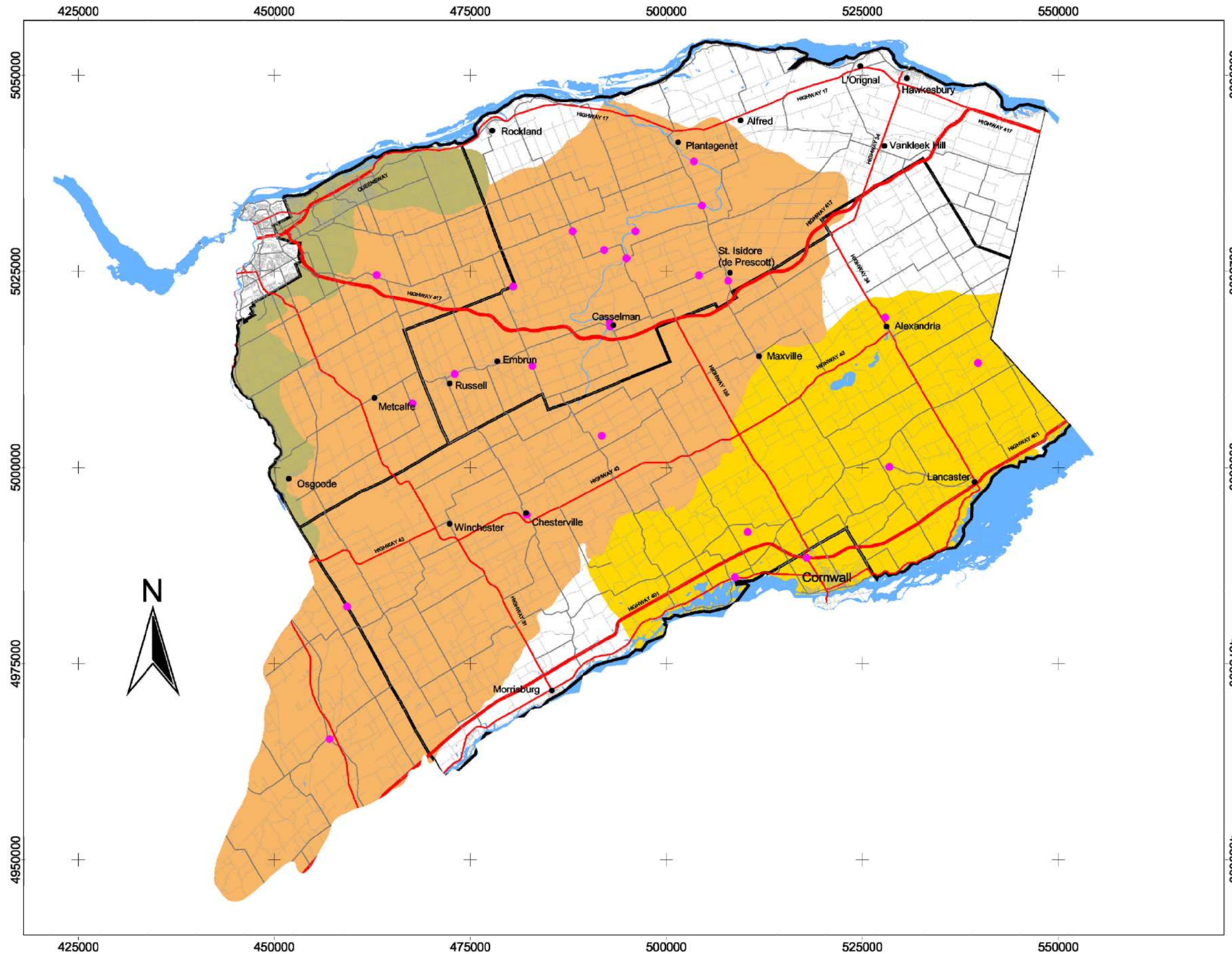

Eastern Ontario
WATER RESOURCES MANAGEMENT STUDY

Legend
- Cities and Towns
- Streamflow Stations

Roads
- Roads
- County Roads
- Highways
- Major Highways
- Counties
- Hydrography

Conservation Authority Watersheds
- Raisin Region
- Rideau Valley
- South Nation

Scale 1 : 500 000

0 6 12 18 Kilometers

Streamflow Stations

Figure 4-1

March 2001

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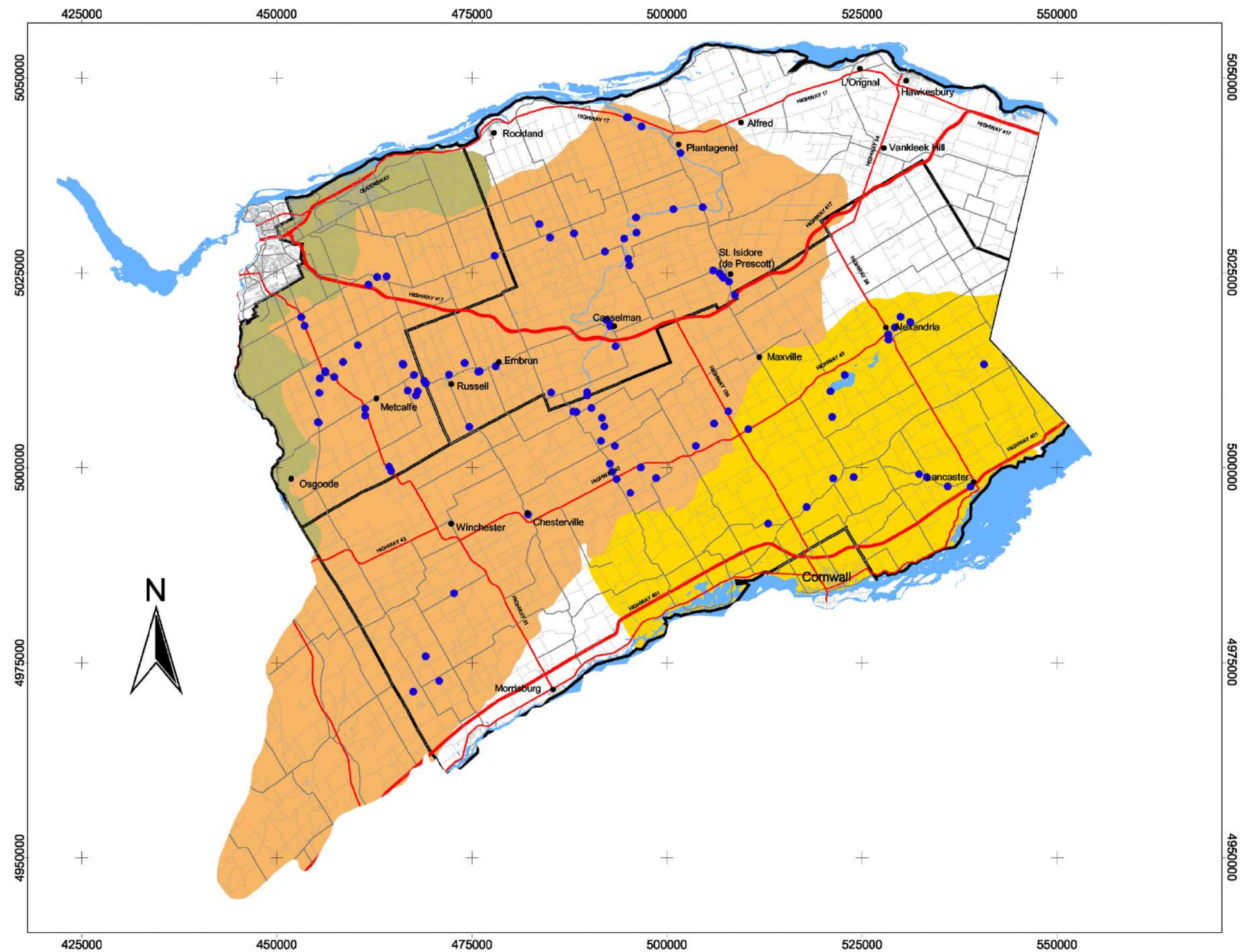
Eastern Ontario
WATER RESOURCES MANAGEMENT STUDY



Legend

- Cities and Towns
- Water Quality Stations

Roads
- Roads
- County Roads
- Highways
- Major Highways
- Counties
- Hydrography

Conservation Authority Watersheds
- Raisin Region
- Rideau Valley
- South Nation

Scale 1 : 500 000

0 6 12 18 Kilometers

Surface Water Quality Stations

Figure 4-2

March 2001

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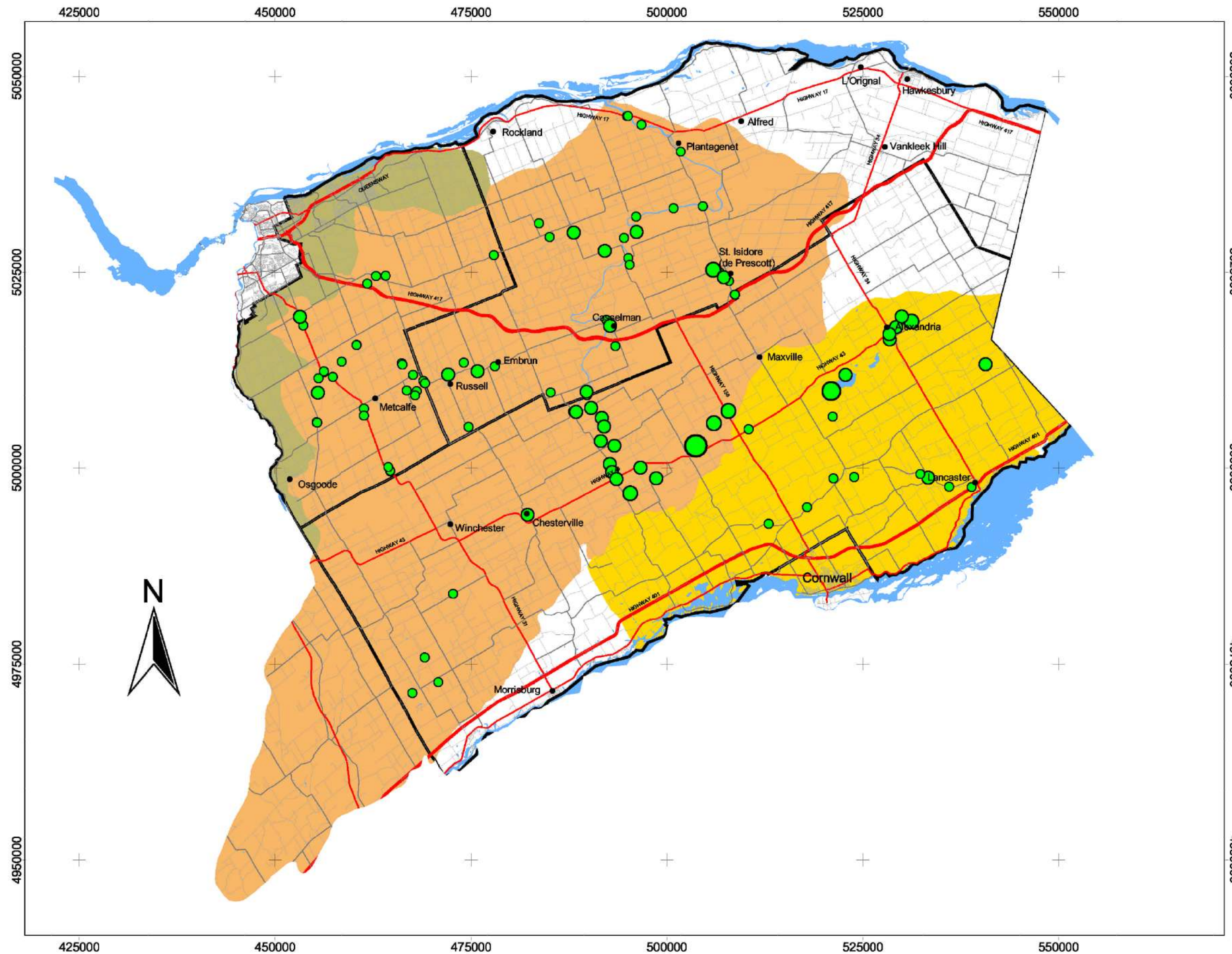


Eastern Ontario
WATER RESOURCES MANAGEMENT STUDY

Legend

- Cities and Towns
- Total Ammonia
  - 0 - 0.02 mg/L
  - 0.02 - 0.1 mg/L
  - 0.1 - 0.3 mg/L
  - 0.3 - 0.6 mg/L
  - 0.6 - 0.9 mg/L
- Roads
  - Roads
  - County Roads
  - Highways
  - Major Highways
- Counties
- Hydrography
- Conservation Authority Watersheds
  - Raisin Region
  - Rideau Valley
  - South Nation

Scale 1 : 500 000

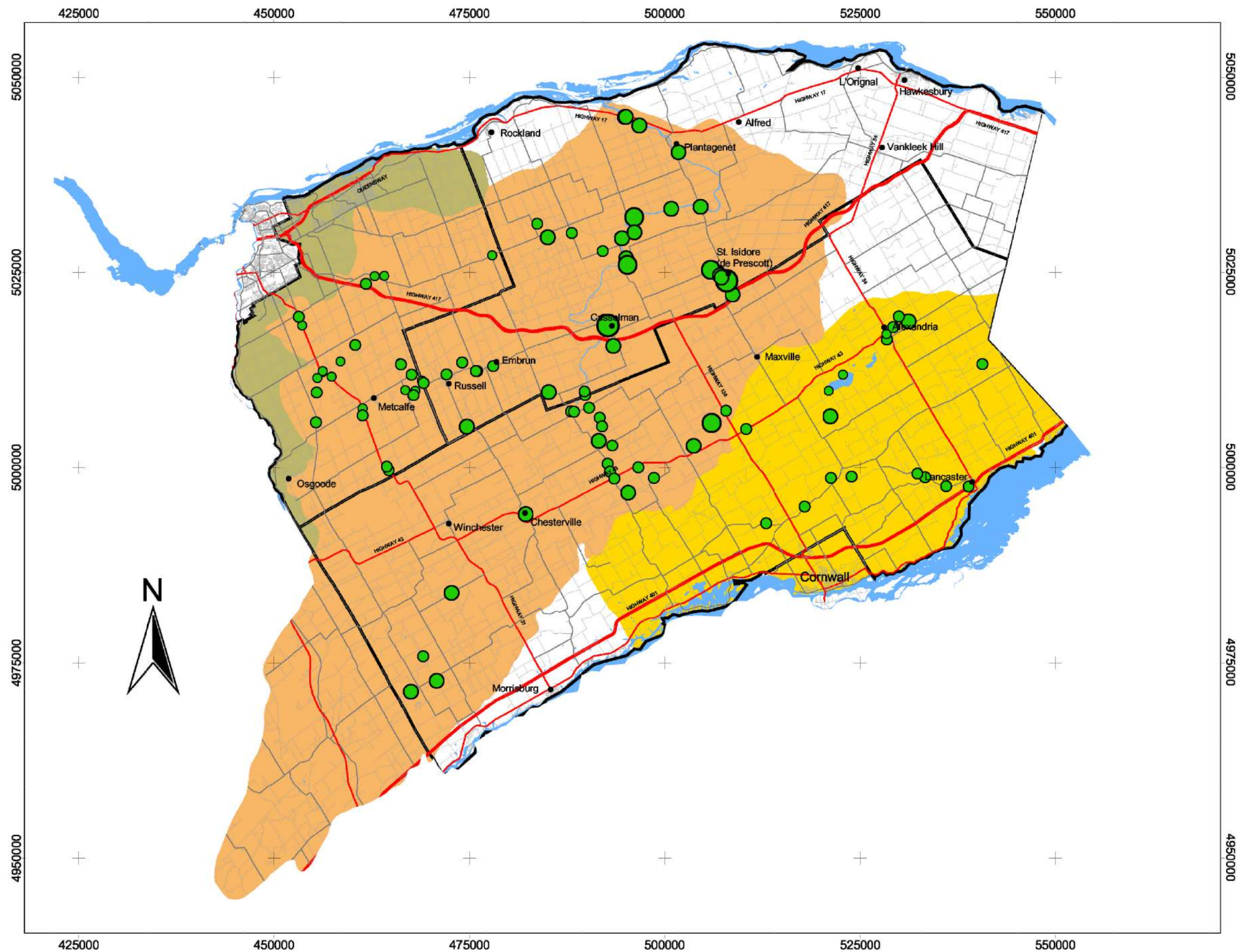
0 6 12 18 Kilometers

Surface Water Quality
Total Ammonia

Figure 4-3

March 2001

CH2MHILL



Eastern Ontario
WATER RESOURCES MANAGEMENT STUDY

Legend

- Cities and Towns
- Total Phosphorus
  - 0 - 0.03 mg/L
  - 0.03 - 0.1 mg/L
  - 0.1 - 0.3 mg/L
  - 0.3 - 0.5 mg/L
  - 0.5 - 0.7 mg/L
- Roads
  - Roads
  - County Roads
  - Highways
  - Major Highways
- Counties
- Hydrography
- Conservation Authority Watersheds
  - Raisin Region
  - Rideau Valley
  - South Nation

Scale 1 : 500 000

0 6 12 18 Kilometers

Surface Water Quality
Total Phosphorus

Figure 4-4

March 2001

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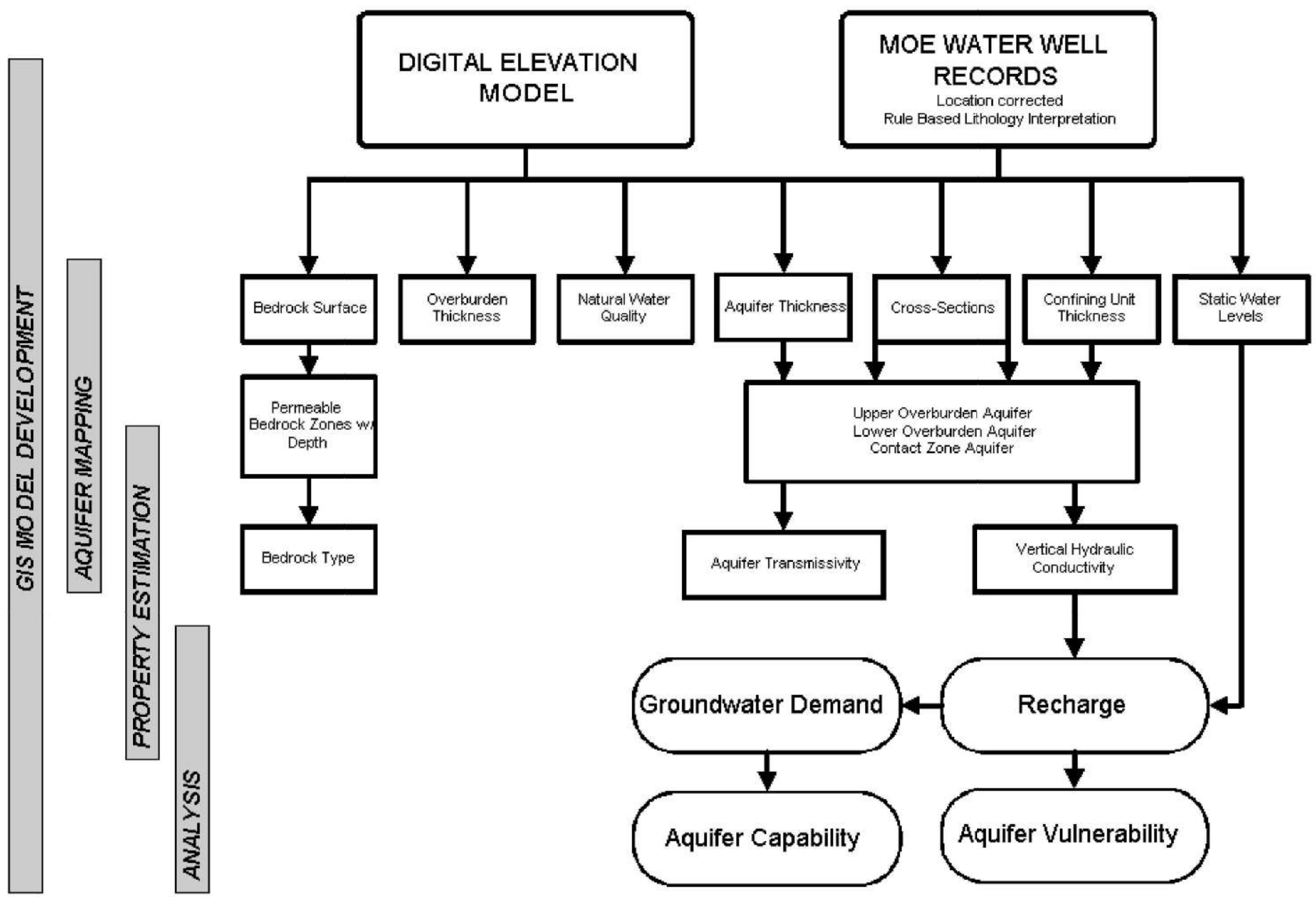
GENERAL OVERVIEW GROUNDWATER COMPONENT



Eastern Ontario
WATER RESOURCES MANAGEMENT STUDY

Overview of Groundwater Analysis

Figure 5-1

March 2001

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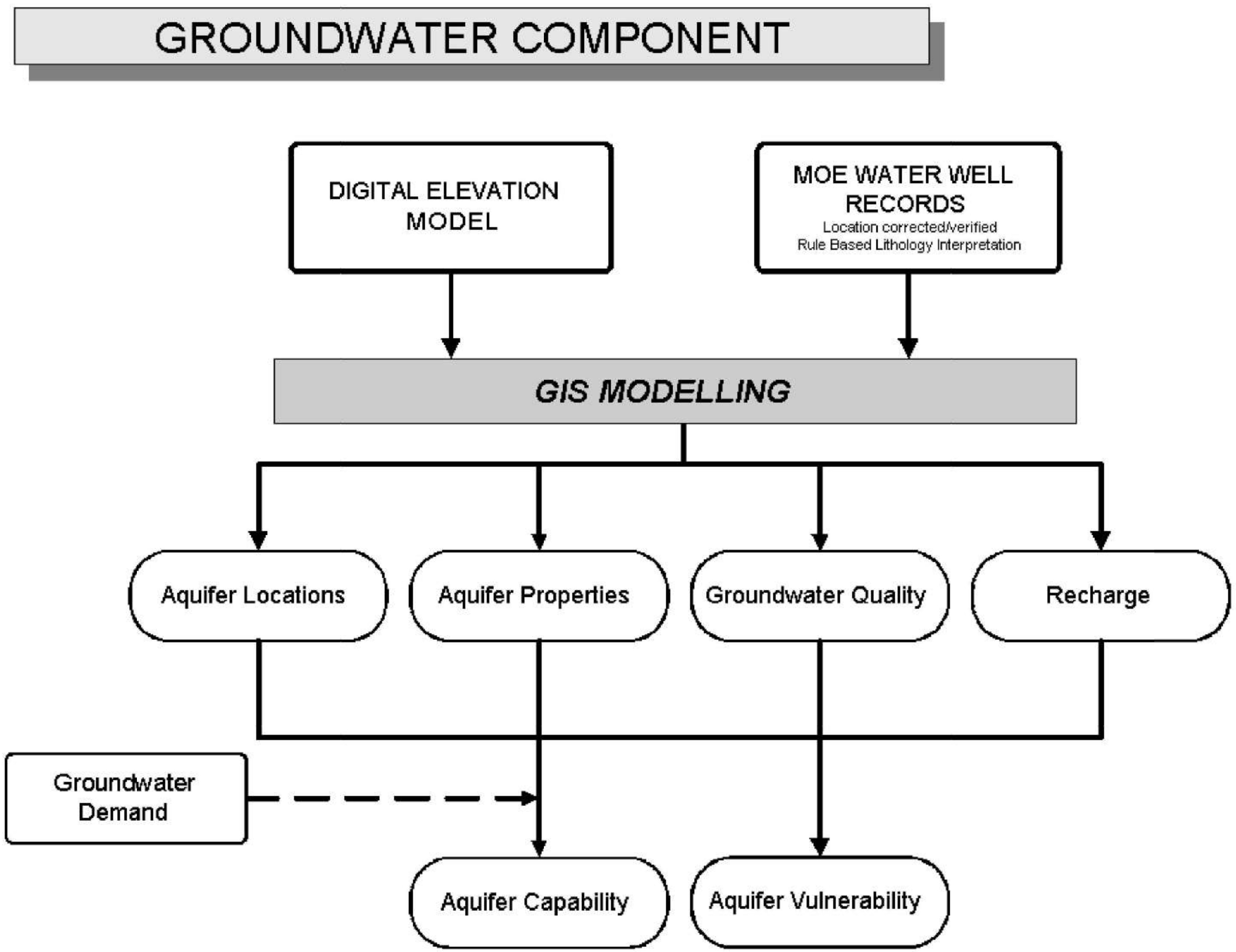

Eastern Ontario
WATER RESOURCES MANAGEMENT STUDY

Groundwater Analysis

Figure 5-2

March 2001

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Eastern Ontario
WATER RESOURCES MANAGEMENT STUDY

Legend

- Wetlands
- Counties
- Roads
- Highway
- Major Highway
- Faults
- Hydrography
- Regional Cross Sections
- Water Wells

Scale 1 : 500 000

0 6 12 18 Kilometers

Water Well Locations

Figure 5-3

March 2001

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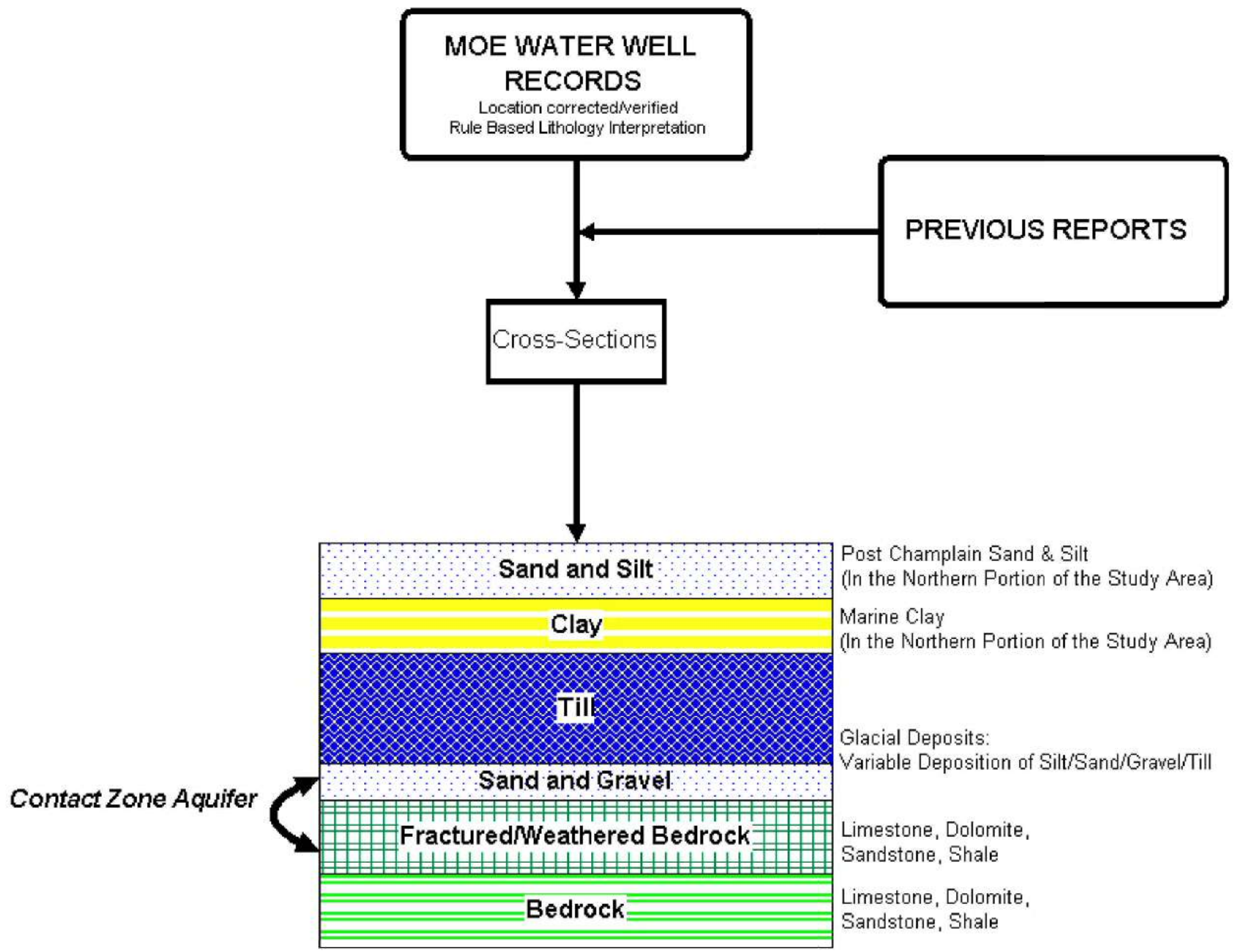
CONCEPTUAL MODEL



Eastern Ontario
WATER RESOURCES MANAGEMENT STUDY

Conceptual Model

Figure 5-4

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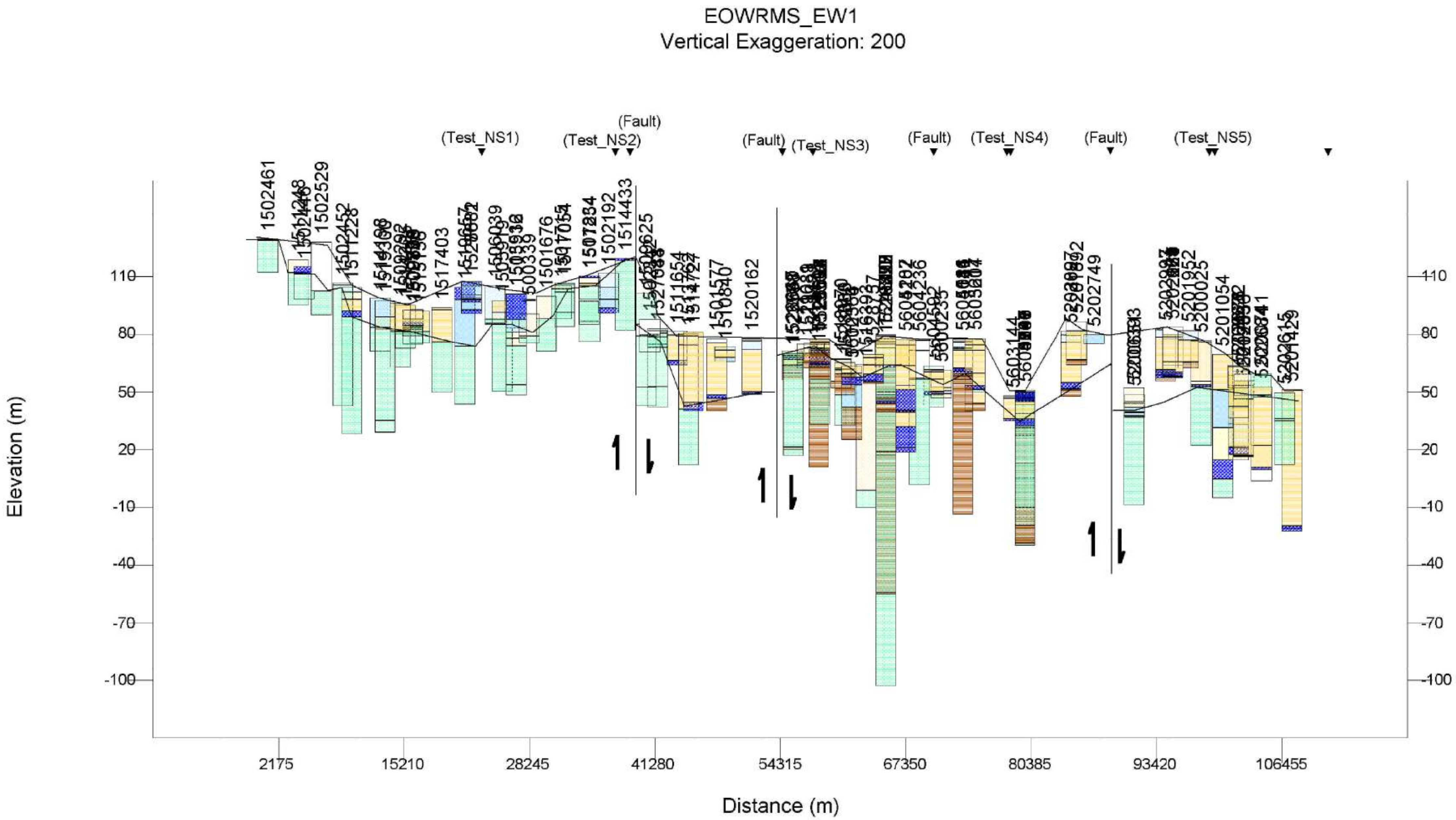
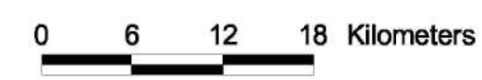
Eastern Ontario
WATER RESOURCES MANAGEMENT STUDY



Legend

Scale 1 : 500 000



Regional West-East Cross-Section

Figure 5-5

March 2001

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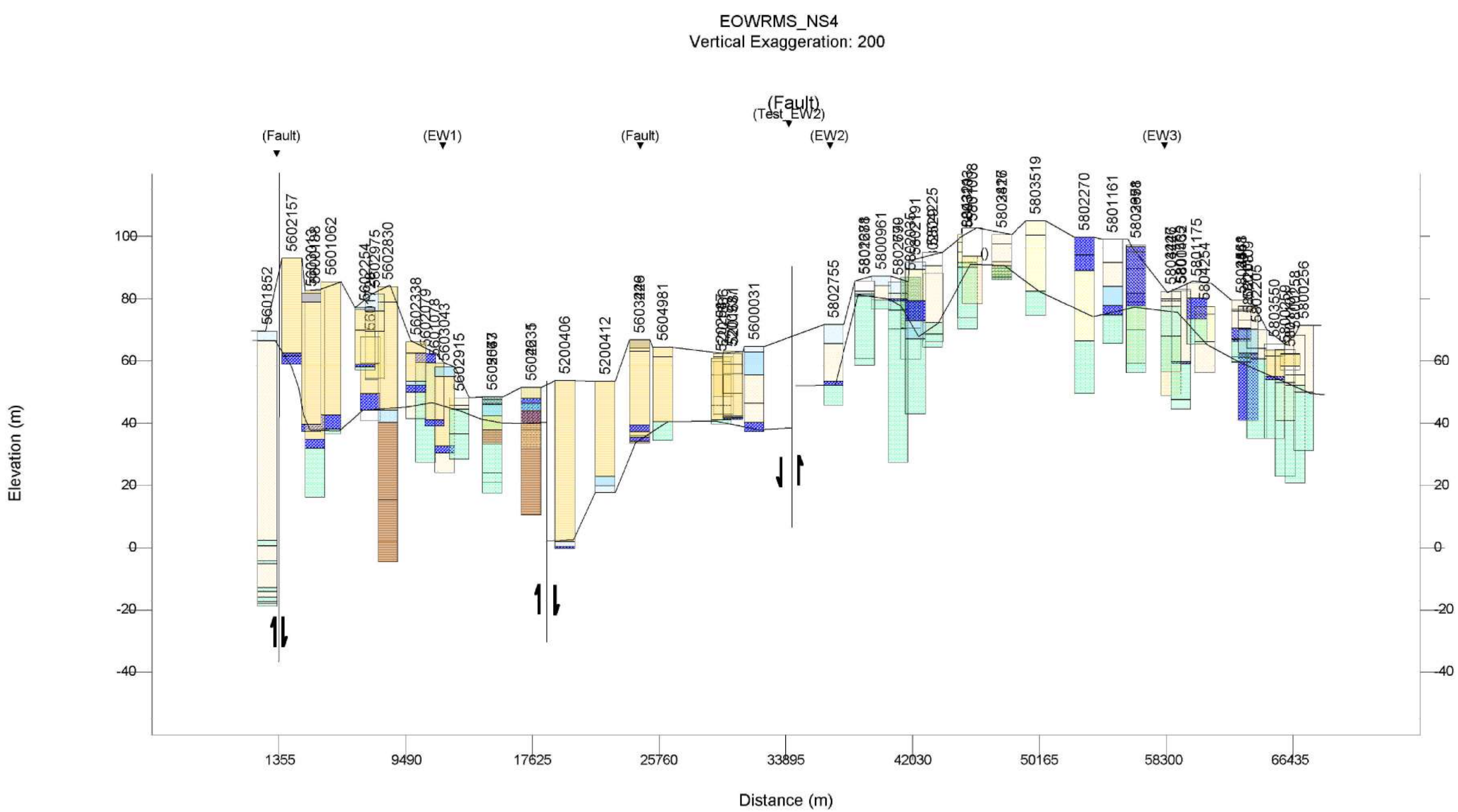
Digital Mapping Sources: Various Ministries of the Province of Ontario and Departments of the Federal Government. Reference should be made to metadata compiled for this study.

Eastern Ontario
WATER RESOURCES MANAGEMENT STUDY



Scale 1 : 500 000

0 6 12 18 Kilometers

Regional North-South Cross-Section

Figure 5-6

March 2001

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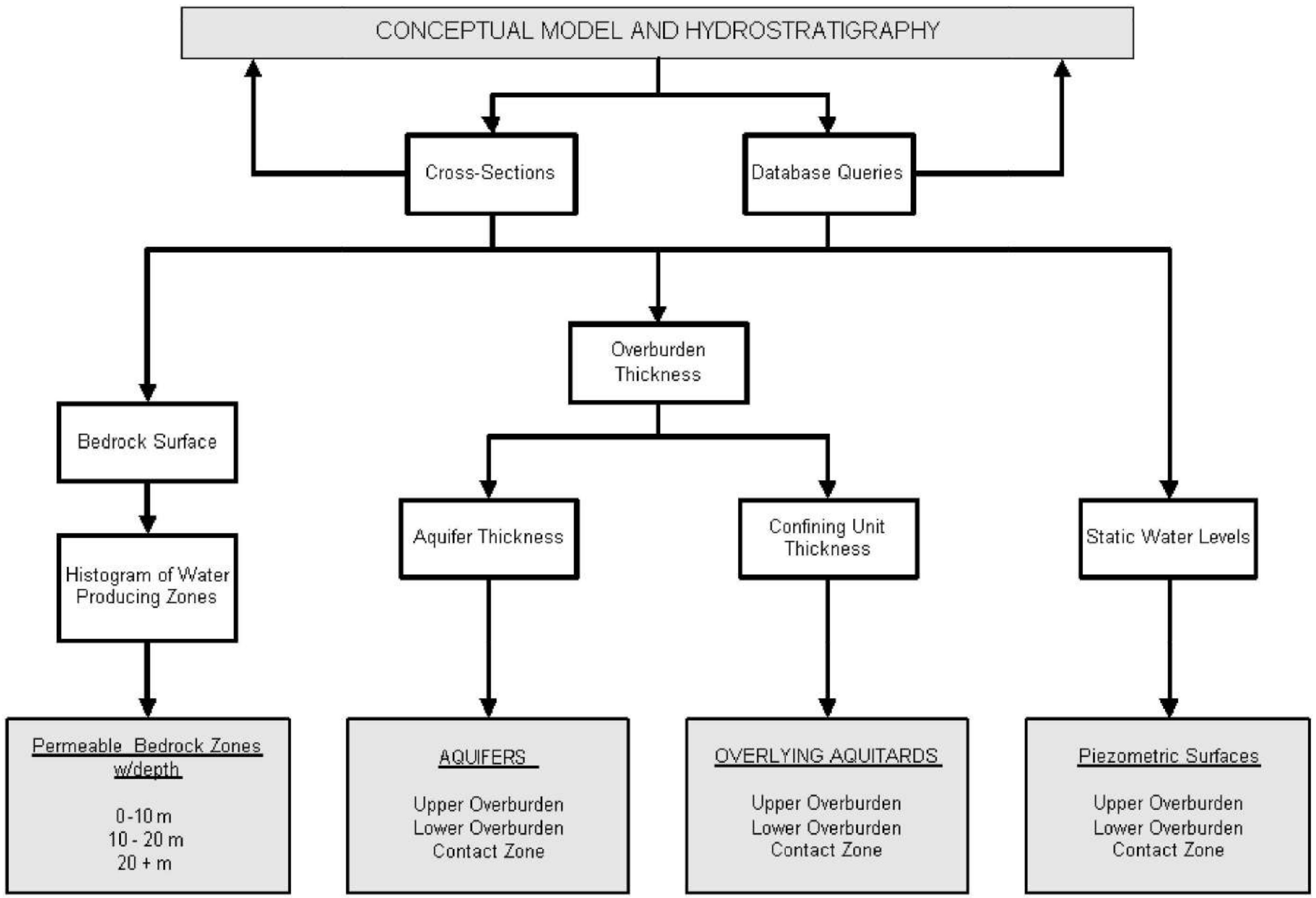
Eastern Ontario
WATER RESOURCES MANAGEMENT STUDY

AQUIFER LOCATIONS



Aquifer Locations

Figure 5-7

March 2001

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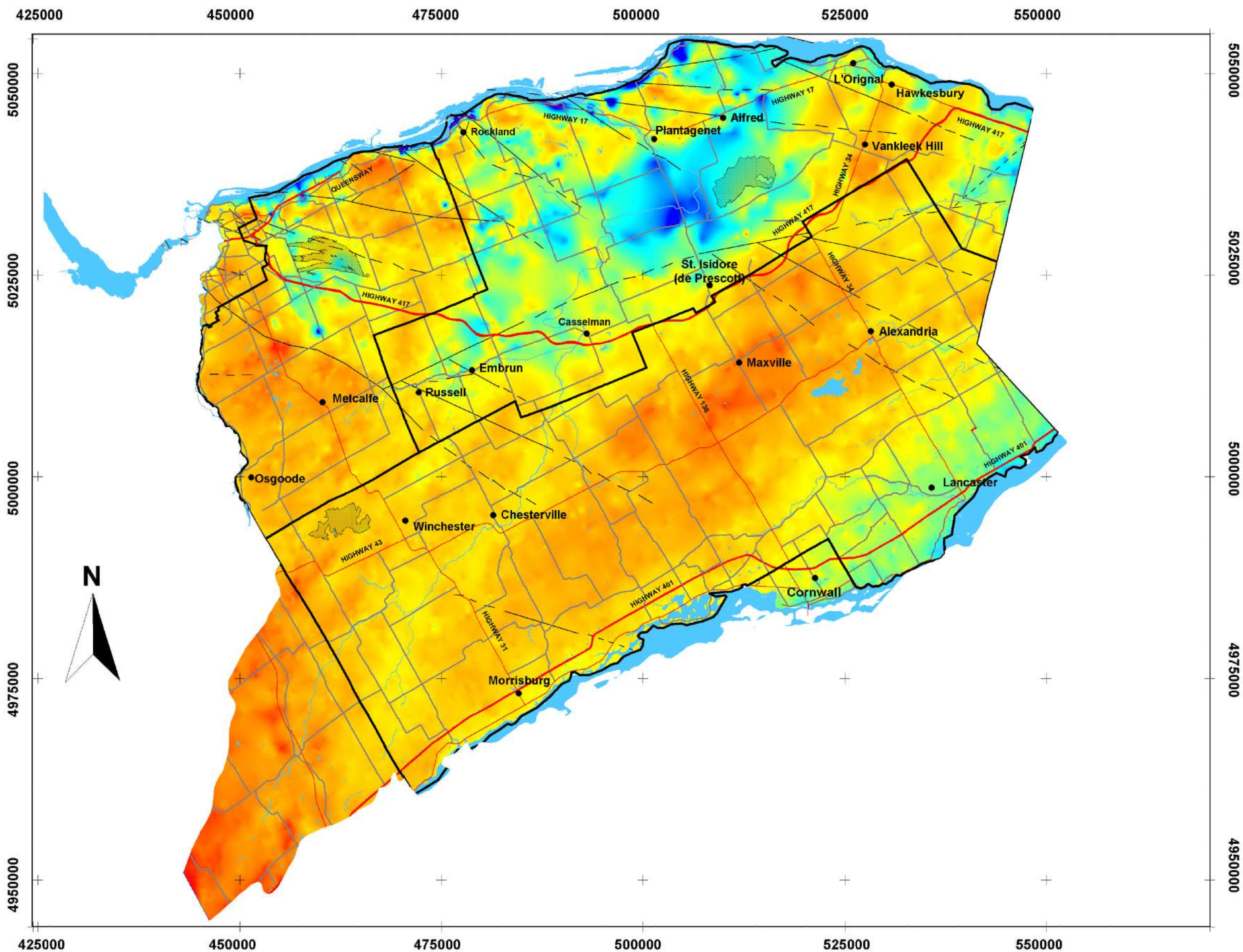
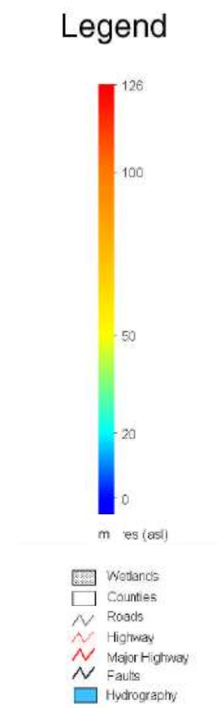
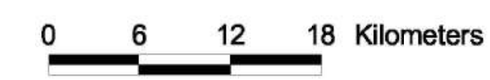


Eastern Ontario
WATER RESOURCES MANAGEMENT STUDY



Scale 1 : 500 000



Bedrock Surface Elevation

Figure 5-8

March 2001

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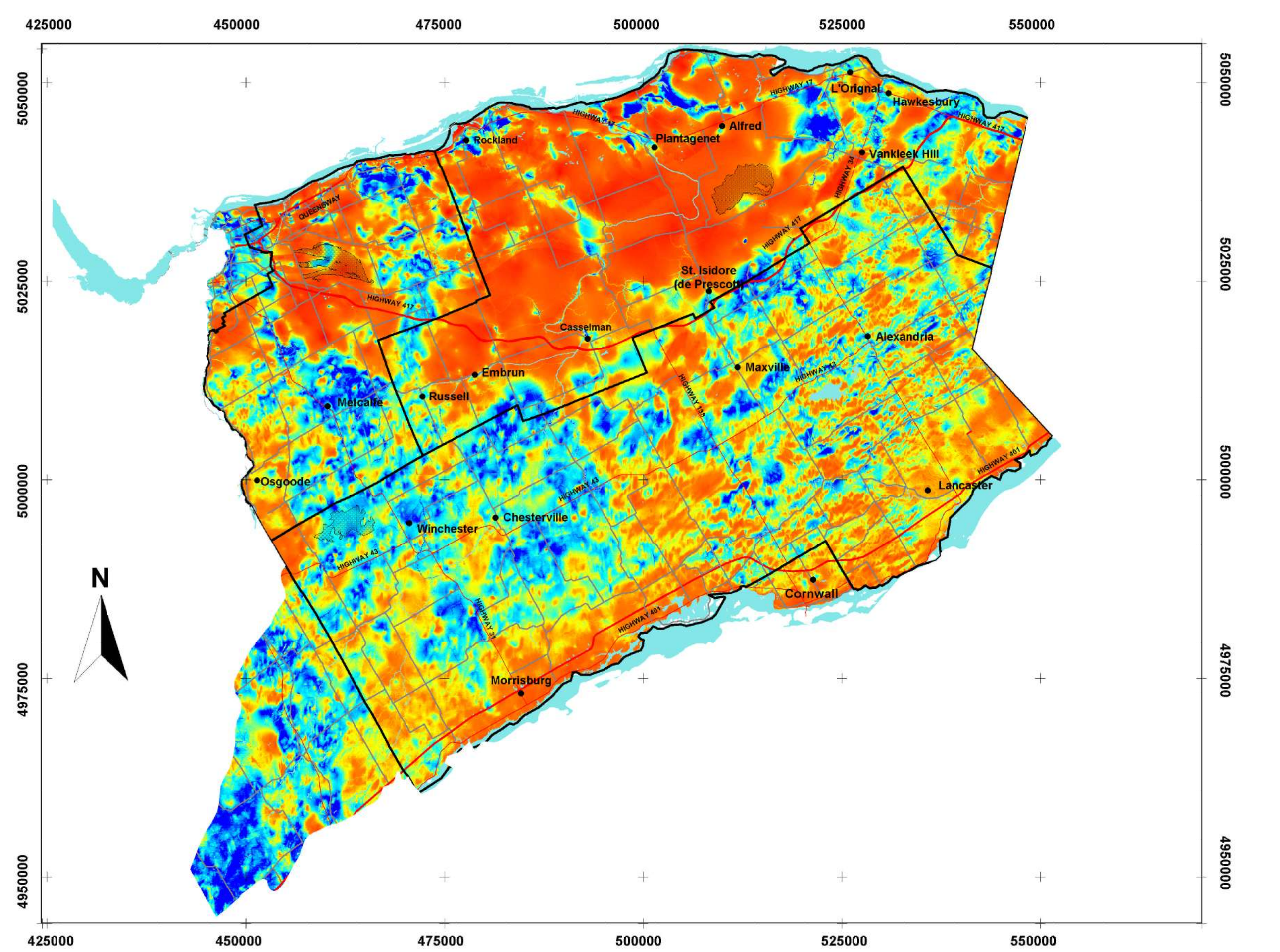

Eastern Ontario
WATER RESOURCES MANAGEMENT STUDY

Legend

1.98
1.26
1.07
0.82
0.00
log (metres)

Wetlands
Counties
Roads
Highway
Major Highway
Faults
Hydrography

Scale 1 : 500 000

0 6 12 18 Kilometers

Overburden Thickness

Figure 5-9

March 2001

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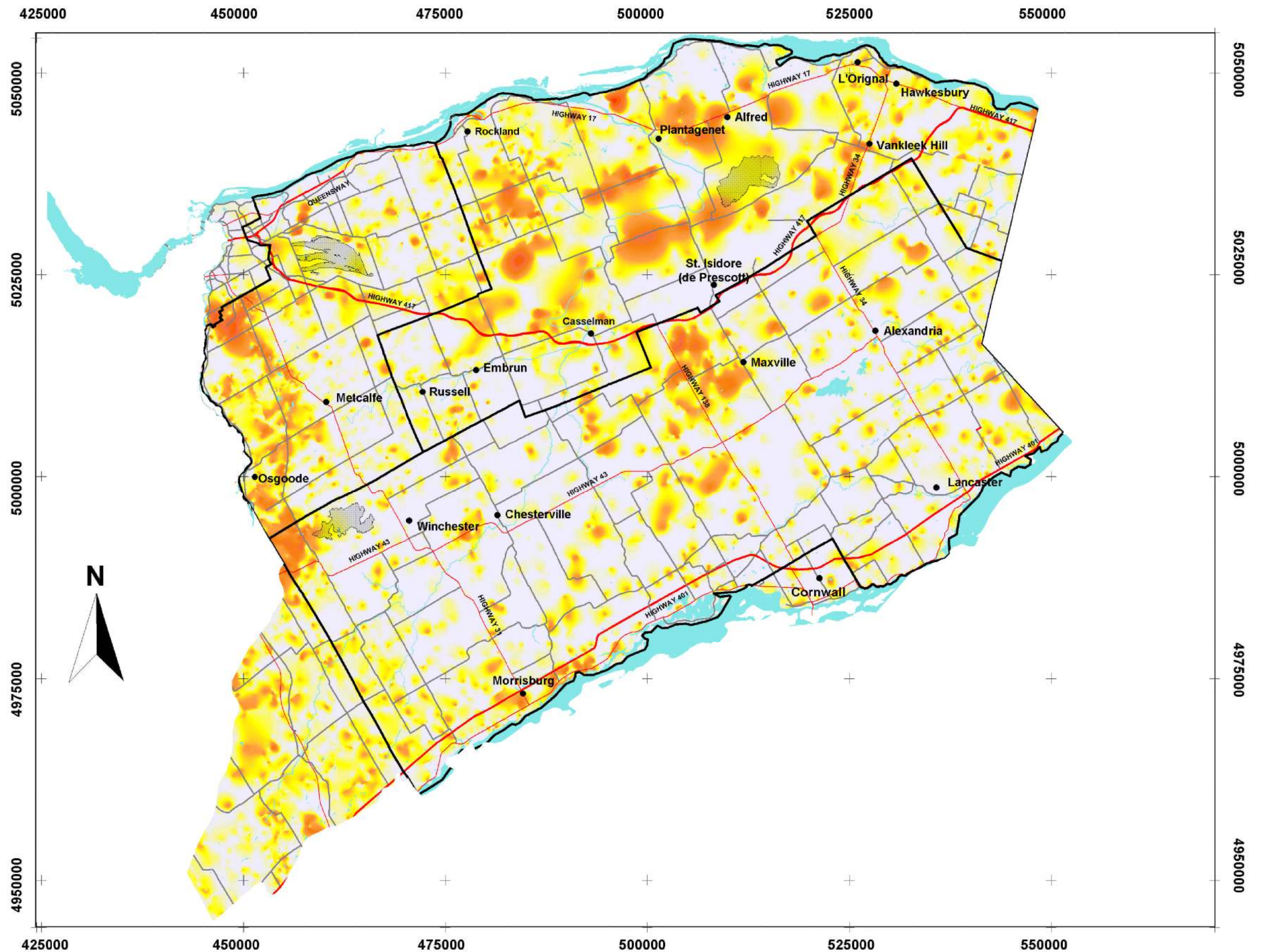

Digital Mapping Sources: Various Ministries of the Province of Ontario and Departments of the Federal Government. Reference should be made to metadata compiled for this study.
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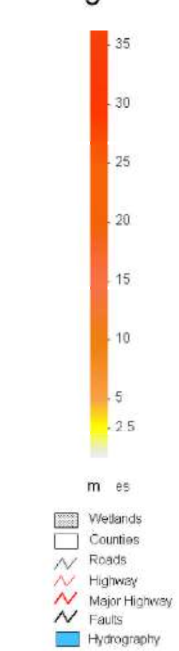
Eastern Ontario
WATER RESOURCES MANAGEMENT STUDY



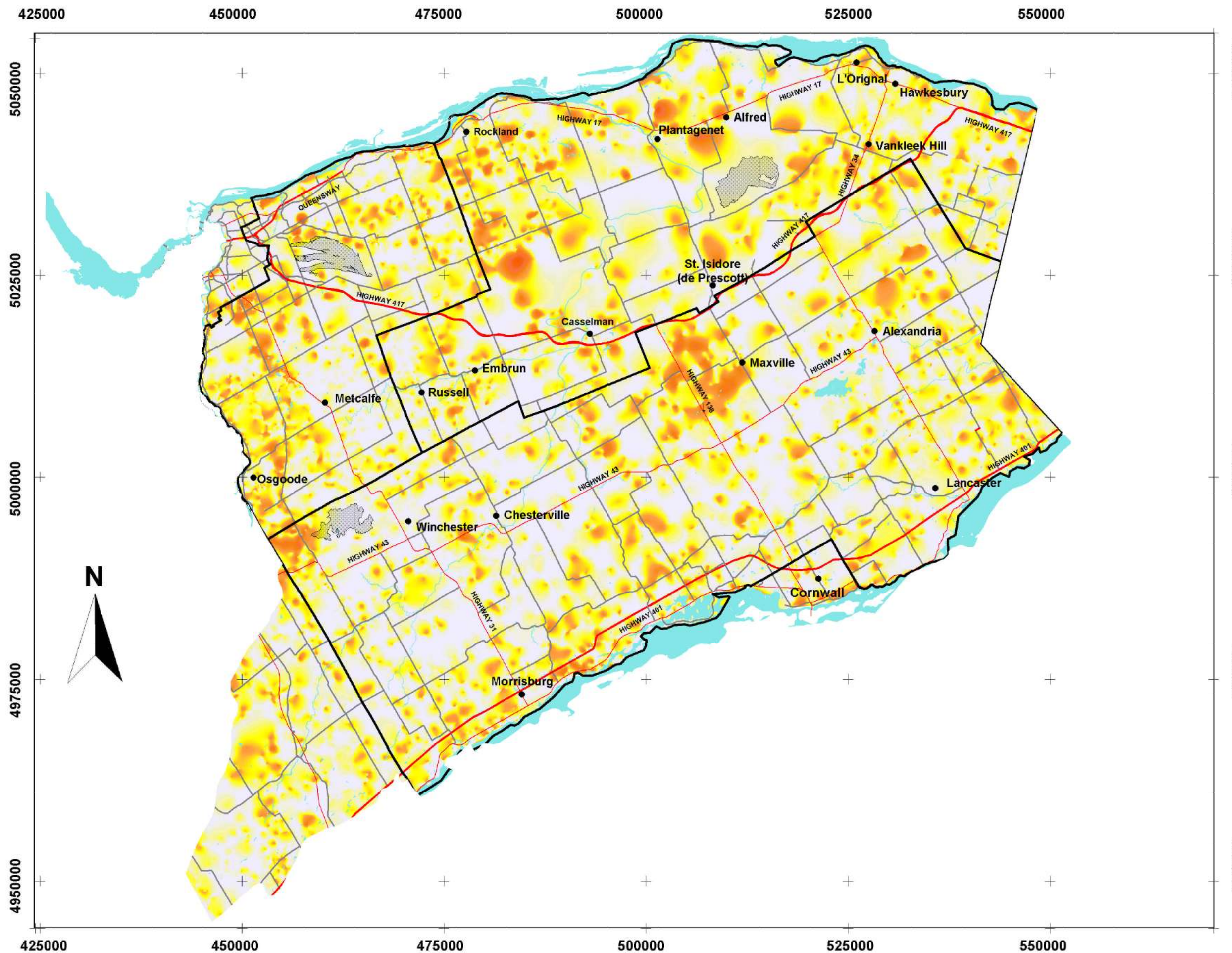
Scale 1 : 500 000

Upper Overburden Aquifer Thickness

Figure 5-10

March 2001

CH2MHILL



Eastern Ontario
WATER RESOURCES MANAGEMENT STUDY

Legend

35
30
25
20
15
10
5
2.5

Wetlands
Counties
Roads
Highway
Major Highway
Faults
Hydrography

Scale 1 : 500 000

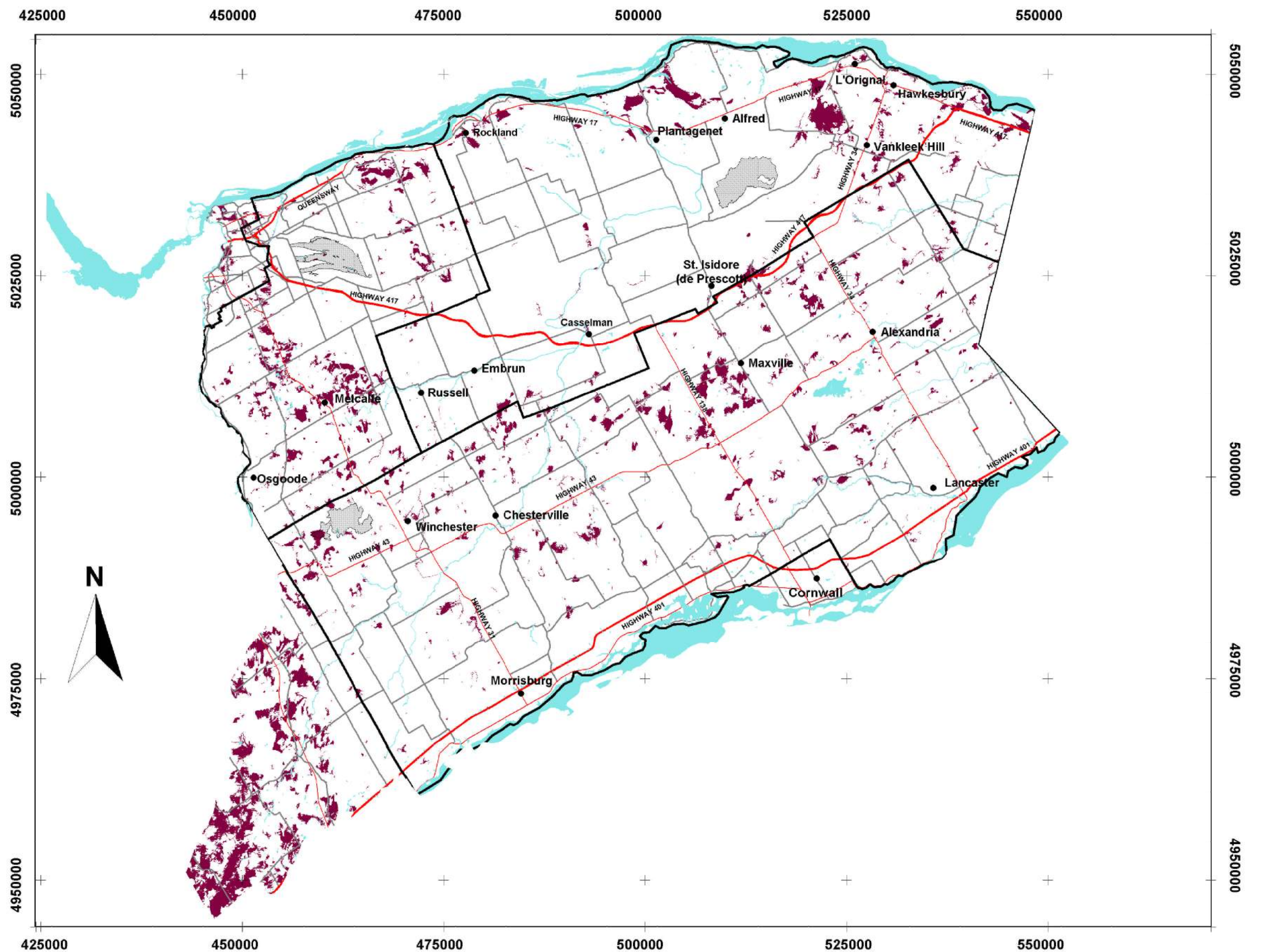
0 6 12 18 Kilometers

Lower Overburden Aquifer Thickness

Figure 5-11

March 2001

CH2MHILL



Eastern Ontario
WATER RESOURCES MANAGEMENT STUDY

Legend

Good connection exists between upper and lower overburden aquifers

Limited to no connection exists between upper and lower overburden aquifers

Wetlands
Counties
Roads
Highway
Major Highway
Faults
Hydrography

Scale 1 : 500 000

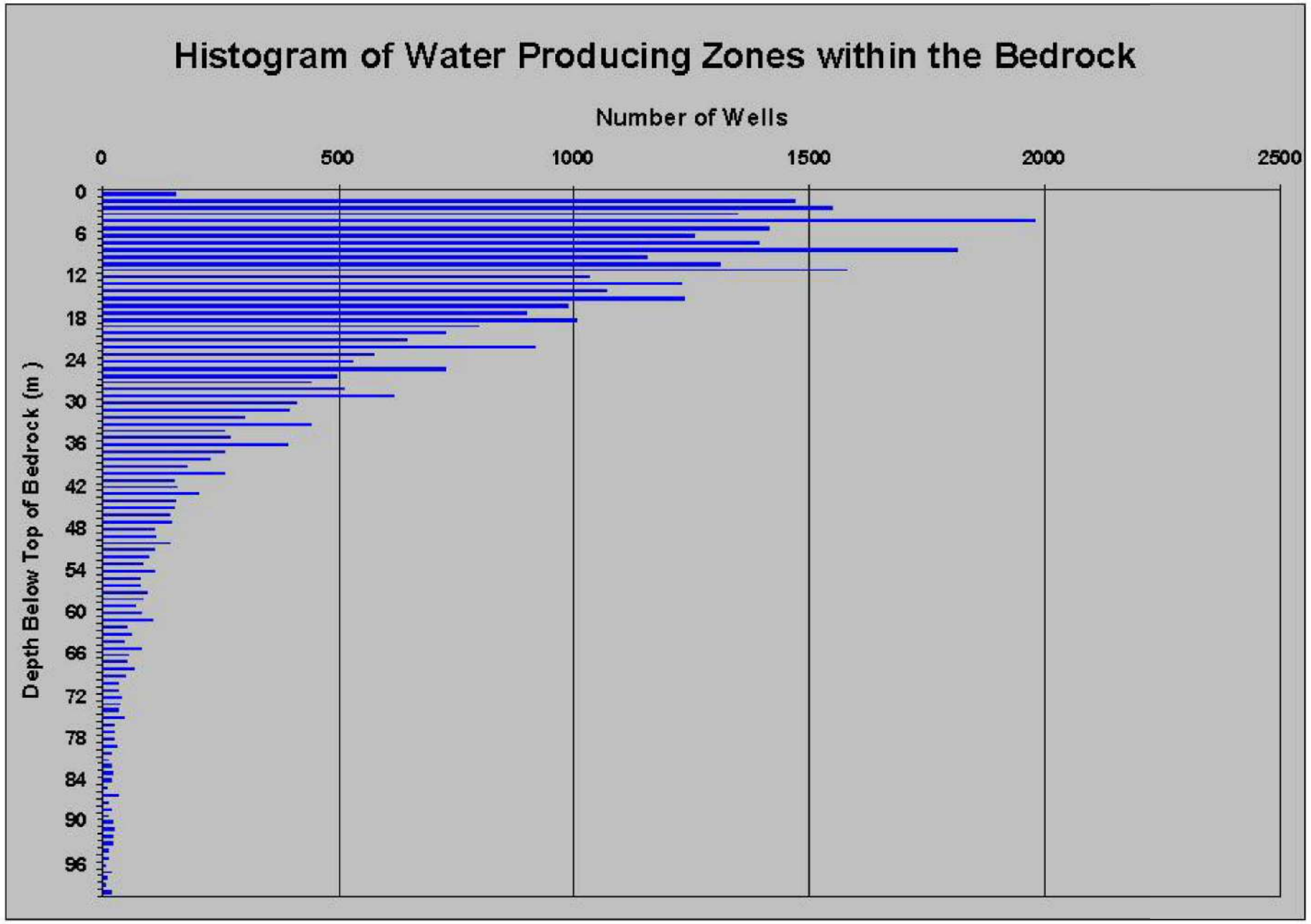
0 6 12 18 Kilometers

Connectivity of Upper and Lower Overburden Aquifers

Figure 5-12

March 2001

CH2MHILL



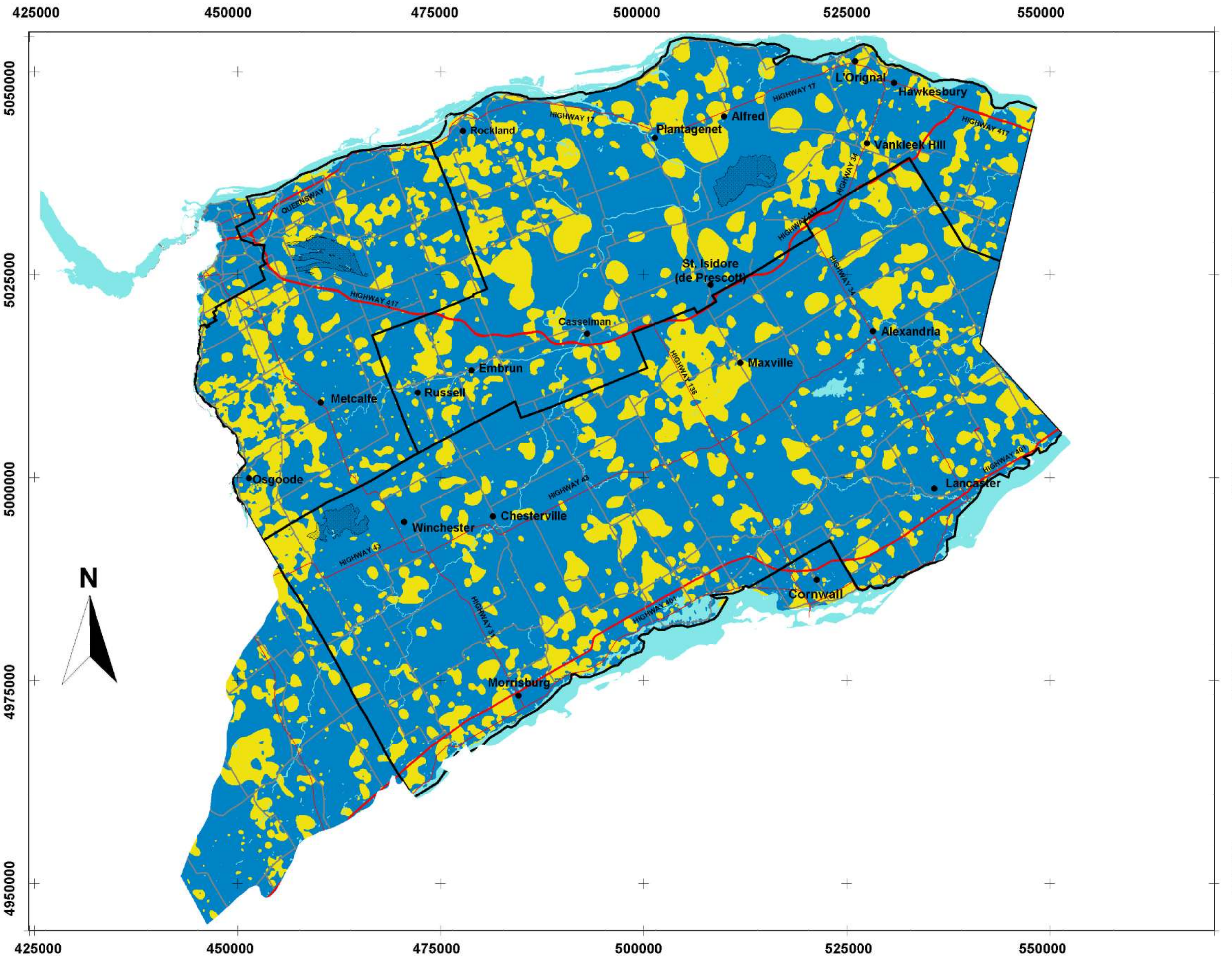
Eastern Ontario
WATER RESOURCES MANAGEMENT STUDY

Histogram of Water Producing Zones within the Bedrock

Figure 5-13

March 2001

CH2MHILL



Eastern Ontario
WATER RESOURCES MANAGEMENT STUDY

Legend

Lower Overburden & Bedrock Aquifer

Bedrock Aquifer Only

Wetlands
Counties
Roads
Highway
Major Highway
Faults
Hydrography

Scale 1 : 500 000

0 6 12 18 Kilometers

Contact Zone Aquifer

Figure 5-14

March 2001

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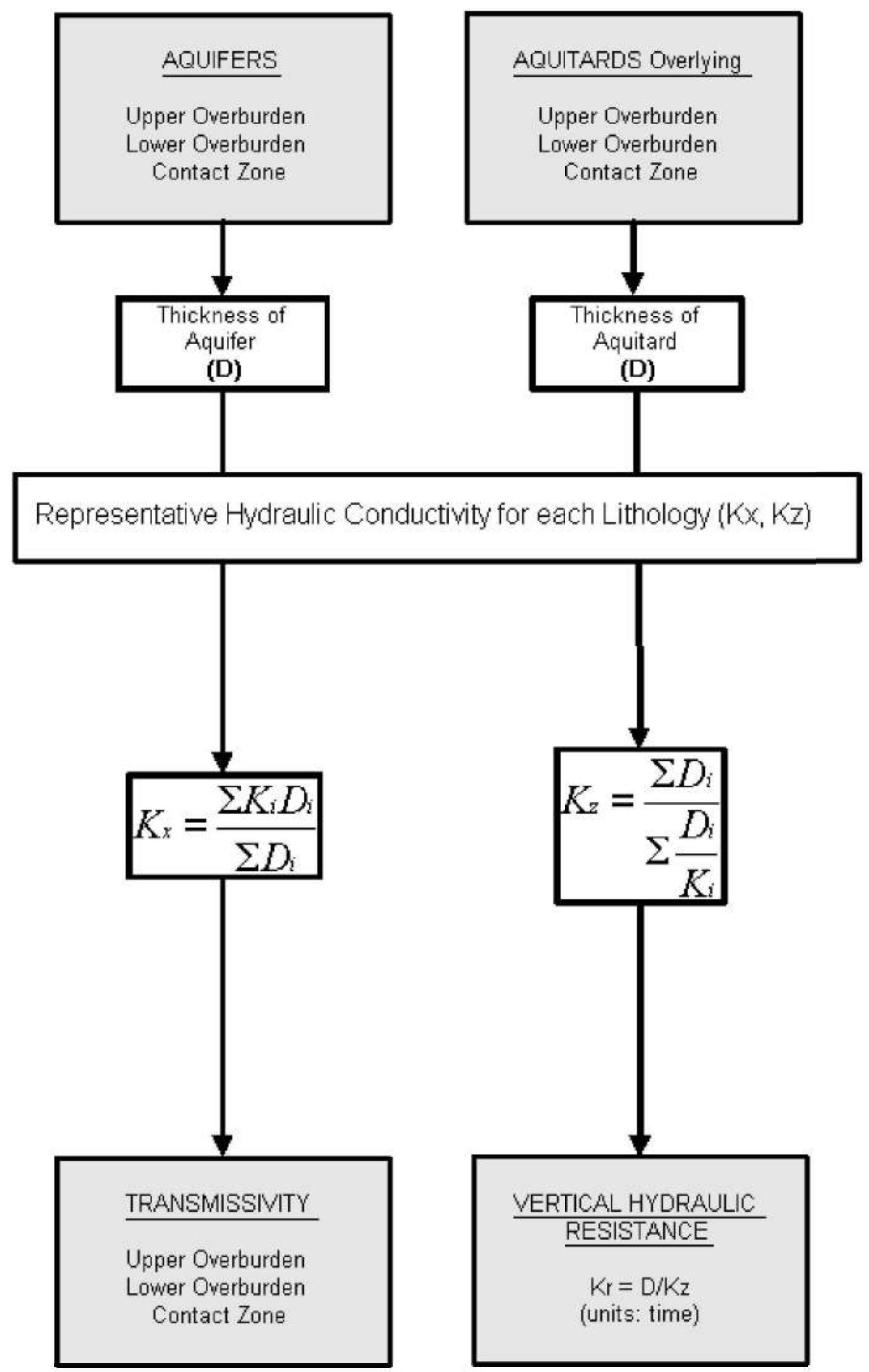
AQUIFER PROPERTIES



Aquifer Properties

Figure 5-15

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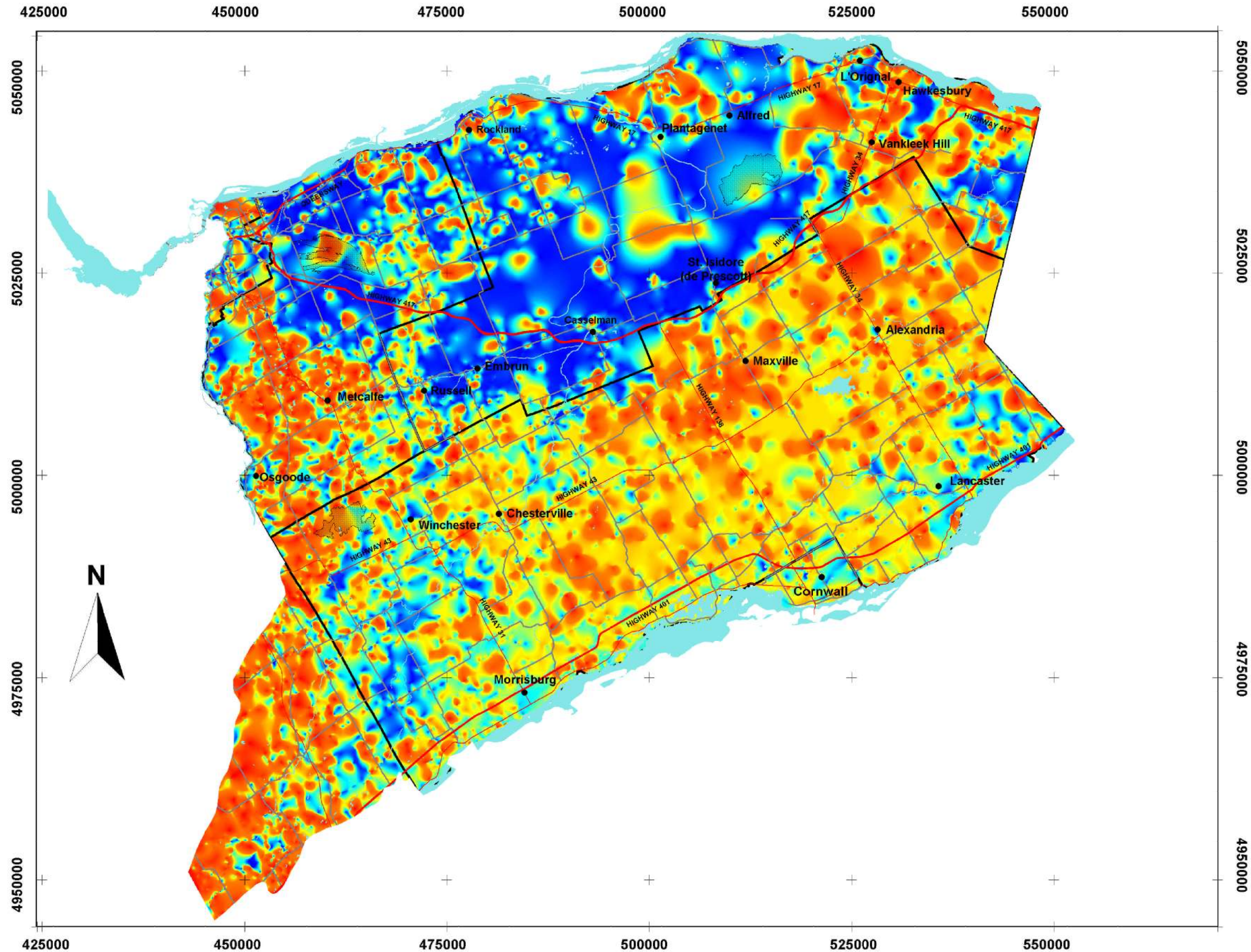
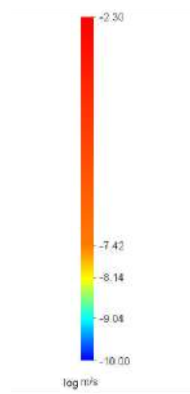
Eastern Ontario
WATER RESOURCES MANAGEMENT STUDY



Legend



Scale 1 : 500 000

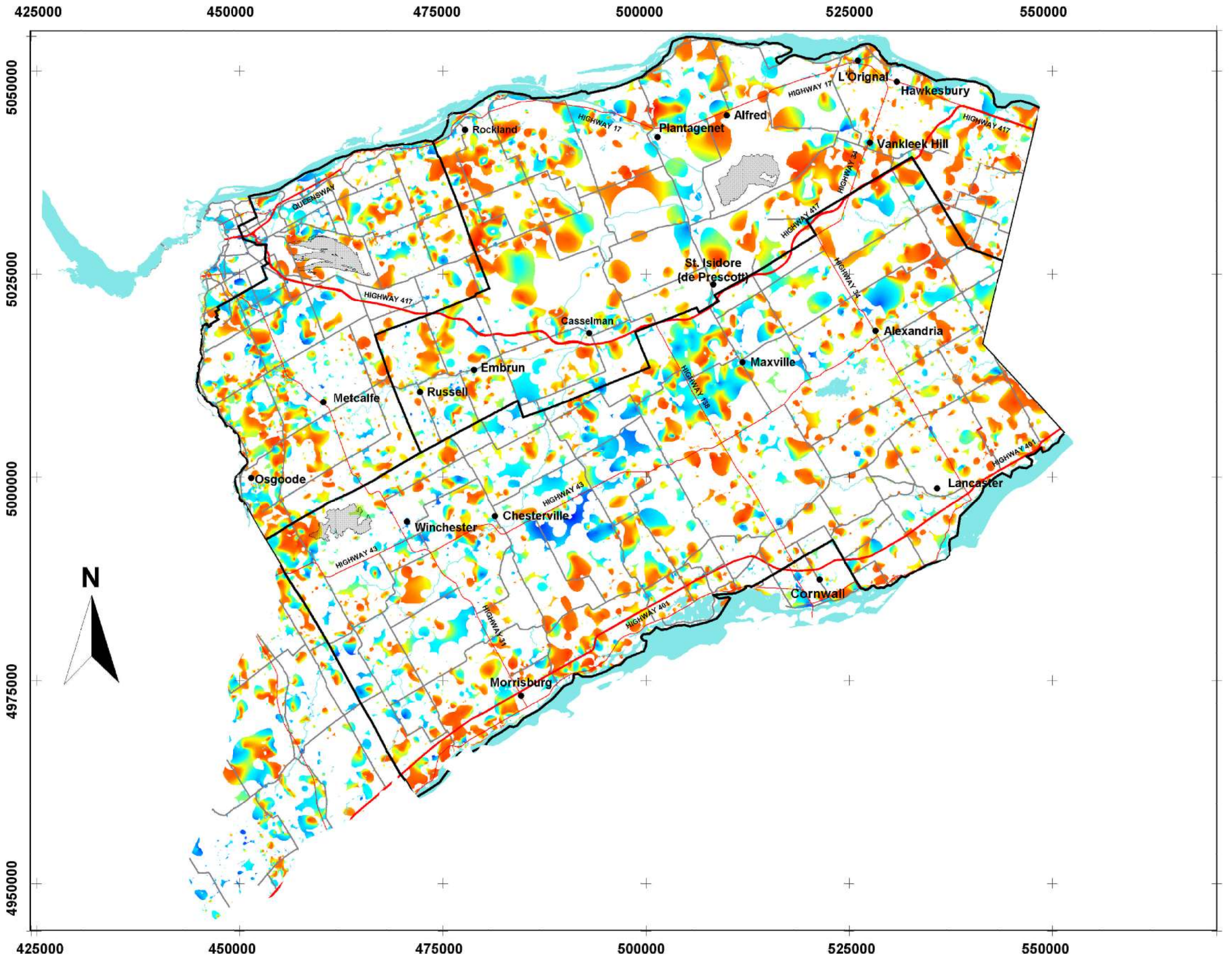
0 6 12 18 Kilometers

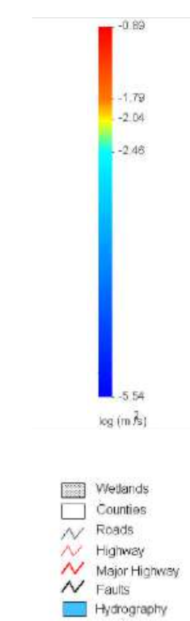
Vertical Hydraulic Conductivity of the Overburden

Figure 5-16

March 2001

CH2MHILL





Eastern Ontario
WATER RESOURCES MANAGEMENT STUDY

Scale 1 : 500 000

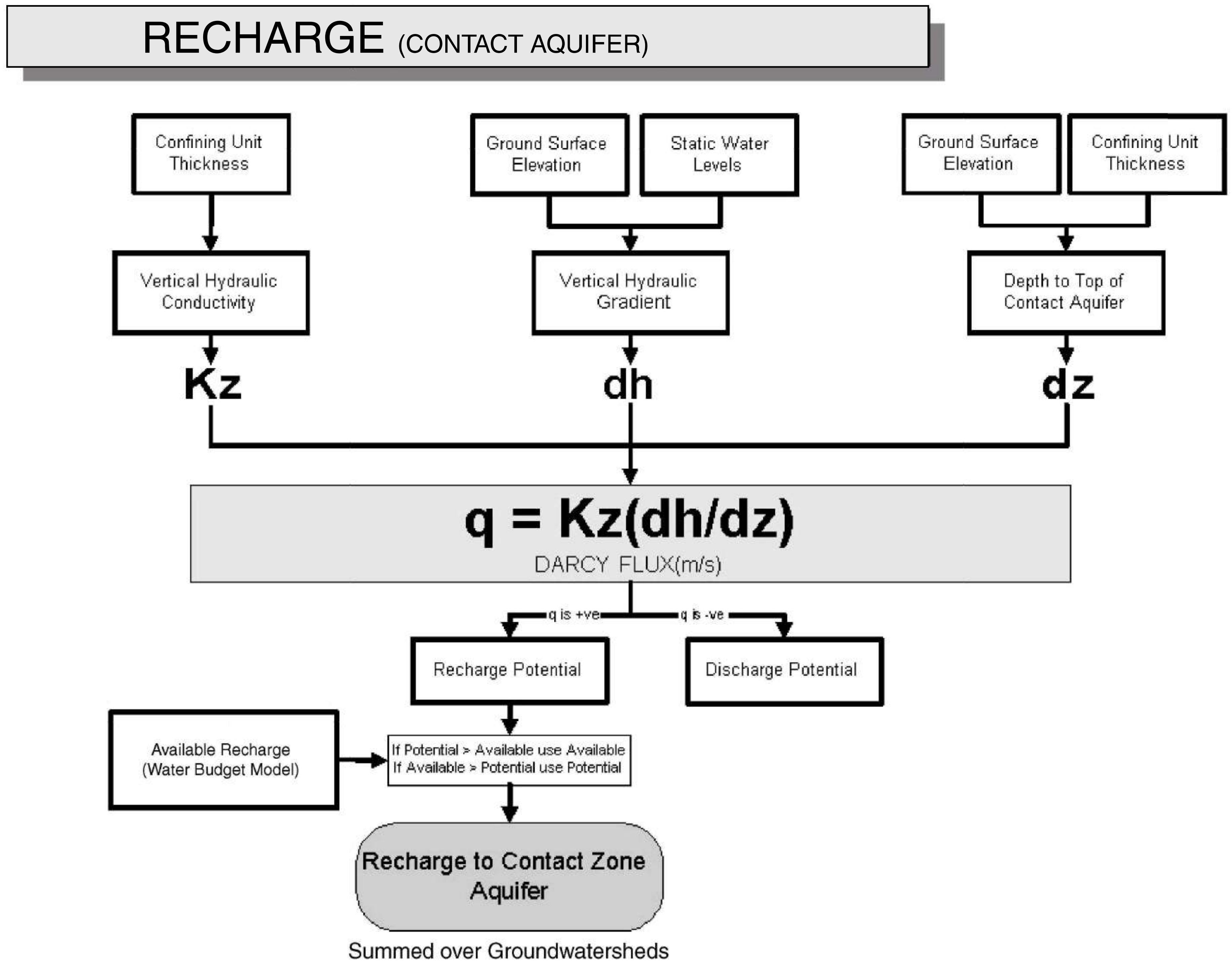
0 6 12 18 Kilometers

Estimated Transmissivity of Lower Overburden Aquifer

Figure 5-17

March 2001

CH2MHILL



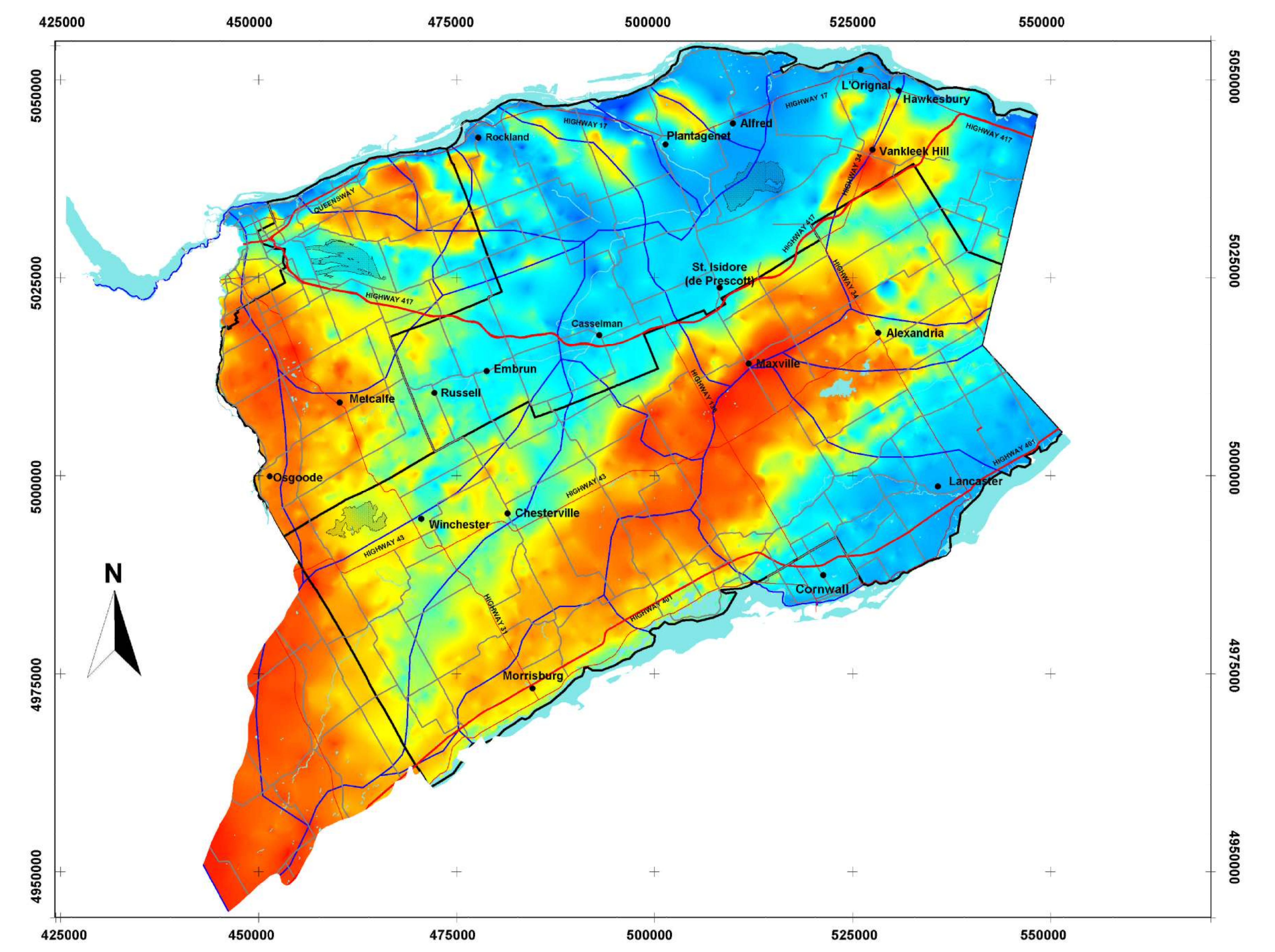
Eastern Ontario
WATER RESOURCES MANAGEMENT STUDY

Groundwater Recharge

Figure 5-18

March 2001

CH2MHILL



Eastern Ontario
WATER RESOURCES MANAGEMENT STUDY

Legend

88
75
63
11
metres (asl)

Interpreted Groundwatershed
Wetlands
Counties
Roads
Highway
Major Highway
Faults
Hydrography

Scale 1 : 500 000

0 6 12 18 Kilometers

Contact Zone Aquifer Piezometric Surface and Groundwatersheds

Figure 5-19

March 2001

CH2MHILL

Digital Mapping Sources: Various Ministries of the Province of Ontario and Departments of the Federal Government. Reference should be made to metadata compiled for this study.
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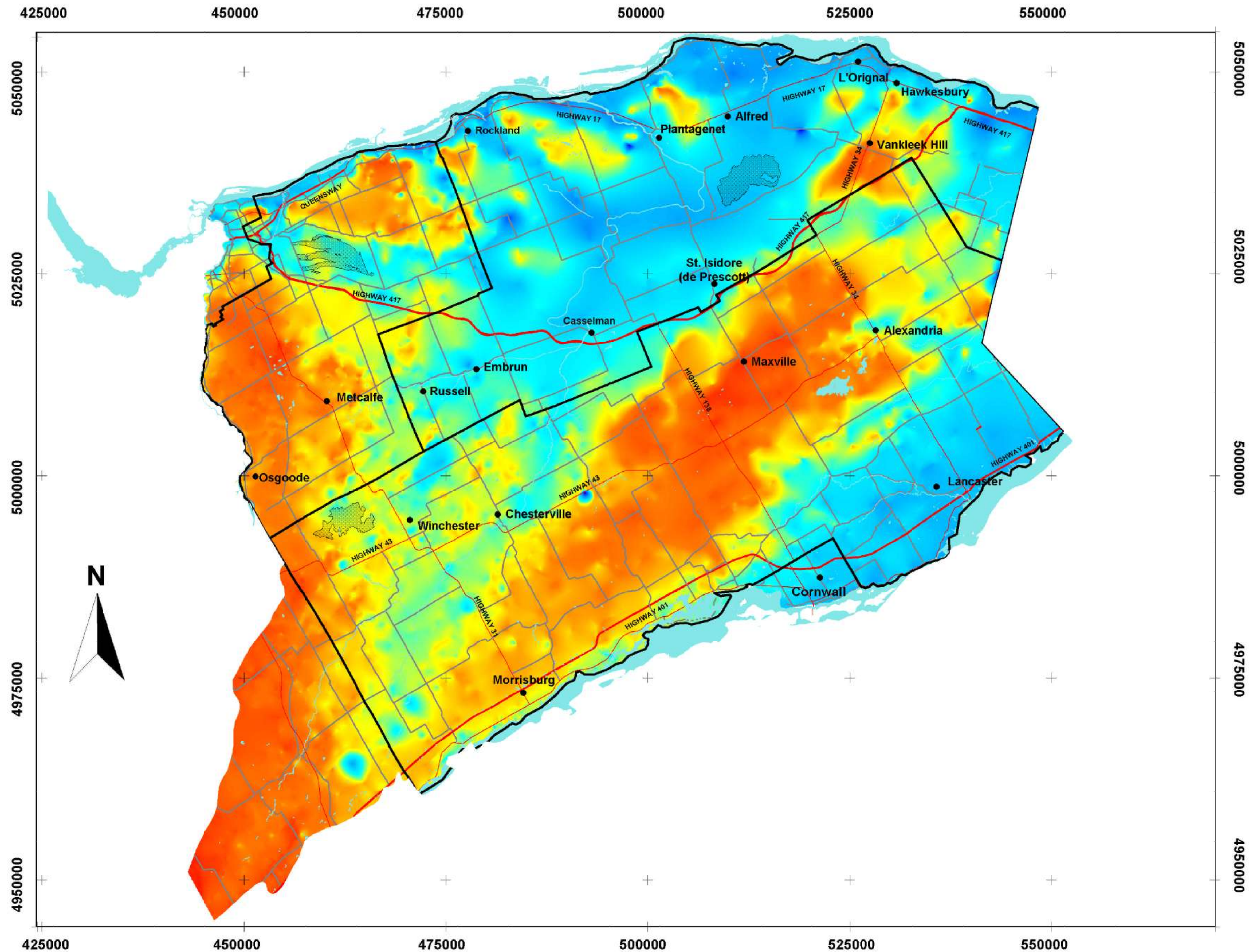
Eastern Ontario
WATER RESOURCES MANAGEMENT STUDY



Legend

130
87
75
63
0
metres (asl)

Wetlands
Counties
Roads
Highway
Major Highway
Faults
Hydrography

Scale 1 : 500 000

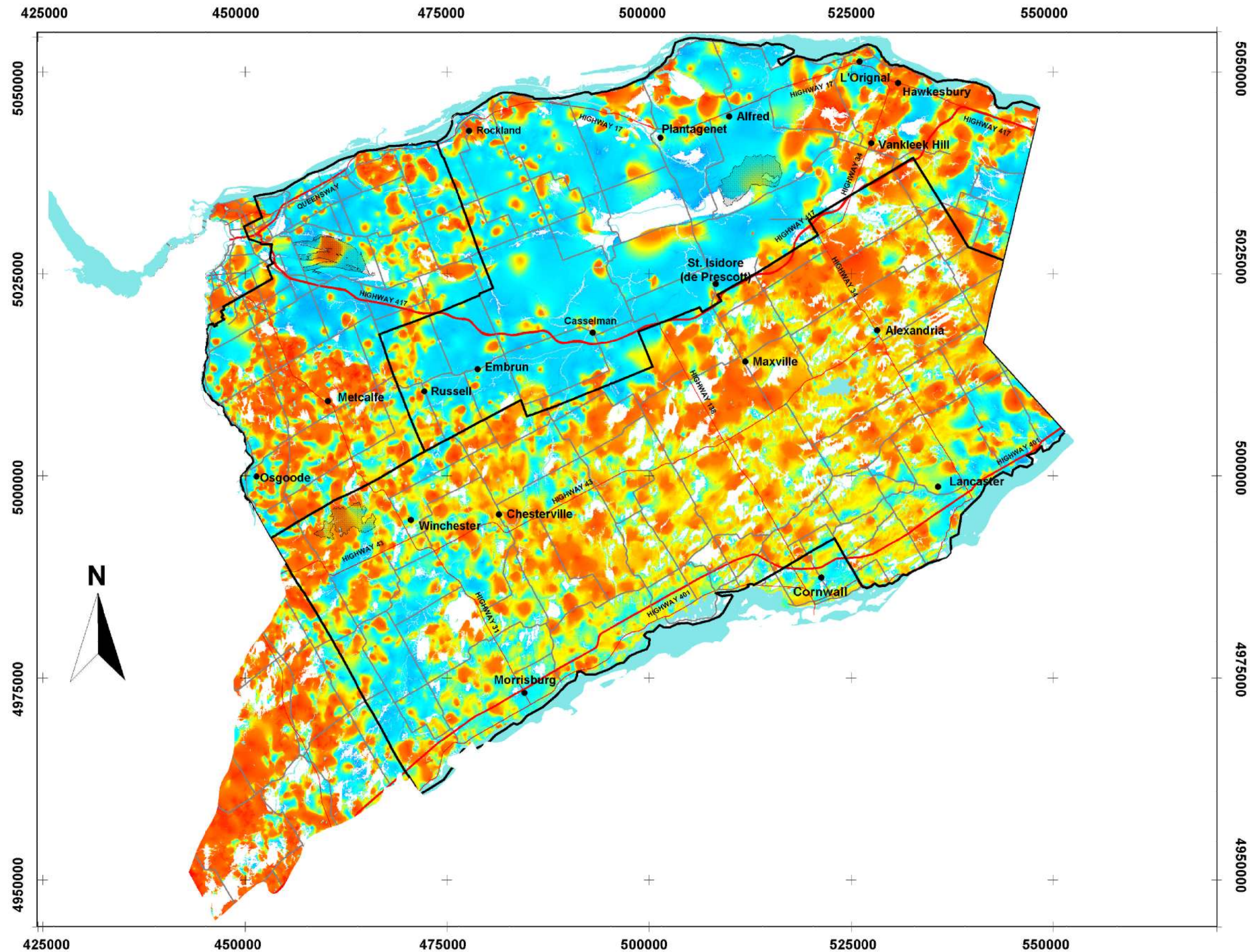
0 6 12 18 Kilometers

Deep Bedrock Aquifer Piezometric Surface

Figure 5-20

March 2001

CH2MHILL



Eastern Ontario
WATER RESOURCES MANAGEMENT STUDY

Legend

11.54
2.78
1.84
0.82
-4.48
log (mm/yr)

Wetlands
Counties
Roads
Highway
Major Highway
Faults
Hydrography

Scale 1 : 500 000

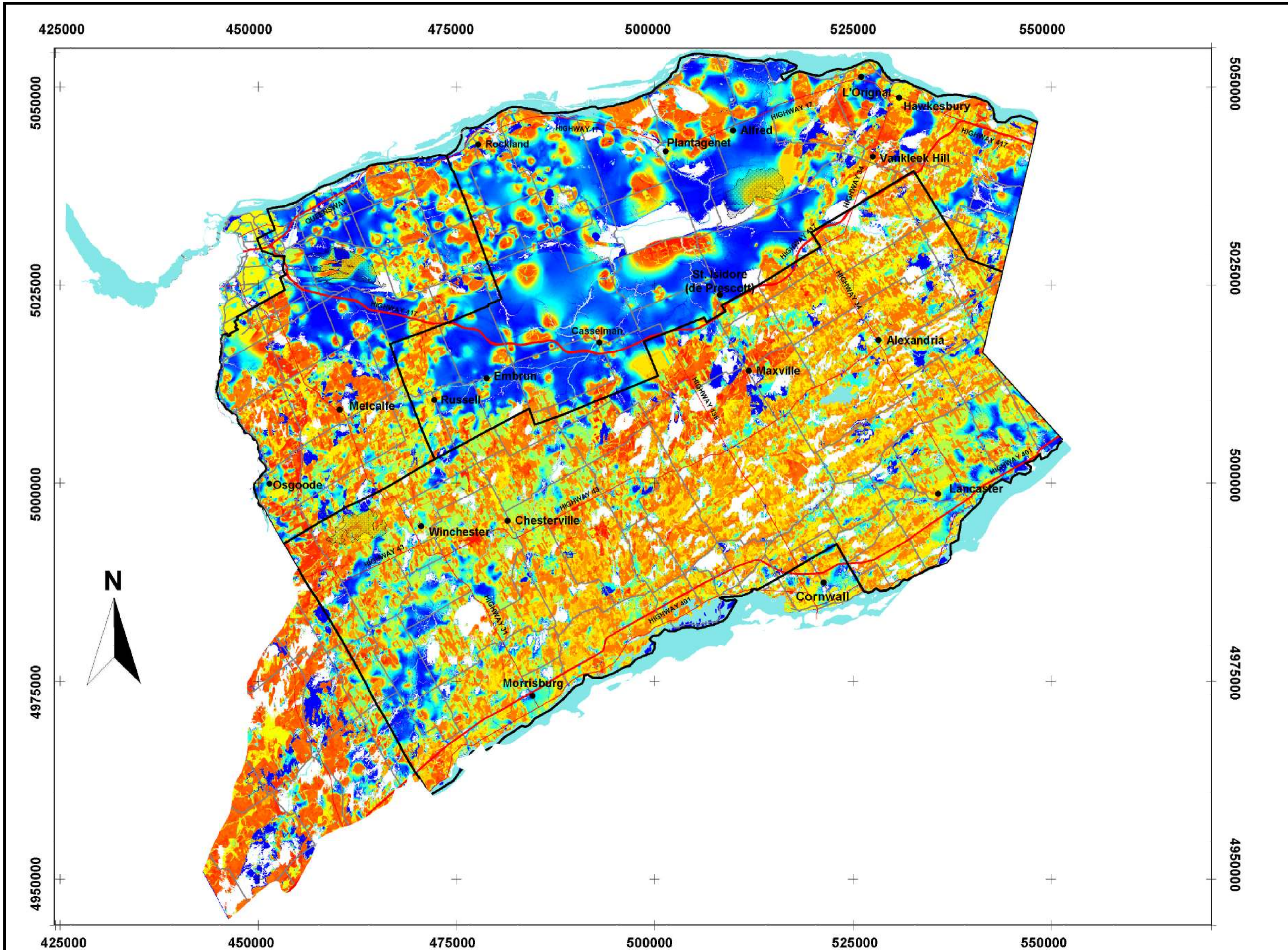
0 6 12 18 Kilometers

Net Groundwater Flux Downwards (Recharge Flux Potential)

Figure 5-21

March 2001

CH2MHILL



Eastern Ontario
WATER RESOURCES MANAGEMENT STUDY

Legend

28
14
3
0
mm/yr

Wetlands
Counties
Roads
Highway
Major Highway
Faults
Hydrography

Scale 1 : 500 000

0 6 12 18 Kilometers

Recharge to the Contact Zone Aquifer

Figure 5-22

March 2001

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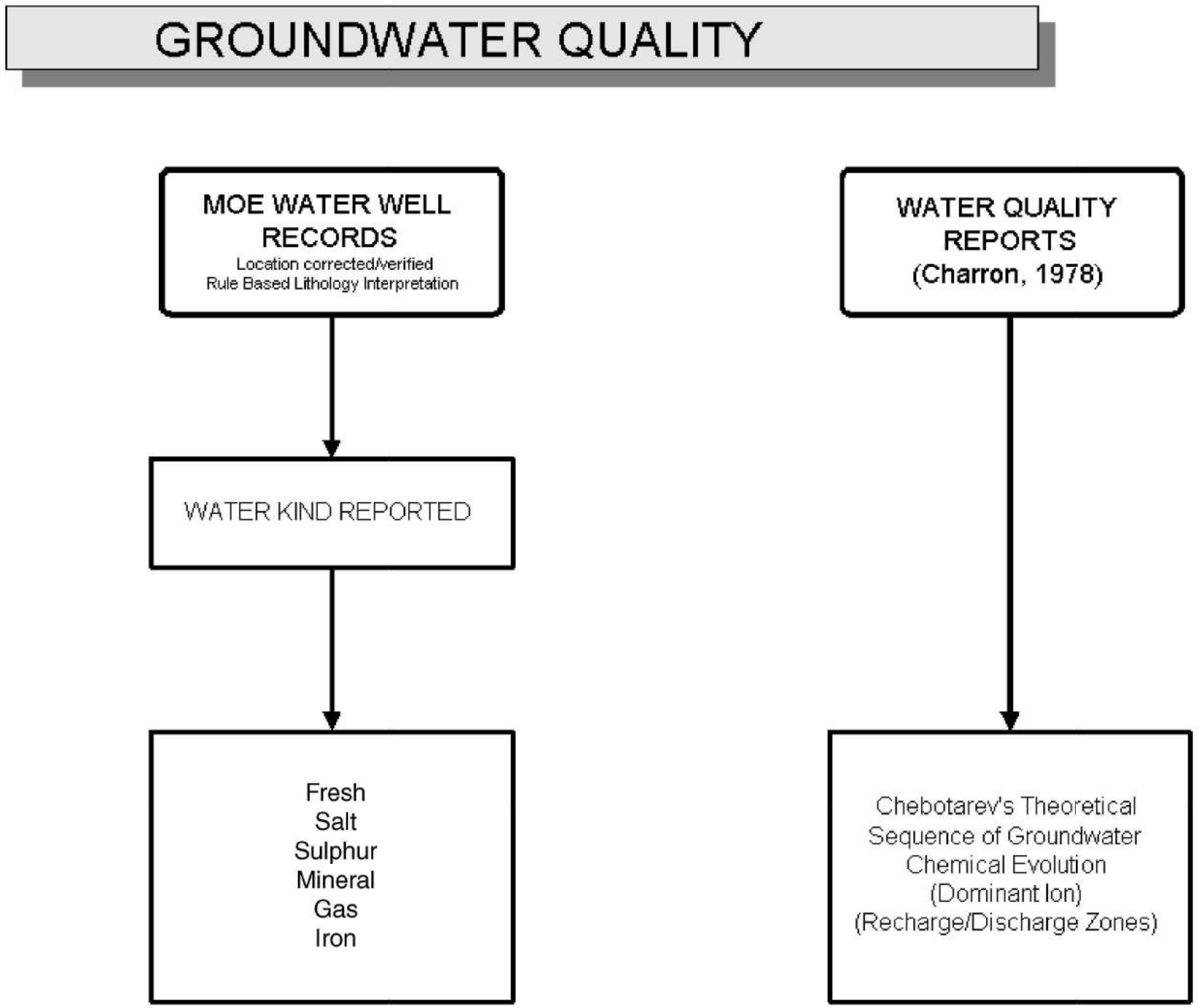

Eastern Ontario
WATER RESOURCES MANAGEMENT STUDY

Groundwater Quality

Figure 5-23

March 2001

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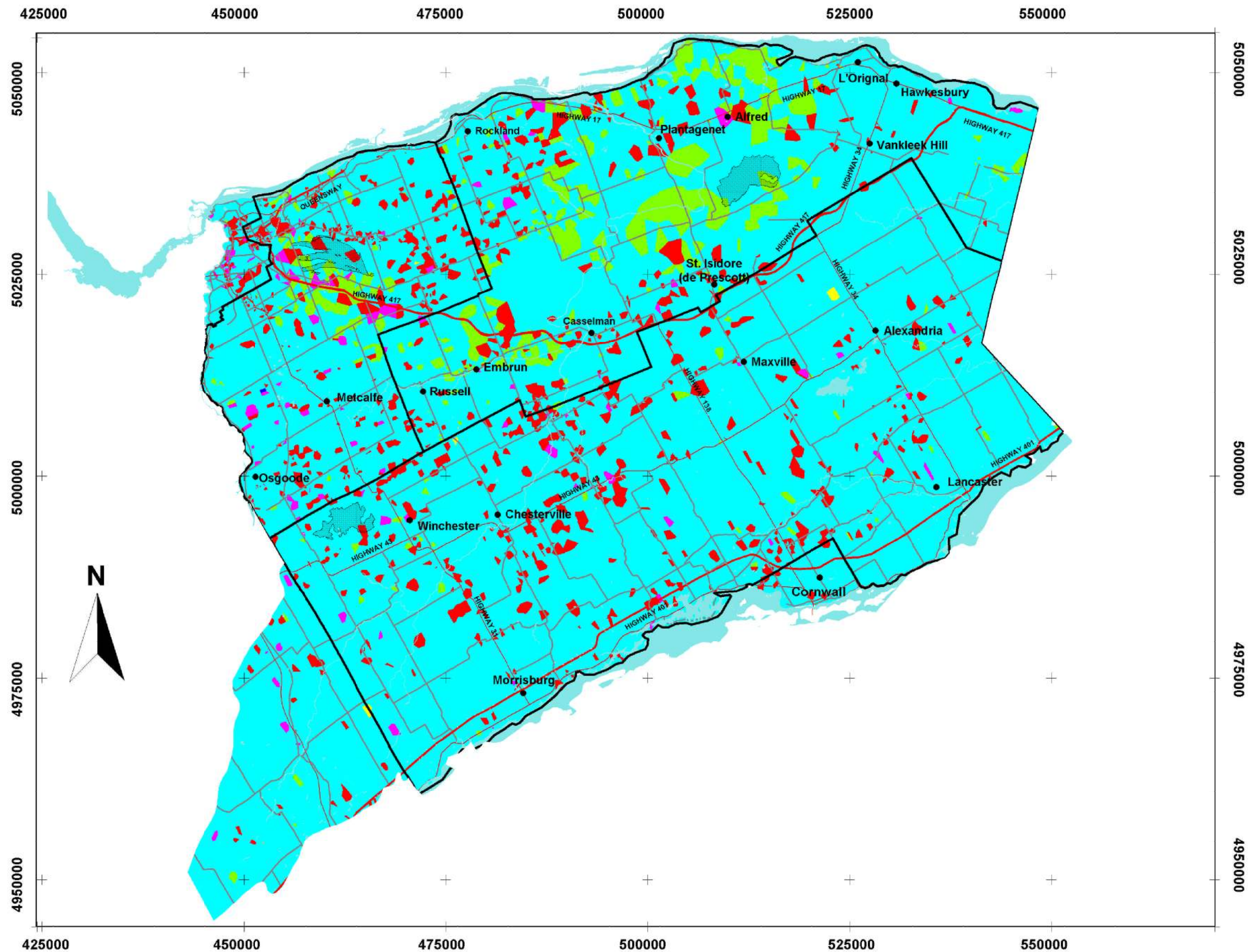

Eastern Ontario
WATER RESOURCES MANAGEMENT STUDY

Legend

Reported Water Type

- Fresh
- Salt
- Sulphur
- Mineral
- Gas
- Iron

- Wetlands
- Counties
- Roads
- Highway
- Major Highway
- Faults
- Hydrography

Scale 1 : 500 000

0 6 12 18 Kilometers

Natural Groundwater Quality

Figure 5-24

March 2001

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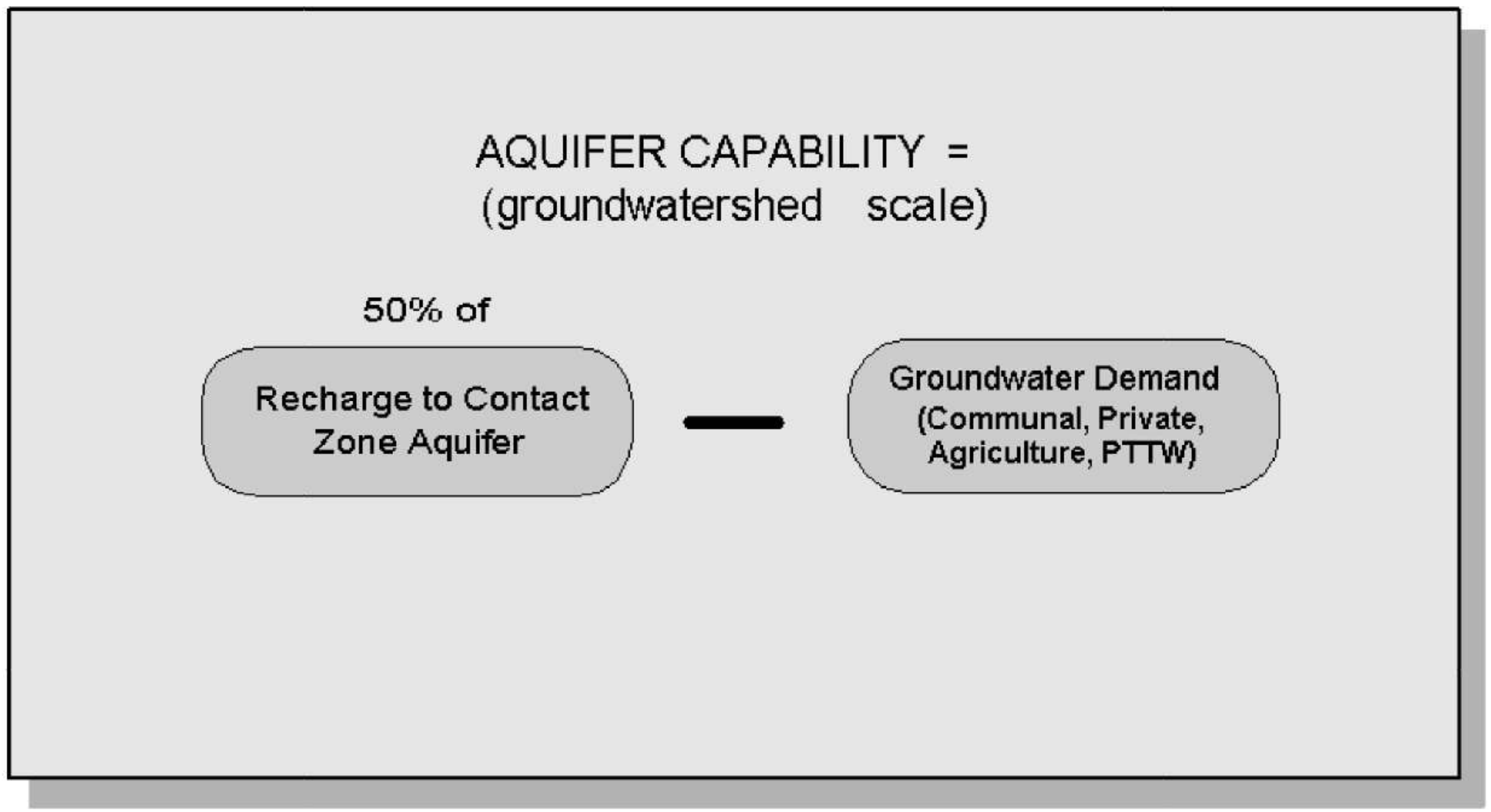
AQUIFER CAPABILITY



Eastern Ontario
WATER RESOURCES MANAGEMENT STUDY

Aquifer Capability

Figure 5-25

March 2001

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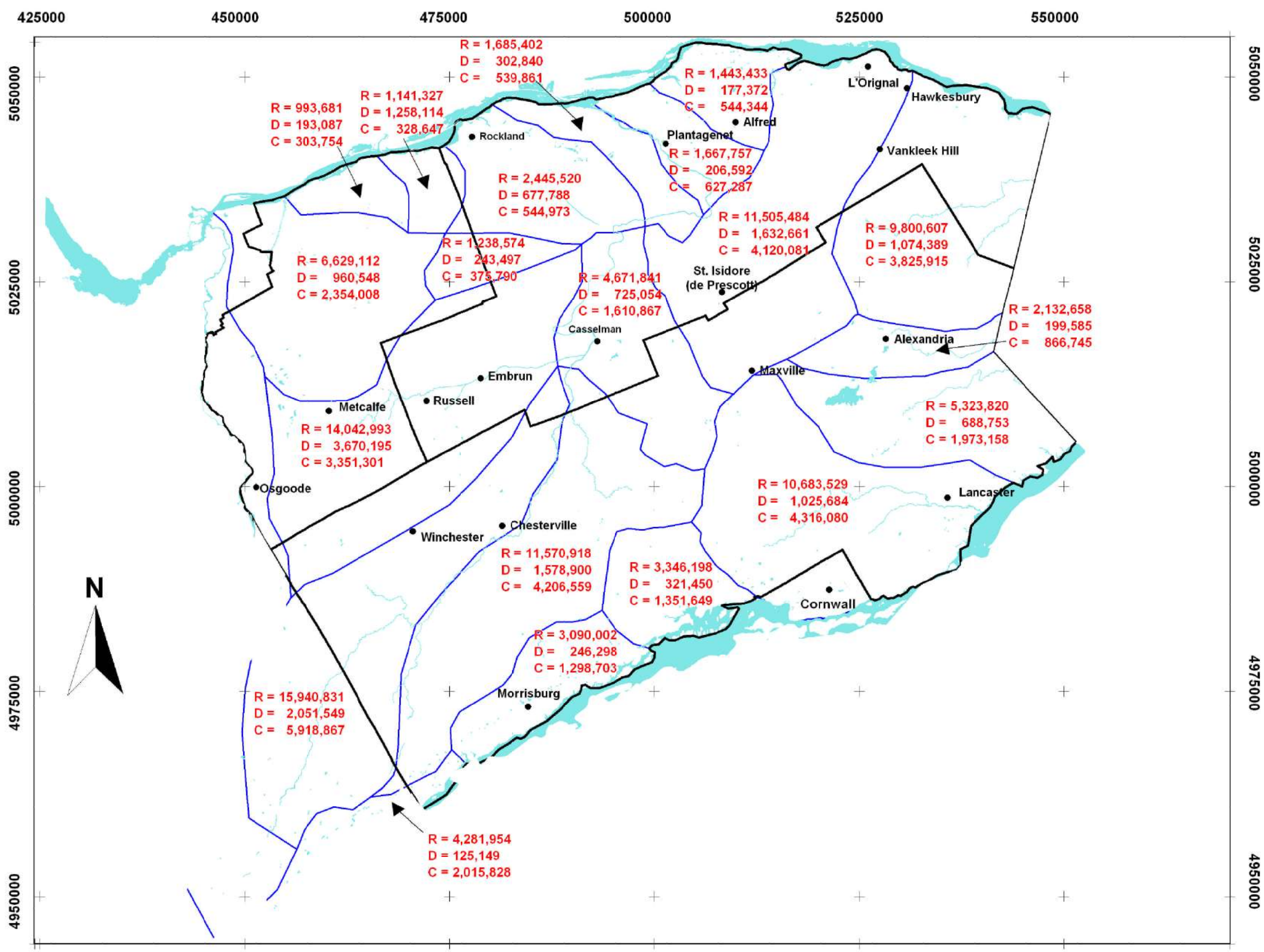

Eastern Ontario
WATER RESOURCES MANAGEMENT STUDY

Legend

- Counties
- Interpreted Groundwatersheds
- Hydrography
- R Recharge m^3/yr
- D Demand m^3/yr
- C Capability (50% of Recharge - Demand) m^3/yr

Scale 1 : 500 000

0 6 12 18 Kilometers

Estimated Aquifer Capability (Contact Zone)

Figure 5-26

March 2001

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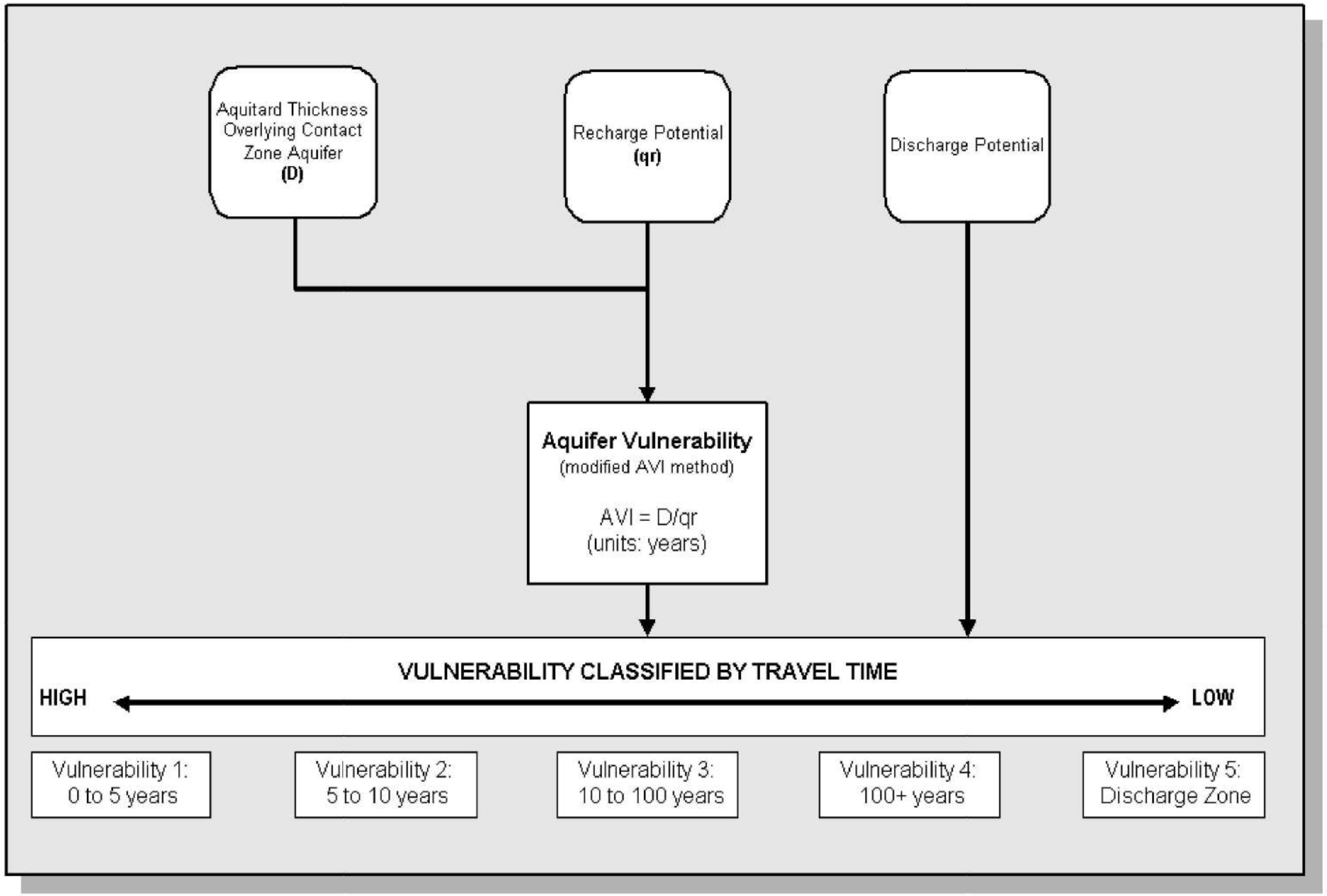
AQUIFER VULNERABILITY



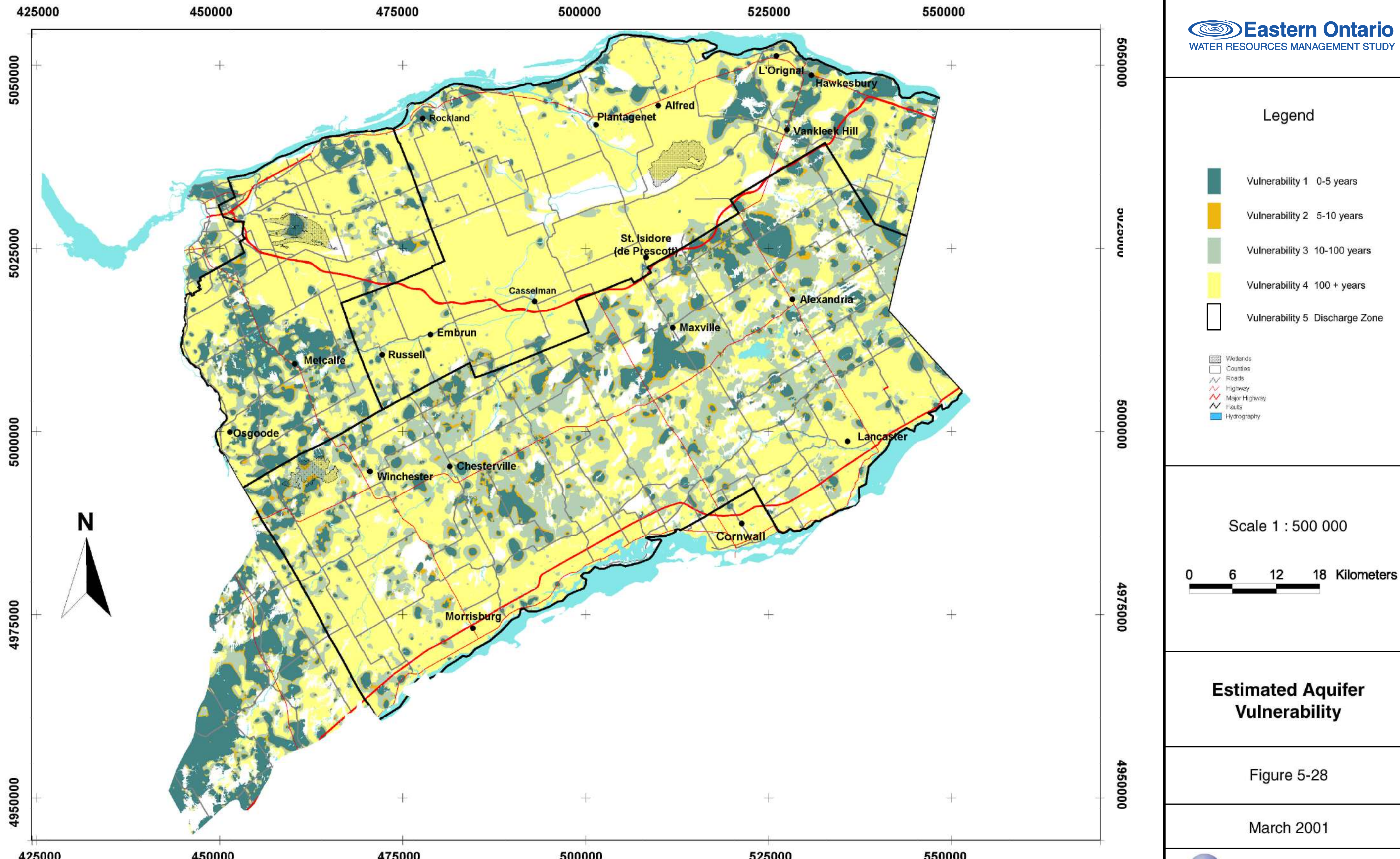
Eastern Ontario
WATER RESOURCES MANAGEMENT STUDY

Aquifer Vulnerability

Figure 5-27

March 2001

CH2MHILL



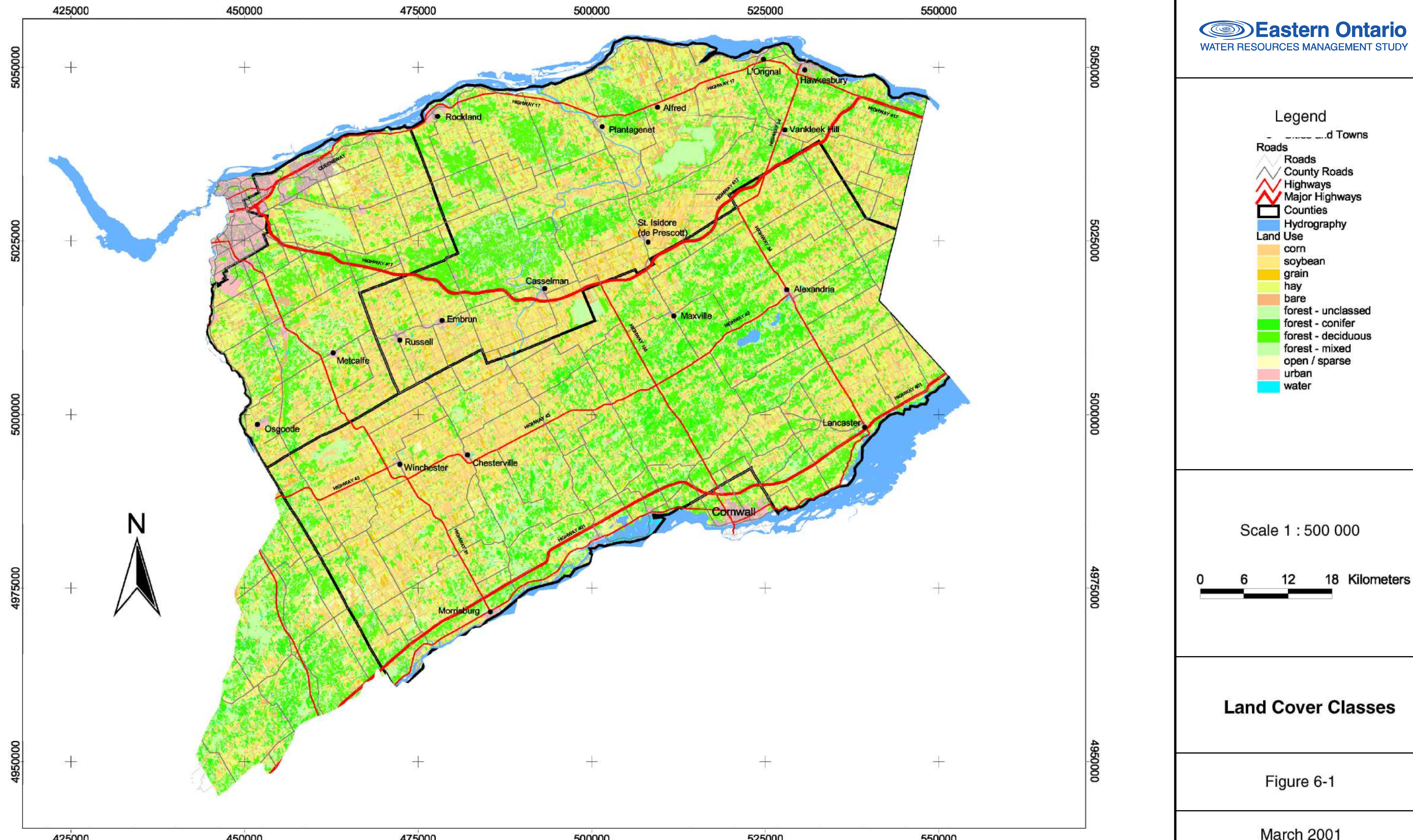
Eastern Ontario
WATER RESOURCES MANAGEMENT STUDY

Estimated Aquifer Vulnerability

Figure 5-28

March 2001

CH2MHILL



Land Cover Classes

Figure 6-1

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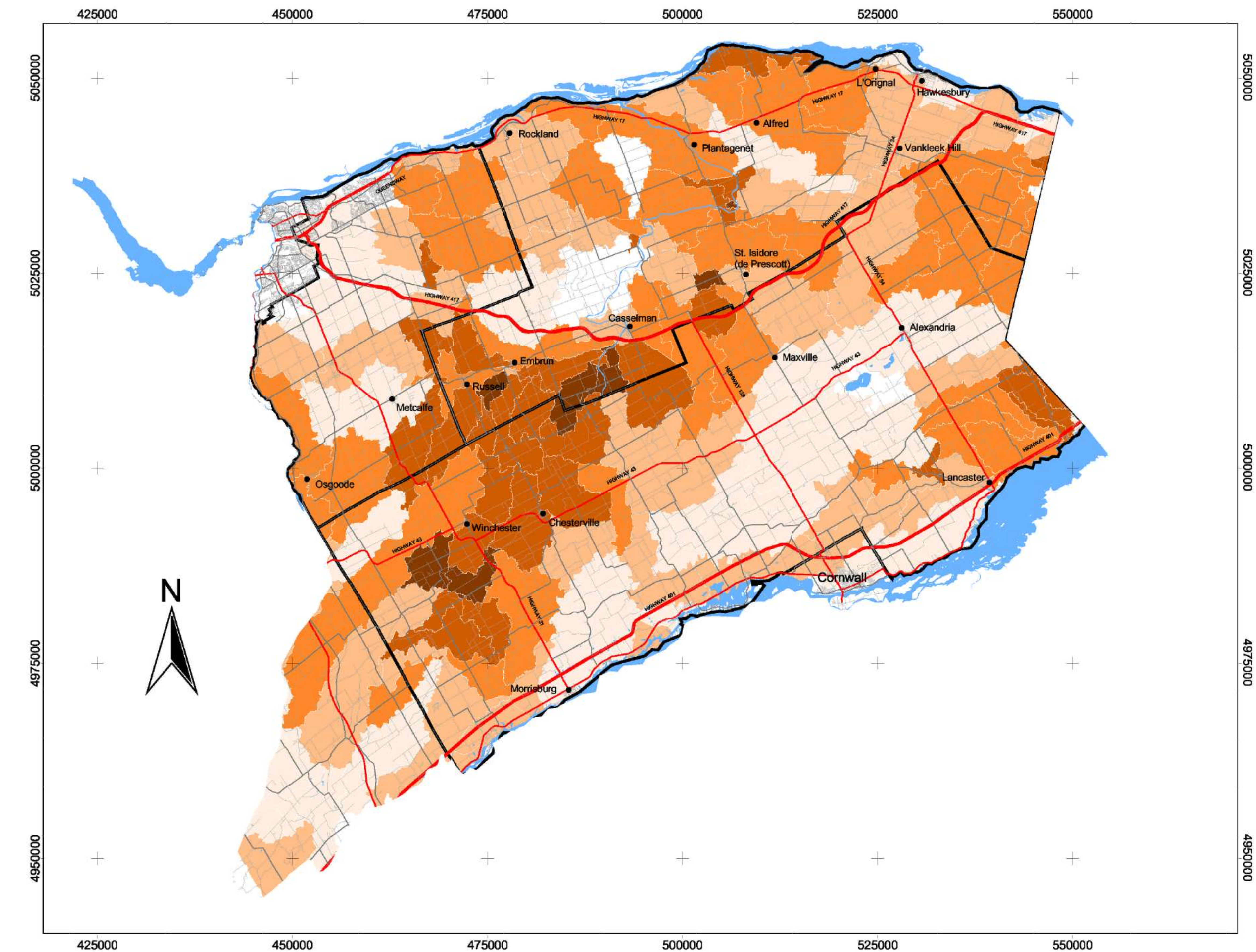
Eastern Ontario
WATER RESOURCES MANAGEMENT STUDY



Legend

- Cities and Towns

Roads
- Roads
- County Roads
- Highways
- Major Highways
- Counties
- Hydrography

Percentage of Agricultural Land by Subwatershed
- 25 - 45
- 46 - 60
- 60 - 75
- 76 - 90
- 91 - 100

Scale 1 : 500 000

0 6 12 18 Kilometers

Percent of Agriculture by Subwatershed-Map

Figure 6-2

March 2001

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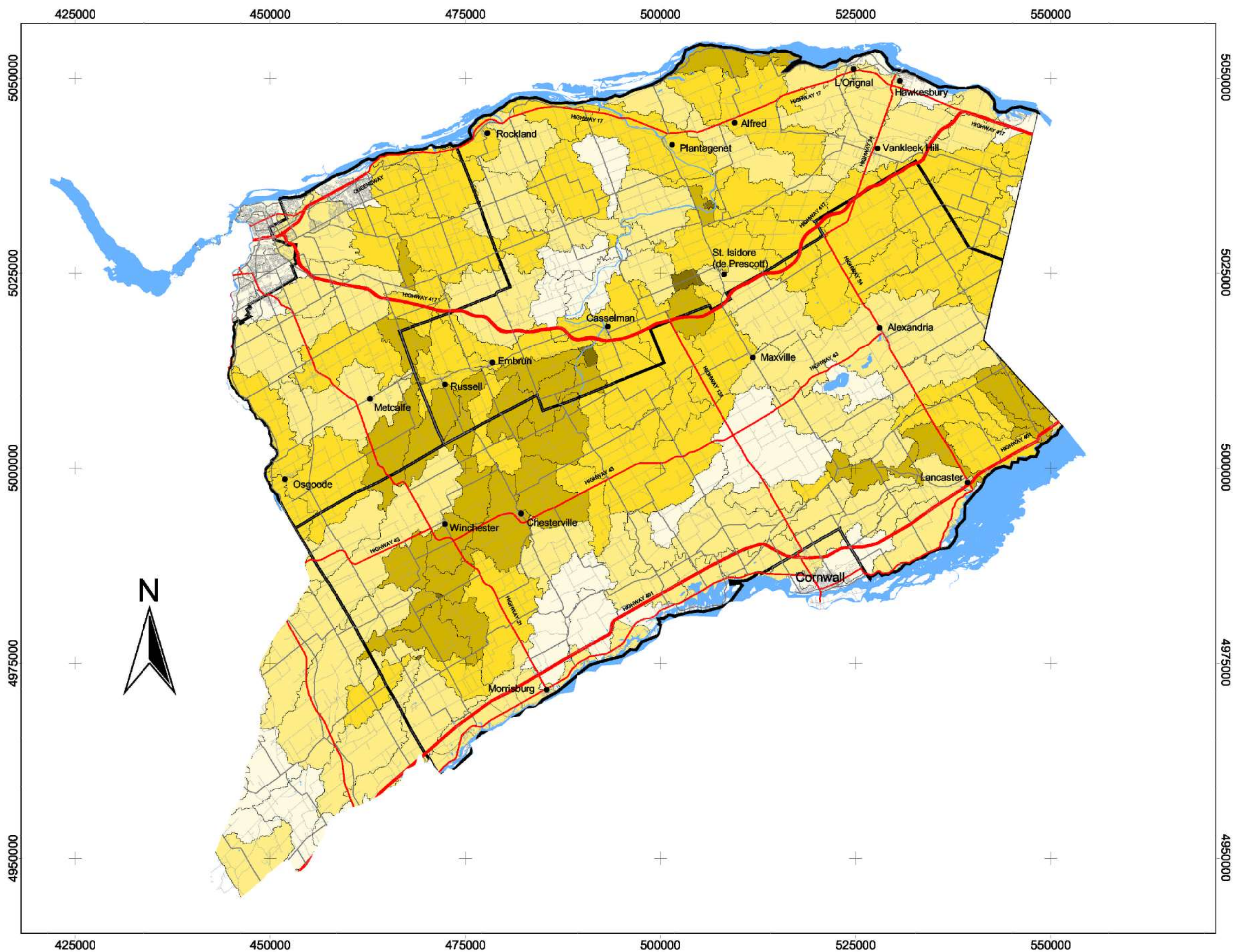
Eastern Ontario
WATER RESOURCES MANAGEMENT STUDY



Legend

- Cities and Towns

Roads

- Roads
- County Roads
- Highways
- Major Highways
- Counties
- Hydrography

Percent

- 0 - 20
- 21 - 40
- 41 - 60
- 61 - 80
- 81 - 100

Scale 1 : 500 000

0 6 12 18 Kilometers

Percent of Agriculture Close to Surface Drainage Network

Figure 6-3

March 2001

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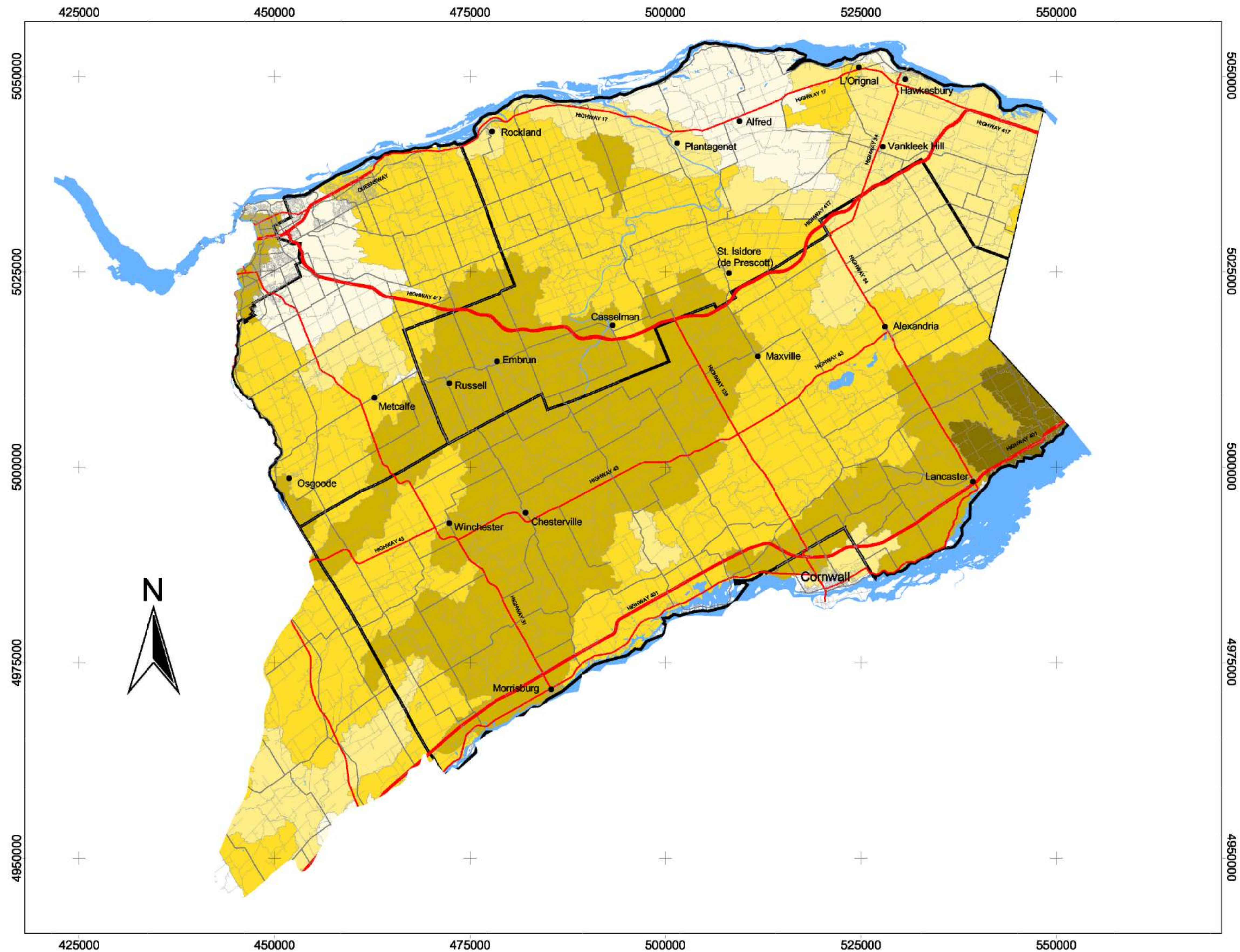
Eastern Ontario
WATER RESOURCES MANAGEMENT STUDY



Legend

- Cities and Towns
- Roads
  - Roads
  - County Roads
  - Highways
  - Major Highways
- Counties
- Hydrography
- Crop Intensity By Subwatershed
  - Low
  - Medium
  - High

Scale 1 : 500 000

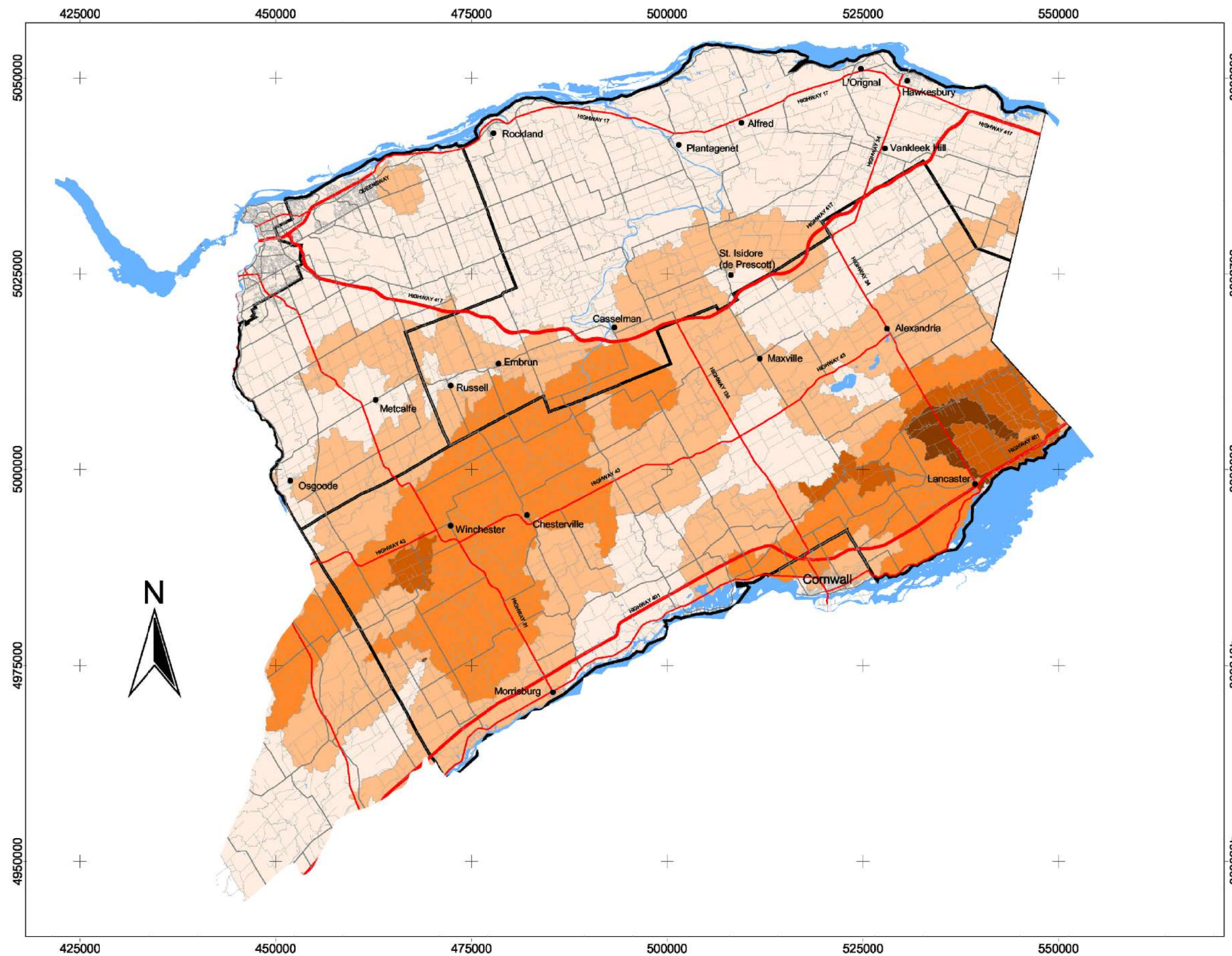
0 6 12 18 Kilometers

Crop Intensity by Subwatershed

Figure 6-4

March 2001

CH2MHILL



Eastern Ontario
WATER RESOURCES MANAGEMENT STUDY

Legend

- Cities and Towns

Roads
- Roads
- County Roads
- Highways
- Major Highways
- Counties
- Hydrography

RUSLE Soil Loss By Subwatershed
- Low
- Medium
- High

Scale 1 : 500 000

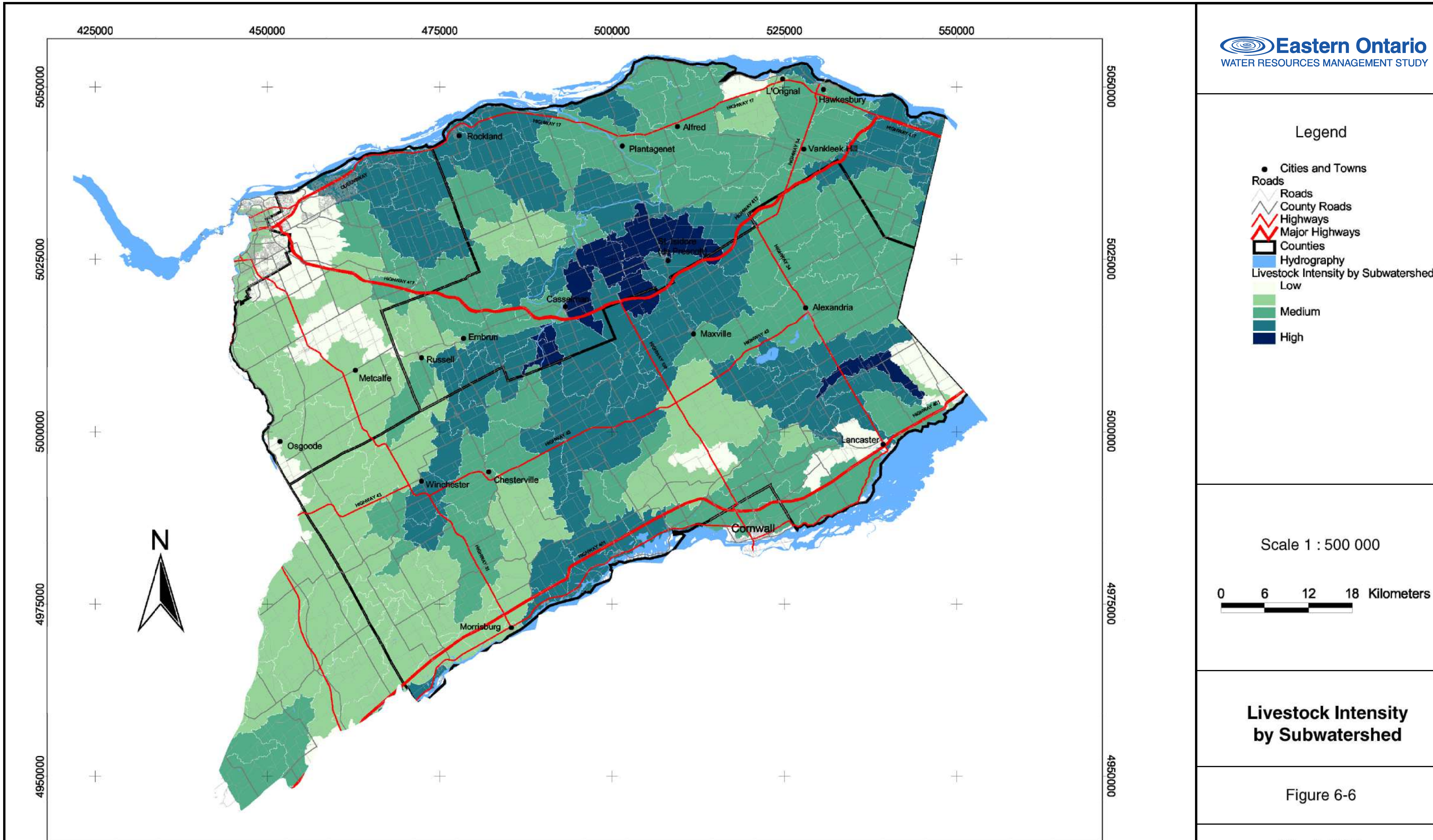
0 6 12 18 Kilometers

RUSLE Soil Loss by Subwatershed

Figure 6-5

March 2001

CH2MHILL



Eastern Ontario
WATER RESOURCES MANAGEMENT STUDY

Livestock Intensity by Subwatershed

Figure 6-6

March 2001

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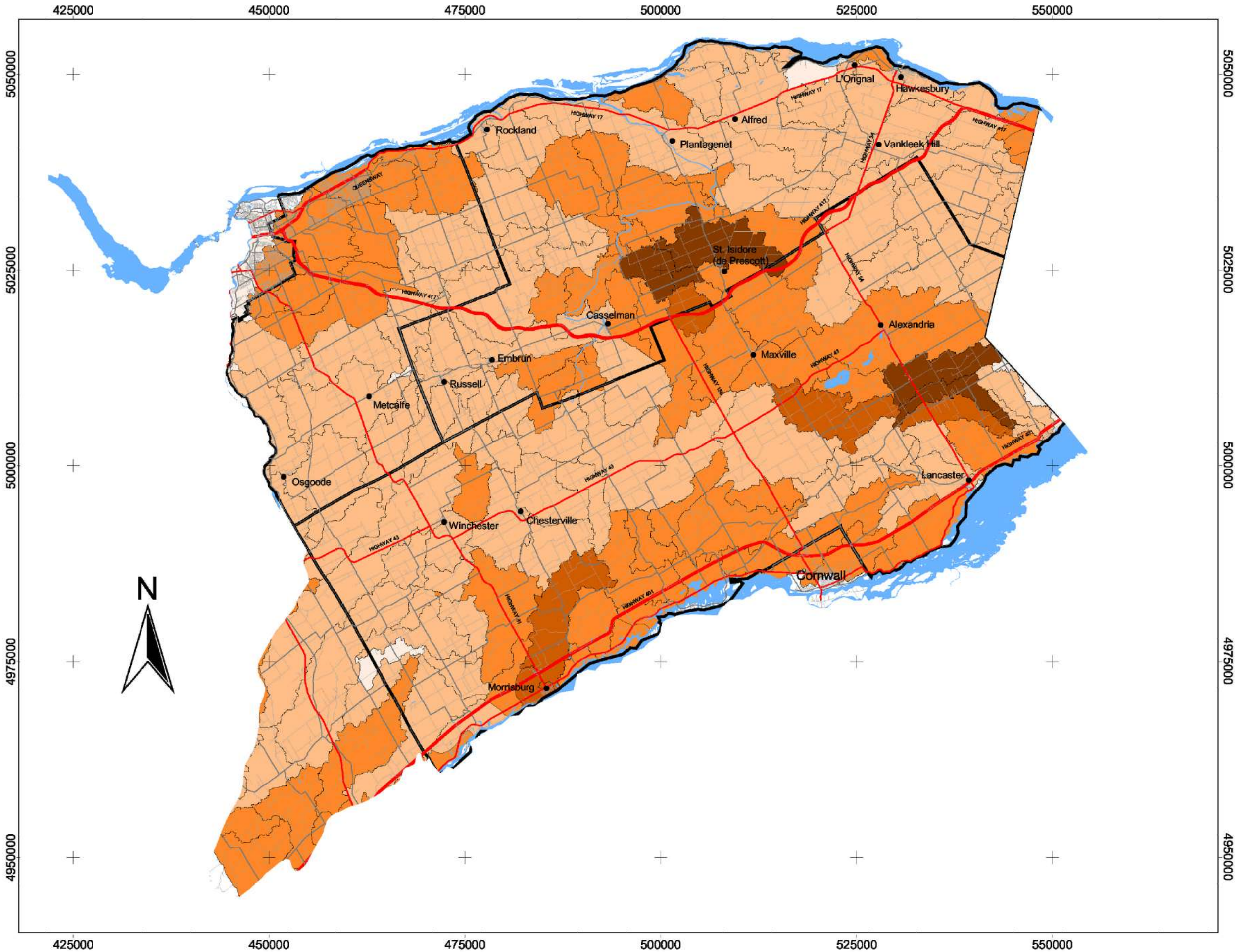

Eastern Ontario
WATER RESOURCES MANAGEMENT STUDY

Legend

- Cities and Towns

Roads
- Roads
- County Roads
- Highways
- Major Highways
- Counties
- Hydrography

Phosphorus balance by subwatershed
- Low
- Medium
- High

Scale 1 : 500 000

0 6 12 18 Kilometers

Phosphorus Balance by Subwatershed

Figure 6-7

March 2001

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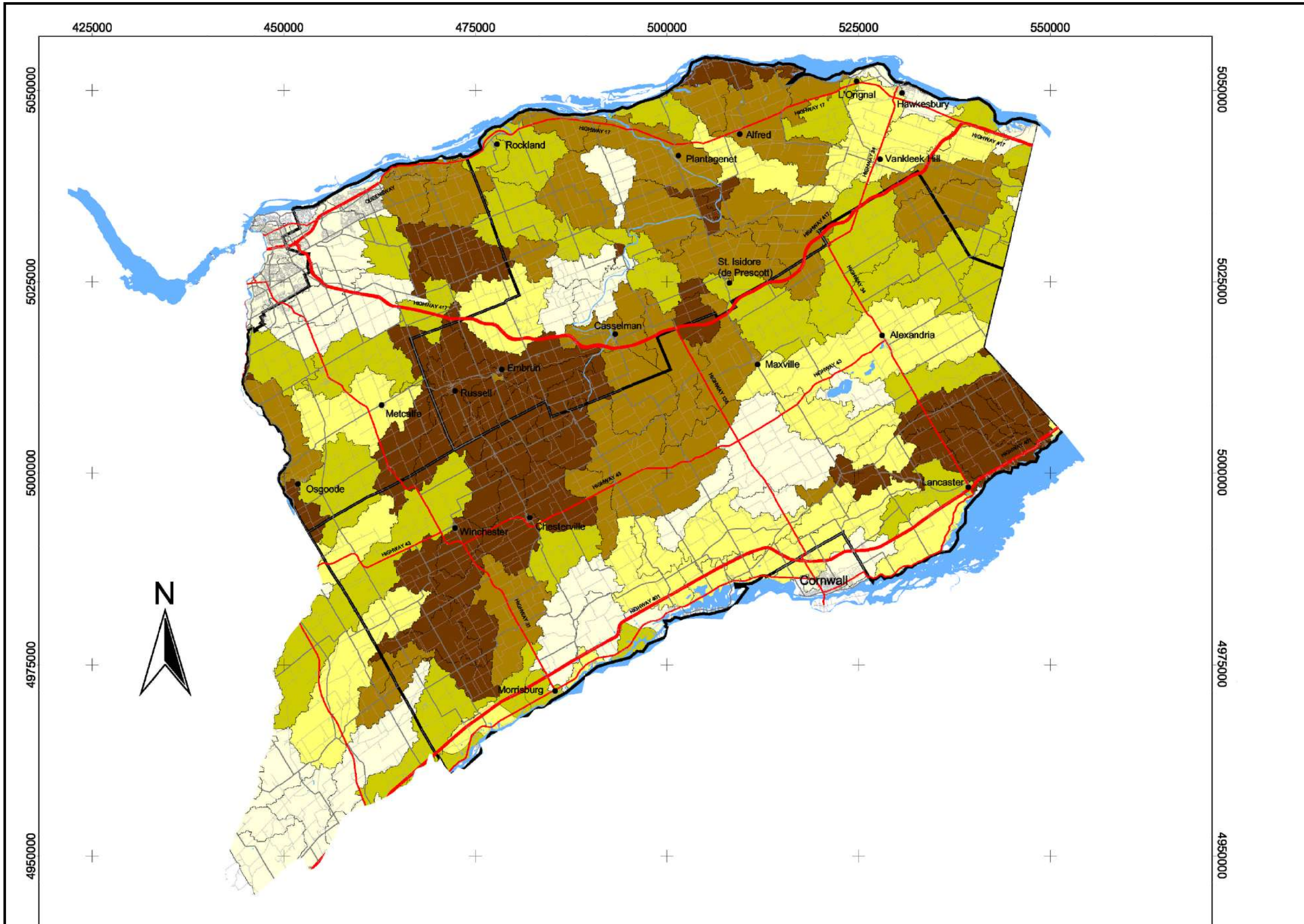


Eastern Ontario
WATER RESOURCES MANAGEMENT STUDY

Legend
Cities and Towns
Roads
Roads
County Roads
Highways
Major Highways
Counties
Hydrography
Extent-proximity-intensity crops
Low
Medium
High

Scale 1 : 500 000

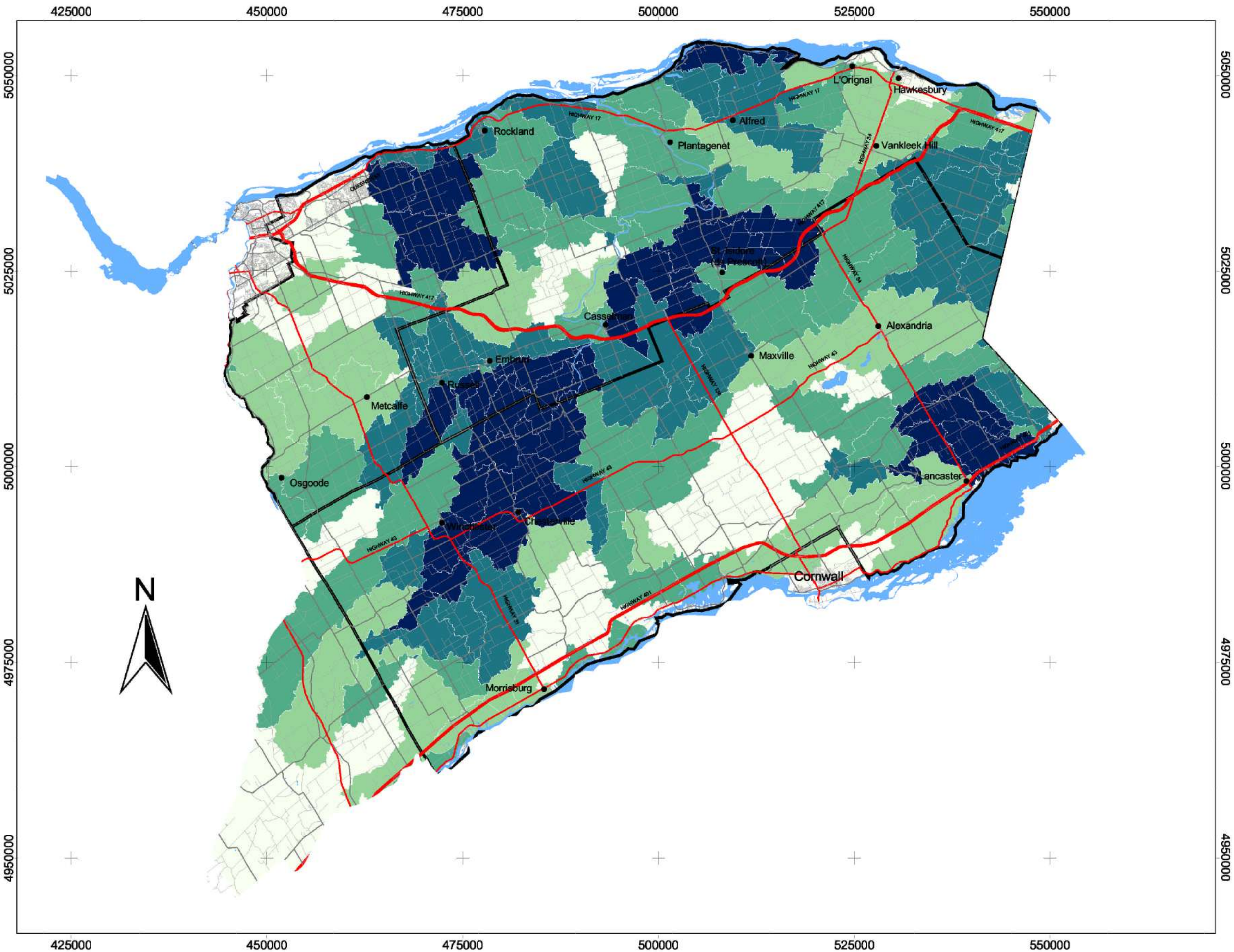
0 6 12 18 Kilometers

Extent/Proximity/Intensity - Crops (Surface Water)

Figure 6-8

March 2001

CH2MHILL



Eastern Ontario
WATER RESOURCES MANAGEMENT STUDY

Legend

- Cities and Towns

Roads
- Roads
- County Roads
- Highways
- Major Highways
- Counties
- Hydrography

Extent-proximity-intensity livestock
- Low
- Medium
- High

Scale 1 : 500 000

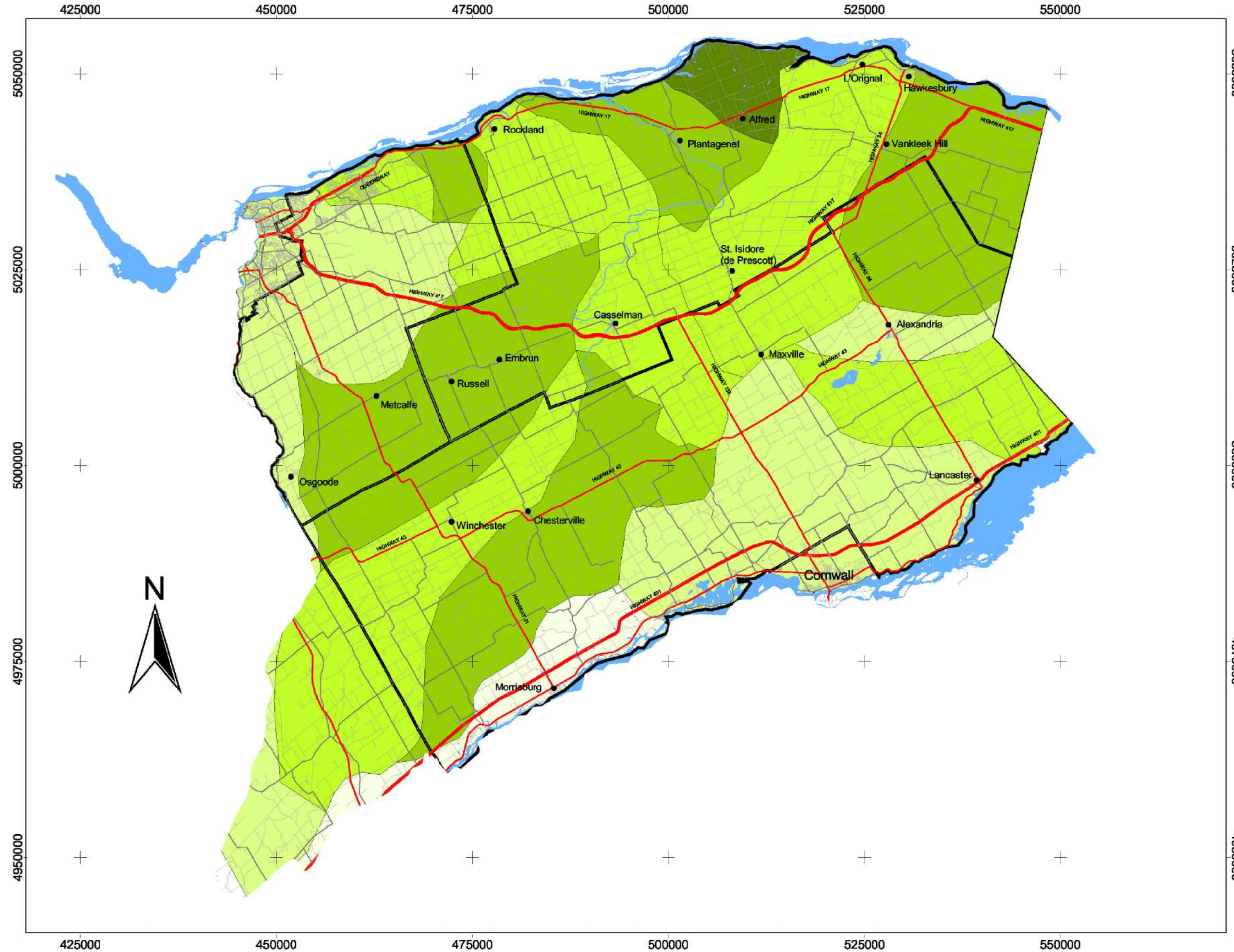
0 6 12 18 Kilometers

Extent/Proximity/intensity - Livestock (Surface Water)

Figure 6-9

March 2001

CH2MHILL



Eastern Ontario
WATER RESOURCES MANAGEMENT STUDY

Legend

- Cities and Towns

Roads
- Roads
- County Roads
- Highways
- Major Highways
- Counties
- Hydrography

Percent
- 30 - 40
- 40 - 50
- 50 - 60
- 60 - 70
- 70 - 100

Scale 1 : 500 000

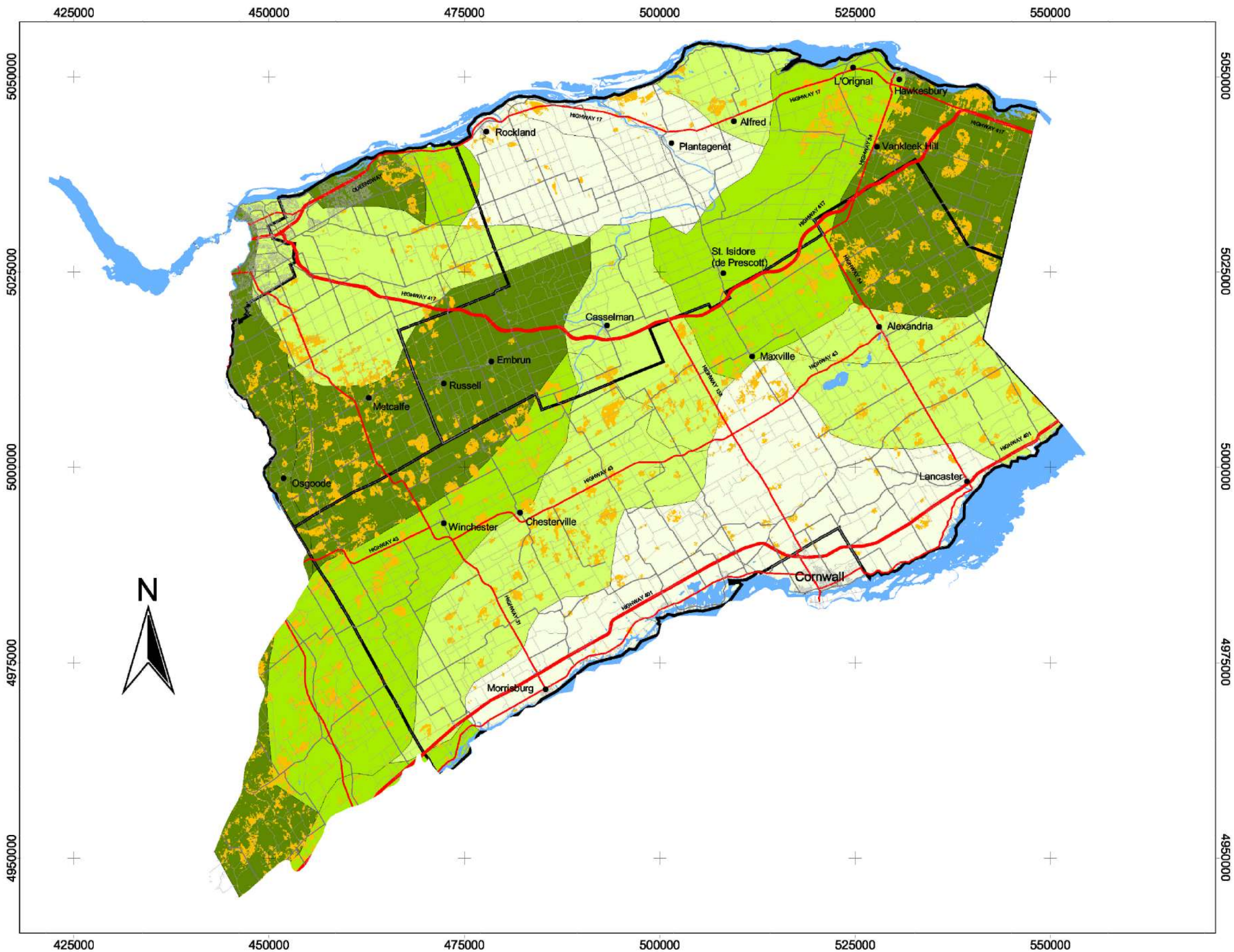
0 6 12 18 Kilometers

Percent of Land in Agriculture by Groundwatershed

Figure 6-10

March 2001

CH2MHILL



Eastern Ontario
WATER RESOURCES MANAGEMENT STUDY

Legend

- Cities and Towns
- Roads
  - Roads
  - County Roads
  - Highways
  - Major Highways
- Counties
- Hydrography
- Agricultural Land within Groundwater Vulnerability Classes 1 and 2

Percent of Agriculture within Vulnerability Classes 1 and 2 by Groundwatershed

- 1 - 4
- 5 - 8
- 9 - 12
- 13 - 16

Scale 1 : 500 000

0 6 12 18 Kilometers

Percent of Agriculture within Groundwater Vulnerability Classes 1 & 2

Figure 6-11

March 2001

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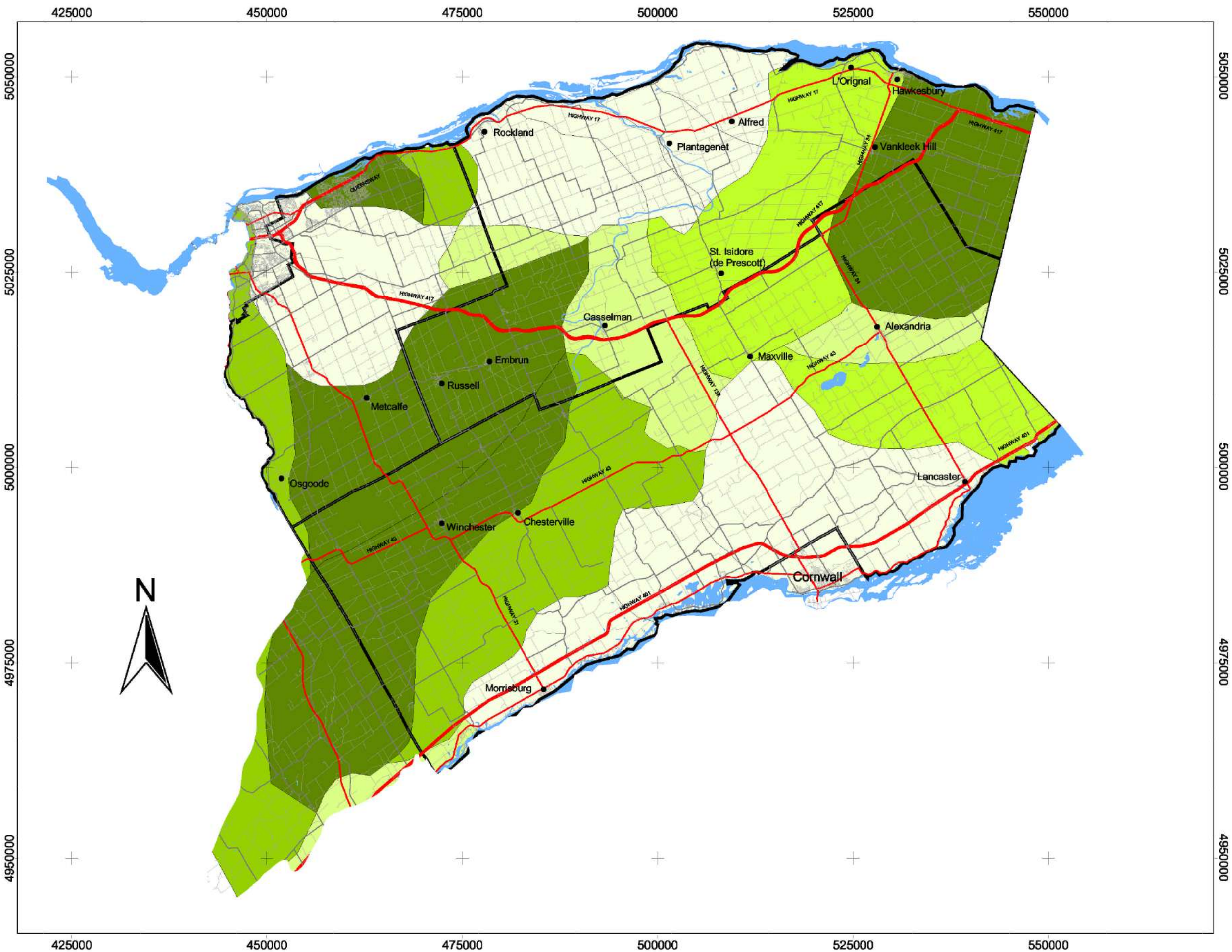

Eastern Ontario
WATER RESOURCES MANAGEMENT STUDY

Legend

- Cities and Towns

Roads
- Roads
- County Roads
- Highways
- Major Highways
- Counties
- Hydrography

Crop Extent/Proximity/Intensity
- Very Low
- Low
- Moderately Low

Scale 1 : 500 000

0 6 12 18 Kilometers

Extent/Proximity/Intensity - Crop (Groundwater)

Figure 6-12

March 2001

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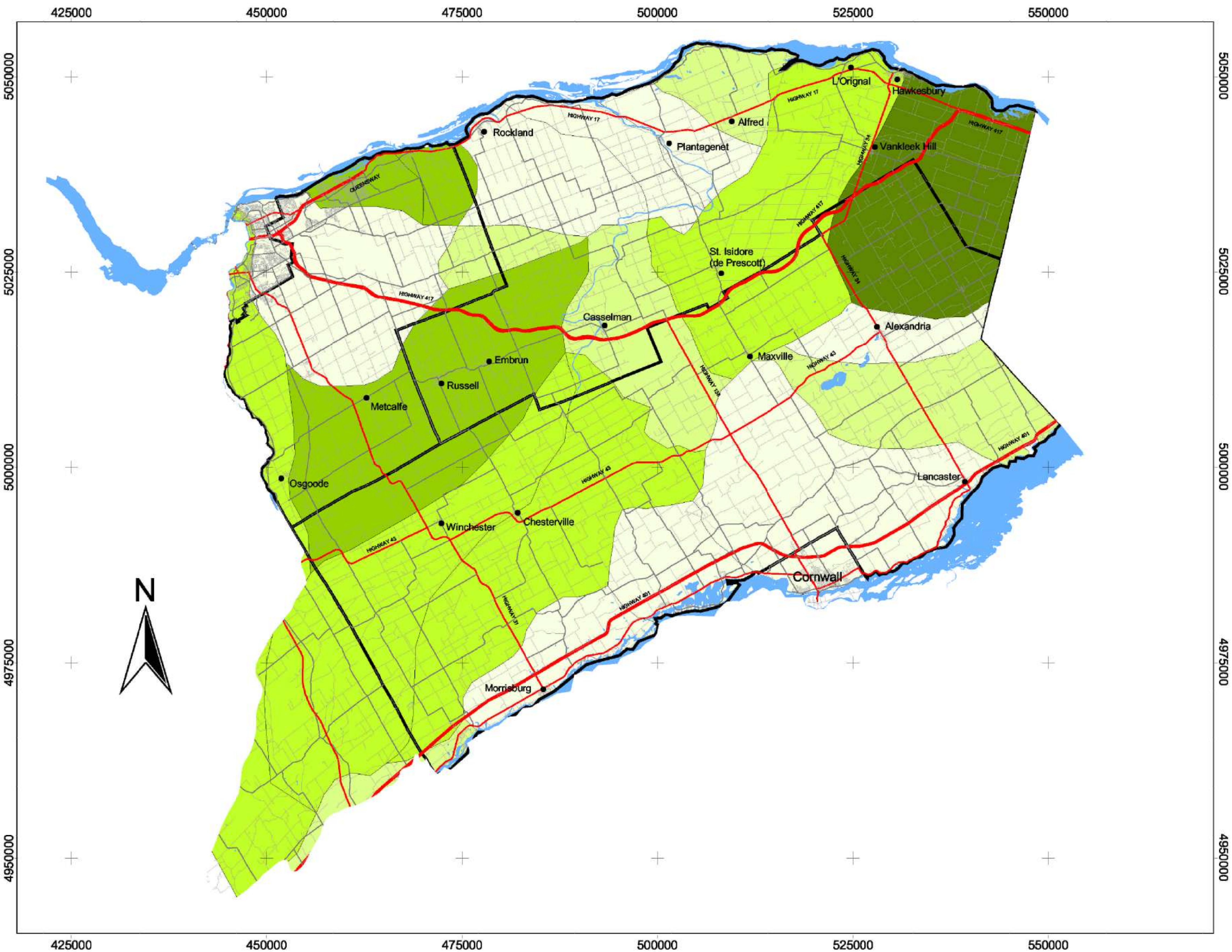
Digital Mapping Sources: Various Ministries of the Province of Ontario and Departments of the Federal Government. Reference should be made to metadata compiled for this study.
This map is intended for illustrative purposes only. Do not rely on it as being a precise indicator of routes, locations of features, nor as a guide to navigation.



Eastern Ontario
WATER RESOURCES MANAGEMENT STUDY

Legend

- Cities and Towns

Roads
- Roads
- County Roads
- Highways
- Major Highways
- Counties
- Hydrography

Livestock Extent/Proximity/Intensity
- Extremely Low
- Very Low
- Low

Scale 1 : 500 000

0 6 12 18 Kilometers

Extent/Proximity/Intensity - Livestock (Groundwater)

Figure 6-13

March 2001

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