

**OBJECT-ORIENTED PROGRAMMING IN
RIVER WATER QUALITY CONTROL**

by

**Maria N. Spanou
Dipl. Chem. Eng.**

A Thesis

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A b s t r a c t

Water quality management of river systems has been a field of continuous research in the last decades. In addition mathematical modelling and computer science developments have become today an accepted part of decision making. Object-oriented methodology offers the potential for an efficient analysis of the complex water resources systems, and for its implementation in software tools that can assist engineers and managers in policy and decision making.

In the present work object-oriented analysis and design have been applied for the quality management of river systems. The physical entities of the river system, as well as the conceptual entities for the flow and water quality analysis, the simulation and the pollution control strategies, have been represented through objects. By distributing appropriate responsibilities to these objects, daily low flows of the river gauge stations can be estimated for a duration of seven and thirty days and a recurrence interval of twenty years. Based on these flows optimum windows for the whole system and minimum daily flows of the river gauge stations can be also estimated. From a regression analysis between the latter flows and the drainage area of the stations, the minimum daily flows of each point of the river system can be provided. These are the design river flows for the management study.

A simple statistical analysis of water quality in the river and the effluent of the plants can be also performed and the background concentrations of major constituents can be estimated. The maximum allowed concentrations of total ammonia and hydrogen sulphide in the plants effluent, that satisfy the in-pipe limits of the toxic undissociated forms of the above compounds can be also estimated. A dilution analysis can be also

performed and identify the river discharge points whose available dilution is not sufficient to meet the required dilution for the compliance with the river limits of the above toxic compounds. In this analysis each plant is considered to discharge separately in the river system under conditions of perfect mixing in the discharge points.

The simulation of water quality in the system can be also performed based on the design conditions of the study. A steady state, node-reach, lumped parameter model has been considered that predicts the variation of the dissolved oxygen, ultimate BOD and total ammonia. The point sources of the plants' effluent, and the non-point sources which are quantified through the background concentration and the design river flows are considered in the model. The point sources are assumed to discharge their annual waste volume within the time period of the optimum windows.

Control strategies for the improvement of water quality can be finally applied and an optimized discharge scheme can be suggested. The alternative options that are considered are staging of the discharge of the plants in the optimum windows, relocation of the discharge points and change of the degree of treatment of the plants. The objective of the optimization is the appropriate utilization of the assimilative capacity of the river and the minimization of the required relocation of the present discharge points as well as of the degree of plants upgrade.

The above have been integrated with an object-oriented design of a data management component and a graphical user interface. These allow the input of data from files and the generation of output reports, the visualisation of the river system, and the flow control of the analysis. The definition and design of all objects has been made having in mind their future development or refinement. The object-oriented analysis and design have been further implemented in terms of Smalltalk/V and resulted in the development of a user-friendly and flexible software tool. The code has been validated for the case of the South Nation River Watershed in the Province of Ontario in Canada. The results have been in very good agreement with previous findings.

A c k n o w l e d g m e n t s

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Nomenclature

A_D	drainage area, km ²
C	cost of the application of a water quality control method
C	concentration of constituent with the name C , mg/m ³ , mg/l
C_b	background concentration in the whole river system, mg/m ³ , mg/l
C_f	final concentration of a constituent C , mg/m ³ , mg/l
$C_{j,j+1}^R(x_f)$	total concentration at the end of the reach between points j and $j + 1$, mg/m ³ , mg/l
$C_{j,j+1}^R(x_0)$	concentration at the beginning of the reach between points j and $j + 1$, mg/m ³ , mg/l
C_m	river concentration after mixing, mg/m ³ , mg/l
C_r	river concentration before mixing, mg/m ³ , mg/l
C_w	point source effluent concentration, mg/m ³ , mg/l
C_0	initial concentration of a constituent C , mg/m ³ , mg/l
<i>const.</i>	regression coefficient in flow-drainage area relation, m ³ /s/km ²
$(H_2S)_{tot}$	total measured hydrogen sulphide, mg/l
$(H_2S)_u$	undissociated hydrogen sulphide, mg/l
DOD	concentration of oxygen deficit, mg/m ³ , mg/l
D	dilution factor, -

d	stream depth, m
D_{av}	available dilution factor, -
D_{req}	required dilution factor, -
DO	dissolved oxygen concentration, mg/l
DO_s	saturation concentration of dissolved oxygen, mg/l
$\%DO_s$	percentage of saturation of dissolved oxygen, %
k_a	specific rate of reaeration, d^{-1}
$k_{a,20}$	specific rate of reaeration at temperature $20^{\circ}C$, d^{-1}
k_{bod}	areal rate of the benthic oxygen demand, $mg/m^2/d$
k_n	specific rate of total ammonia oxidation to nitrate, d^{-1}
$k_{n,20}$	specific rate for the ammonia hydrolysis at $20^{\circ}C$, d^{-1}
k_r	specific removal rate of BOD, d^{-1}
$k_{r,lab}$	specific laboratory rate of BOD removal, d^{-1}
k_T	specific rate of change in concentration at temperature T , d^{-1}
L	concentration of carbonaceous biochemical oxygen demand, mg/m^3 , mg/l
$I_{np}^{A,B}$	nonpoint source load between river sites A and B , mg/s
L_u	concentration of BOD_u , mg/m^3 , mg/l
l_w	treatment plant waste load, mg/s
L_5	concentration of remaining oxygen demand after 5 days, up to the ultimate value, mg/m^3 , mg/l
N	concentration of total ammonia, mg/m^3 , mg/l
$(NH_3)_{tot}$	total measured ammonia, mg/m^3 , mg/l
$(NH_3)_u$	undissociated ammonia, mg/m^3 , mg/l
P	probability, -
Q	volumetric flowrate, m^3/d , m^3/s

$Q_{j,j+1}^R(x_0)$	flowrate at the beginning of the reach between points j and $j + 1$, $m^3/d, m^3/s$
$Q_{j,j+1}^R(x_f)$	total flowrate at the end of the reach between points j and $j + 1$, $m^3/d, m^3/s$
$Q_{min,low}^7$	$Q_{t,low}^7$ flow which is always equaled or exceeded in the time period $[t_1, t_2], m^3/s$
$Q_{min,low}^{30}$	$Q_{t,low}^{30}$ flow which is always equaled or exceeded in the time period $[t_1, t_2], m^3/s$
$(Q_{min,daily}^7)_{wType}$	minimum daily $Q_{t,low}^7$ flow of a station during the window of type $wType, m^3/s$
Q_r	river flowrate, $m^3/d, m^3/s$
$Q_{t,y}^7$	average daily flow over the 7-day period with median day, day $t, m^3/s$
$Q_{t,y}^{30}$	average daily flow over the 30-day period with median day, day $t, m^3/s$
$Q_{t,low}^7$	7-day, 20-yr. low flow of day $t, m^3/s$
$Q_{t,low}^{30}$	30-day, 20-yr. low flow of day $t, m^3/s$
$Q_{t,i}$	measurement of river flow at time t , at river site $i, m^3/s$
Q_w	waste discharge flowrate, $m^3/d, m^3/s$
$q_{\tau,y}$	measured average streamflow at a river gauge station in day t of year $y, m^3/s$
R	reach
r_{bod}	rate of benthic oxygen demand, $mg/m^2/d$
r_{BODrem}	<i>overall removal or exertion</i> rate of BOD, $mg/m^3/d$
r_C	the total rate of change in the concentration of the modelled constituents C due to (bio)chemical transformation, $mg/m^3/d$
r_{nitr}	rate of nitrification, $mg/m^3/d$

r_{reaer}	reaeration rate, $\text{mg}/\text{m}^3/\text{d}$
S_i	scale of waste reduction measures at site i
S	salinity, ppt
t	time, s, d
T	temperature, °C, K
T_{ref}	reference temperature, °C
$(t_{1,opt})_i$	starting date of the optimum window for river site i
$(t_{2,opt})_i$	ending date of the optimum window for river site i
v	average velocity, m/s
V_{max}	maximum value of volume, m^3
V_w	water volume of a treatment plant, m^3
X_5	concentration of BOD ₅ , oxygen demand satisfied after five days, mg/m^3 , mg/s

Useful Abbreviations

Aer.	Aerobic
Aver.	Average
Biol.	Biological
<i>bod</i>	benthic oxygen demand
BOD	biochemical oxygen demand
BOD_u	ultimate BOD
BOD₅	5-day BOD at standard laboratory conditions
CBOD	carbonaceous biochemical oxygen demand
CRC	Class, Responsibility and Collaboration

CREAMS	Chemicals Runoff and Erosion from Agricultural Management Systems
DO	Dissolved Oxygen
DSS	Decision Support System
EPA	Environmental Protection Agency
GIS	Geographic Information System
HEC	Hydrologic Engineering Center
HSM	Hydrocomp (Stanford) Simulation Model
HSPF	Hydrologic Simulation Program Fortran
Max.	maximum
Min.	minimum
MOE	Ontario Ministry of Environment
NBOD	Nitrogenous BOD
OMT	Object Modelling Technique
PWQO	Provincial Water Quality Objectives
SCS	Soil Conservation Service
Sec.	Secondary
SOMA	Semantic Object Modelling Approach
SPUR	Simulation of Production and Utilization of Rangelands model
SWMM	Storm Water Management Model
SWRRB	Simulator for Water Resources in Rural Basins
TWDE	Texas Water Development Board
WSC	Water Survey of Canada
WRE	Water Resources Engineers

Greek Symbols

Δ	difference
$\Delta Q_{np}^{A,B}$	flow of nonpoint sources between two river points A and B, m ³ /d, m ³ /s
Δs	distance, m
Δt_{study}	whole study period of interest
$\Delta t_{opt,wType}$	optimum discharge window of type wType, days
Δt	time interval, s
θ	temperature correction factor, -

Subscripts

<i>a</i>	reaeration
<i>av</i>	available
<i>aver</i>	average
<i>b</i>	background
<i>hod</i>	benthic oxygen demand
<i>rem</i>	removal of CBOD
<i>C</i>	name of modelled constituent (<i>L, N or D</i>)
<i>D</i>	drainage
<i>dp</i>	discharge point
<i>f</i>	final
<i>g</i>	gauge
<i>i</i>	river site
<i>jn</i>	junction node
<i>lab</i>	laboratory
<i>low</i>	low flow
<i>m</i>	after mixing
<i>max</i>	maximum
<i>min</i>	minimum
<i>n</i>	ammonia oxidation
<i>nitr</i>	nitrification
<i>np</i>	nonpoint
<i>opt</i>	optimum
<i>r</i>	river, CBOD removal
<i>ref</i>	reference

<i>req</i>	required
<i>s</i>	saturation
<i>study</i>	study period
<i>T</i>	temperature, °C
<i>t</i>	day of measurement or median day
<i>tot</i>	total measured
<i>trib</i>	tributary
<i>u</i>	ultimate
<i>w</i>	waste
<i>wType</i>	window type (min, aver, max)
<i>y</i>	year
<i>0</i>	initial
<i>1, 2</i>	starting, ending day of a time interval
<i>5</i>	5-days
<i>5,u</i>	ultimate 5-days
<i>20</i>	at 20°C

Superscripts

A, B	from river site A to river site B
R	reach
s	individuals or groups of individuals that discharge pollutants into the river
7	7-day period
30	30-day period

Mathematical Symbols

$\frac{d}{dt}$	time derivative
$[t_1, t_2]$	time interval, days

Chapter 1

Introduction

1.1 Water Resources Management

Water resources analysis, planning and management has been a field of scientific and social interest and continuous research within the last decades. The increased rate of changes in water resources systems and the severe problems that have occurred led to a higher concern about their structure and function (Moss, 1988). The significant impact of human activities on them also heightens the need for a sound scientific study of water resources management and the demand to use its results in policy making (Somlyody and van Straten, 1986).

The major objective of water resources management is the determination of *preventative* and *restorative* strategies to satisfy its *multipurpose use*, that is, to ensure:

- (i) the adequate provision of water to all its potential users in a region (*quantity management*) and
- (ii) the appropriate quality of water for all its beneficial uses (*quality management*).

Water resources systems that can be under management include the following components (Krenkel and Novothy, 1980; Biswas, 1981; Nemerow, 1985).

1. *Hydrologic Systems*: They include precipitation, melted snow and glacier water. Their dynamics are related not only to natural phenomena such as geographic and climatic factors but also to human activities that can in the long term affect adversely those factors and be a cause of problems such as drought or acid rain. They are a primary water supply source and also an input for other water systems (Krenkel and Novothy, 1980).

2. *Overland Flow Systems*: They correspond to the excess of precipitation that flows over the soil surface of rural and urban areas towards a receiving water body. The purpose of their management is the protection of productivity in the terrestrial ecosystems, the control of nutrient flow and sediment transfer and deposit to the aquatic ecosystems, and the control of water level and quality in the groundwater (Brooks et al., 1991; Hetrick and Travis, 1988).

3. *Rivers*: Rivers as well as lakes, estuaries and oceans are categories of *surface water*. Rivers and most lakes are also *freshwater* systems, while estuaries and oceans are *saline* systems. Rivers are a basic water supply source for *flow uses*, such as hydroelectric energy production, wastewater dilution, water based recreation, maintaining of fish and plant life. They also provide a major part of the water required for *intake uses* of supply for irrigation, industrial and municipal use (Krenkel and Novothy, 1980). Their *quantity management* usually includes the control of the distribution of water flows in time and amount through reservoirs that can reduce the flood damages, and the allocation of water flows to supply in a more beneficial way the water for intake uses (Biswas, 1976). Their *quality management* is usually achieved with methods that reduce or spatially redistribute the discharged pollutant load and improve the constituent concentration levels of the receiving water (Biswas, 1981). River quality management is the subject on which the present study will be concentrated.

4. *Lakes*: They represent a form of water *in storage* and can be used for the same purposes as rivers except for the production of hydroelectric energy. The major problem that has to be managed is *eutrophication*, which may have destructive effects on water quality and the lake ecosystem and also has a negative impact to the water body that accepts the lake's effluent (Somlyody and van Straten, 1986; Moss 1988).

5. *Estuaries*: They have the same flow uses as rivers but are more complex systems to analyze due to the tidal effects and the low flows that characterize them. They require mainly quality management, and due to the low flows, are often a disposal site for sediments with concentrated absorbed contaminants (O'Connor et al., 1973; Tetra Tech, 1978b).

6. *Oceans*: They can supply water for similar uses with estuaries. Due to their bulk and the buffering properties of water, oceans are the water resource that can accommodate the impact of human activities. So they have been disrupted to a smaller extent than freshwater systems and have experienced gradual changes in their ecosystems (Moss, 1988). However they also require quality management to control the sludge as well as oil and other chemicals disposal that are applied near shore-zones close to urban or industrial centers and in central zones of the ocean (Krenkel and Novothy, 1980).

7. *Groundwater*: It is a source of water for intake uses, similarly to surface water. Its quantity control is focused on developing a safe or alternate yield approach that will preserve the water level of the aquifer (Kisiel and Duckstein in Biswas, 1976). Its quality control has as a purpose to prevent or reduce the aquifer contamination with chemical substances that are introduced during its sampling or are transferred from the used water, and are dispersed in the groundwater body. A detailed study of groundwater systems has to deal with a significant degree of complexity and uncertainty. Major factors responsible for this are the difficulty in measuring the recharge of large aquifers, the complex hydrodynamic patterns that prevail, the possible time lag between a pollutant action (such as irrigation with increased amount of fertilizer) and the groundwater response, and

generally the problem to ascertain the origin of pollutants (Magette et al., 1989).

Groundwater has a direct interaction with surface water with respect to both its quantity and quality. So the management of *conjucent use* of those two water resources would increase the net gained benefits even though it would increase further the complexity of the coupled system. Implementation of this strategy is still limited, however it is expected to increase and is an issue of current research (El-Kadi, 1989).

The unit for planning of water resources system has been broadened from the water resource itself to the region which includes it or to a smaller scale to its *watershed* (Mather, 1984). A *watershed for a surface water system* is strictly defined as a topologically delineated area that is drained by the receiving surface water system, that is, the total land area above some point on the water body that drains past that point (Brooks et al., 1991). However it integrates not only the hydrologic features but also the physical, biological, social, economic and political features of an area that have to be considered in a planning study of this area or of its parts (Brooks et al., 1991).

The rural areas activities such as agricultural cultivation, grazing of livestock, or forestry and the urban areas activities and requirements (industrial operation, sewerage for the population, wastewater treatment) are all related to the use of water. They are also associated with the topography of the land, the soil character, the percentage and distribution of vegetation cover and the percentage of impervious areas in it, as well as with the degree and the methods of rural, urban and technological development and economic growth. The above-mentioned activities together with climatic, geologic and social factors influence the overland flow and the quality of the water resource, in which the watershed drains. They also have an impact on the downstream areas. Thus a water resource management which will not affect adversely other natural resources and ecosystems and will be beneficial in long term has to be coupled with the management of the watershed as a whole (Brooks et al, 1991; Bennett, 1989; Alexander and Harper, 1989).

A major difficulty in the evaluation of alternative management strategies is the quantification and relative weighting of the criteria to be used. These are related to economic and social values, and the impact of the various strategies on them has to be estimated in a short as well as long term basis. Their estimation is assisted by methods of economic and social sciences, but it has a high degree of uncertainty associated with it. An economic efficiency or cost-benefit analysis would require the quantification of benefits, which is rather difficult and is also influenced by political criteria (Lynne, 1988).

Another limitation in the determination of the optimal water quality management strategy comes from the difficulty in specifying an acceptable water quality, which will be the minimum quality that the alternative methods are required to achieve. The appropriateness of the quality depends mainly on the specific use of water but also on the accepted risk that may change with the time passage and the system knowledge as well as on the economic state and resources of the area being studied. The appropriate quality is represented by *standards*, which are specified by scientific institutions and organizations. These are the basis for the establishing of *legislation* measures that help the control of waste discharge in the water resource.

As can be seen, a thorough study of water resources has to synthesize major principles of wide ranging fields (Helweg, 1985). The physical actions that take place can be understood with the aid of natural sciences. The development and application of control methods require use of engineering as well as of economics and other social sciences disciplines. In the above fields, mathematics and computer science are also involved and facilitate the description of the system and the decision making. Finally, legislation can make easier and more respectable the implementation of any practices and regulations that are required for the achievement of management objectives.

1.2 Models and Decision Support Systems in Water Quality Management

Mathematical models describe with mathematical expressions those characteristic features and processes of a real world system that are required for the *survey* or *management* of certain aspects or problems of its structure and functionality (Jorgensen, 1986). Mathematical models of various degrees of abstraction have been widely used in water quality management during the last four decades and have become today an accepted part of the decision making process (Volp and Lambrechts, 1988; Vansteenkiste, 1976).

A categorization of them with respect to their relation to problem solving distinguishes between simulation and optimization models.

Simulation or predictive models estimate quantitatively the *response* of a system to a defined set of *forcing functions* (Jorgensen, 1983). In other words they estimate the *endogenous variables* that characterize the state of the system for defined input functions or variables that are of an *external* nature to it but influence its state. In a simulation model of river water quality, the response is usually quantified in terms of mass or concentration of constituents that are indices of water quality and participate in the cycle of basic elements (*biogeochemical* models). The forcing functions include among others the variation of physical factors and the flow and load pattern of input streams. Thus alternative policy plans or management strategies for pollution control can be indirectly input to the model through the changes in the above patterns, and can be rapidly quantitatively assessed in terms of the system water quality.

The solution of simulation models requires the simultaneous solution of a set of linear or non-linear equations and is achieved usually through formal integration, regression analysis or with the aid of numerical methods such as finite differences, control volumes or finite elements.

The initial trend in ecologic modelling was in the *development* of models that could describe better complex river systems, their ecologic interrelations, the impact of human activities. Thus a plethora of river water quality models have been developed with differences in the degree of linearity, of temporal and spatial aggregation, of their deterministic character, of the explicit interrelation of the system states with the inputs and outputs and of other features. However, gradually the focus shifted towards the *calibration* of those models against the specific river in which they were implemented and towards their *validation* that would allow their implementation in other river systems and their acceptance in decision making (Orlob, 1983).

When there is a large number of alternative management practices that have to be evaluated in order to reach a decision, the number of required simulations becomes large as well, and there is also a possibility of not accounting for an important alternative. In such a case simulation models could be further developed or replaced by *optimization models*. These models attempt to derive an *optimal policy* for given management *objectives*. Their mathematical formulation describes the water resource system itself, and also includes expressions and explicit variables that define the alternative management policies under consideration, the *objective function(s)* and the possible *constraints* in the variables included in each of those functions.

The solution strategy of optimization models has as a goal the identification of an *optimal solution*, i.e., a solution that minimizes or maximizes the value of the single objective or of all of the objectives simultaneously, if there is one or multiple objectives correspondingly. However, optimal solutions generally do not exist in multi-objective problems. In that case the set of decision options may be significantly narrowed through identification of a set of *non-inferior* solutions, that is of solutions in which a decrease in any of the objectives can be achieved only at the cost of increase in at least one of the other objectives. The final *preferred* solution for the problem can be then determined from this set of non-inferior ones using some additional criteria (Haimes et al., 1975).

The solution strategy of optimization models requires the use of constrained optimization algorithms, usually of linear, nonlinear or dynamic programming. The identification of solution(s) exhibits an increased difficulty in multi-objective optimization models which are quite appropriate in water resources systems. However, several solution methodologies have been proposed and implemented in water systems. Those methodologies include the use of a utility function, of indifference functions, the lexicographic, parametric and e-constraint approach, goal programming, goal attainment method, adaptive search method as well as other approaches.

Optimization models may be associated with a less detailed description of a system than simulation models, in order to balance the overall complexity. However, they can give better results when they are developed from a *validated* simulation model.

Both simulation and optimization models require generally a significant quantity of input information for their implementation and also produce a large quantity of results that have to be maintained and interpreted. These features may "reduce" the acceptance and efficient use of models as management tools and stress the need for development of *Decision Support Systems (DSS)* (Guariso and Werthner, 1989; Bugliarello and Gunther, 1974; Koch and Allen, 1986; Arnold and Orlob, 1989; Zukovs et al., 1991; Masciopinto et al., 1991; Nolasco et al., 1989).

A decision support system can be defined as *an integrated assemblage of models, data, interpretive routines, and other relevant information that efficiently processes input data, runs the models, and displays the result in an easy-to-interpret format* (Orlob, 1992). The major components of a DSS are (i) a *data manager*, (ii) a set of *analytical tools* and (iii) a *user interface*.

- (i) The data manager is responsible for the maintenance and organization of input data and output results, and is usually a *database* or a *geographic information system (GIS)*. A database system has the ability to access very large quantities of data

efficiently and safely, and is usually based on the relational, deductive or object-oriented model. A geographic information system has extended capabilities for management of spatial data. These include data capture, input manipulation, transformation, visualization, query analysis, modelling and output (Bonham-Carter, 1994). Thus GIS has also the features of a user interface and is considered by some researchers as the basis for the building of the whole decision support system.

- (ii) The set of analytical tools may include mathematical simulation or optimization models, statistical or other auxiliary routines as well as interpretive routines. The analytical tools may also consist of the knowledge representation, inference engine and reasoning components of *expert systems*. Expert systems, which also consist of a user interface, are part of artificial intelligence developments and have been another promising tool used in water resources management (Barnwell et al., 1989). Expert systems can help the user understand better how complex decision problems might be solved and at the same time explain to the users why one particular decision may be better than another (Jackson, 1990; Pedersen, 1989; Rolston, 1988; Keller, 1987).
- (iii) The (graphical) user interface allows the user to input, edit, display (in tabular, graphical or other form), and comprehend input and output data. It also allows the user to interact, use his knowledge and control the flow of simulation, optimization, interpretation or inferencing process (Suzuki et al., 1989).

1.3 Objectives

The objective of this study is the application of object-oriented analysis in the management of water quality in a river system.

The object-oriented approach is a promising philosophy of systems development in software engineering (Graham, 1994). The major areas of its application include among others the fields of user interfaces, of GIS, of artificial intelligence, of database systems, of simulation and process control. The main benefit of the object-oriented approach is that it provides a natural way to model many real world situations, to understand and manipulate their complexity (Brown, 1991; Nahouraii and Petry, 1991). It is based on partitioning a domain into objects which represent physical and conceptual entities of the real world. When these objects are well defined, they result in reusable components that can be assembled in the development of a larger system or can be easily extended. This reusability can be gained not only at the level of programming but also at the level of analysis and design.

The delivery of the above benefits would be of major importance in the field of river water quality management due to the complexity and the wide range of factors that have to be accounted and described in that field.

In the present work the application of object-oriented approach has as a purpose to identify and define the objects and specify their relationships that will accomplish the water quality management objectives.

The major emphasis from the quality management perspective is in the control of the river system point-source pollution and the improvement of its quality to levels that meet specified standards and prevent the existence of adverse or toxic effects in its ecosystem and for its water users. The methods for the achievement of the above should use but also protect the assimilative capacity of the river water.

The model of objects that will be developed should be able to accomplish the following:

- estimate *design hydrological conditions* for the management study,
- simulate the transport of pollutants in the river system,

- quantify the extent of pollution in the receiving water body as well as the contribution of the point and nonpoint sources in the total load,
- identify the point-sources whose discharges result in violation of effluent or river quality limits and the point sources which are allowed to have additional growth in population equivalents,
- estimate the allowable degree of increased waste production of the latter ones and account for future potential water discharges,
- propose optimum discharge periods of the point sources,
- suggest a waste load allocation scheme for the river or consider changes and upgrades in the treatment facilities.

For the achievement of the above, the following analysis will be applied:

- modified low flow analysis,
- dilution factor relationships,
- relationships of the transformation of compounds to toxic constituents,
- a steady-state, node-reach, dissolved oxygen (DO), biochemical oxygen demand (BOD), total ammonia model,
- algorithms of waste allocation and of temporal discharge schemes.

The corresponding object-oriented analysis will be performed using the Coad/Yourdon methodology (Coad and Yourdon, 1991; 1993; Coad and Nicola, 1993). The analysis will be followed by an object-oriented design and will be implemented using the *object-oriented programming language Smalltalk* (Digitalk, 1992).

The analysis and the implementation will be applied to the real world case of the South Nation River, located in the area east of Ottawa, Ontario, between the Ottawa and the St. Lawrence rivers. The case was selected due to the availability of quality and

quantity measurements of the river and of the treatment plants. These data were gathered during an extended study that was prepared for the South Nation River Conservation Authority (Droste et al., 1992). The results of this study have been also presented in detail. The agreement of our results with the available ones will reinforce the validity of the analysis and of the implementation, and will enable their extension to other quality management aspects and their use in other case studies.

1.4 Thesis Outline

- Chapter 1:** An introduction to the field of water resources management is presented. The application of mathematical models, decision-support systems and object-oriented analysis in water quality management is discussed. The objectives of this work are presented and a thesis outline is given.
- Chapter 2:** The pollution sources of a river system and the methods of its water quality control will be outlined. A short review of models that predict the river water quality will be presented. The principles of optimization models for the control of point-source pollution will be discussed. Finally, a series of mathematical models for the description and control of nonpoint-source pollution will be reviewed.
- Chapter 3:** The basic principles of object-oriented programming will be introduced and explained. The benefits obtained from the implementation of each principle will be explained along with the types of object-oriented languages. The selection of Smalltalk versus C++ for the implementation of the current project will be reasoned through a comparison of the features, the strengths

and the weaknesses of both languages, keeping in mind the special features of the project.

Chapter 4: The principles and mathematical equations of river quality management applied in the present work will be analytically presented. In particular, the estimation of design river flows, optimum discharge windows and travel times will be explained. The toxicity-dilution analysis in the conditions of the point-sources effluent and of the receiving river will be outlined. The objective of the analysis is the compliance of water quality with the toxicity standards. The hydrodynamic model coupled with the water quality model will be described. Finally, the strategies of point-source pollution control that are under consideration will be discussed.

Chapter 5: The object-oriented analysis of river quality management that was developed in the present work will be described. The major components of the analysis will be explained and also schematically represented. Major points of the reasoning for the analysis will be discussed. An implementation of this analysis will be also presented.

Chapter 6: The application of the above to the case study of the South Nation River system will be discussed. The available data, the model parameters, and the results obtained will be presented.

Chapter 7: The conclusions of this thesis for the object-oriented approach developed in river quality management and its application to a case study will be presented. Recommendations for further work will be outlined.

Chapter 2

River Water Quality Management

The pollution sources of a river system and the methods of its water quality control will be outlined. A short review of models that predict the river water quality will be presented. The principles of optimization models for the control of point-source pollution will be discussed. Finally, a series of mathematical models for the description and control of nonpoint-source pollution will be reviewed.

2.1 River Pollution Sources - Methods for Pollution Control

A river system may be considered a collector of pollutants from various point and nonpoint sources, that are discharged from human activities and natural processes, either directly or during the stages of the hydrologic cycle. As represented in Figure 2.1, the pollutant load of a river can result from the following sources (Jorgensen, 1983; Brooks et al., 1991; Mather, 1984).

- (i) Concentrated, point sources such as industrial, sewage, or treatment plant effluent discharge.

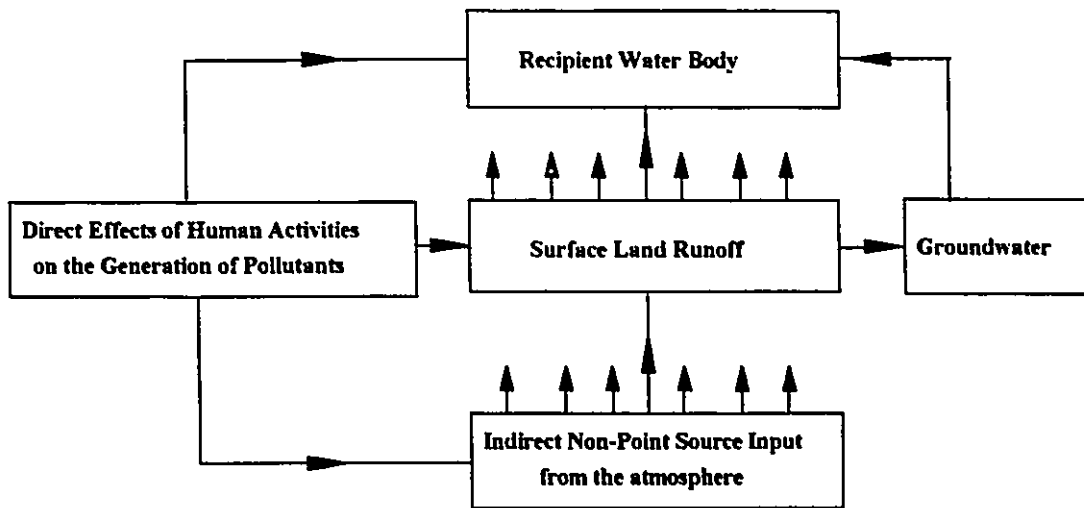


Figure 2.1: Paths of river water pollution (from Jorgensen, 1983).

- (ii) Pollutant fallout through rain, snow or other deposition mechanisms, directly on the river surface.
- (iii) Pollutant transport through surface land runoff or overland flow. This runoff corresponds to the amount of precipitation that reaches the ground and is in excess of the *infiltration capacity* of the soil, that is of the soil capacity to store the water that was moved into it from the surface, with the action of capillary and gravity forces.
- (iv) Pollutant transport by subsurface runoff. This runoff corresponds to the water in soil that is moved either laterally to a river or downward to a groundwater aquifer and then with seepage, similarly to the river.

The control and management of river water quality is directly related to the control of the above interrelated components of the pollutant load. The improvement of water quality in rivers that exhibit water pollution problems can be achieved by a variety of methods that act on these load components and are applied distinctly or more usually in combination.

In particular, the control of *point-source* pollution by organic matter and of the DO concentration can take the following forms (Dorfman et al., 1972; Loucks in Dorfman et al., 1972; Biswas, 1976, 1981; Orlob, 1983; Buras, 1972; Joshi and Modak, 1989).

1. *Reduction of the quantity of discharged waste.* This may be achieved through the appropriate degree of *wastewater treatment* at some pollutant point sources. So construction of treatment facilities or upgrade of existing ones, and suggestion of an optimal operating policy for them may be required.

2. *Waste load allocation in the river basin.* The load of some waste sources can be transported and discharged to other locations where the river has a greater assimilative capacity and the target pollutant and DO concentrations are met.

3. *Artificial aeration.* A quantity of DO that depends on the available oxygen in the river and the deficit from its saturation value can be artificially transferred from the atmosphere through aeration devices.

4. *Flow augmentation.* It can be applied with the release of the appropriate volume of water from storage reservoirs at the specified sites. It can take place during critical periods of low river flows with the purpose to increase the dilution and assimilative capacity at certain river sites. However, it may increase the dilution only if the augmented flow is of higher quality than the base flow, while it may also deteriorate the quality due to higher scour of the benthic deposits (Loucks in Biswas, 1976). Finally, differences in temperature between the added and base flows may have adverse effects on the saturation concentration and mass transfer of oxygen, thus the reaeration of the river or the biota in the river ecosystem are affected.

The control of nonpoint sources is significantly more difficult than the control of point-sources. This is mainly because the size, the composition and the time of appearance (with respect to the time of a causing action) of a nonpoint pollutant load in a river is related to a variety of physical factors and processes of high complexity as well as with the indirect effects of human activities (Brooks et al., 1991). Thus the identification and quantification of nonpoint sources requires accurate monitoring and extensive surveillance of the related processes. Their control has also to be studied in a wider watershed management basis and may require alterations in the land uses in the watershed, in the percentage, distribution and composition of its vegetation cover, in the agricultural and farming practices, in the degree of urbanization, as well as in combinations of the above.

2.2 Predictive River Quality Models

Predictive river quality models can be used to simulate water quality for *projected* point and nonpoint loads, river flows and other model inputs that reflect the above quality control methods. Some of the available models of high degree of complexity can estimate directly the requirements in flow augmentation or in the treated waste water quality in order to reach target values of certain modelled constituents. Similar estimates can be obtained from the simpler models after performing the appropriate simulations. However, these models have to be *verified* and *calibrated* using statistically reliable sets of data. When these predictions are combined with economic considerations they can be used in the evaluation of alternative water quality control strategies.

A major component of river water quality models is the representation of the mass balance of DO in a river. The DO is a *primitive indicator* of river water quality, since it is essential for the maintenance of aerobic conditions that support aquatic life and safe water use (Krenkel and Novothy, 1980). DO utilization is multivariate within the aquatic environment and as a result has a high degree of interaction with other water constituents. These interactions are expressed in river models through the *coupling* of DO equation with the equations describing the cycle or at least the balance of those other constituents in the water.

The first DO-BOD model was developed by Streeter and Phelps in 1925 and is considered now *a classic statement of water quality in streams* (Orlob, 1992). This model describes the variation of DO deficit below saturation (DOD) and of BOD with respect to the distance of travel below a point-source discharge point in a stream of known steady hydraulic characteristics. The DOD variation is assumed to be a result of the DO utilization in the biochemical oxidation of the dissolved organic matter and of the DO transfer from the atmosphere through the air-water interface. BOD variation is assumed

to be a result of the above oxidation process. The system of these two equations can be solved analytically for given boundary conditions.

The Streeter-Phelps model has been the basis for a series of models that were developed between 1950 and 1970 (Jorgensen, 1983, 1986; Orlob, 1983, 1992). These models accounted for additional processes that take part in a river system or proposed alternative mathematical formulations for their description. The models take into account pollution due to point as well as nonpoint sources. The additional processes include among others:

- sedimentation of part of BOD,
- addition of BOD to the overlying water from bottom deposits,
- benthic oxygen demand,
- production of oxygen in photosynthesis and consumption of it in respiration,
- longitudinal mixing effect through dispersion,
- distinct description of carbonaceous and nitrogenous BOD (CBOD and NBOD),
- addition of distributed constituents discharge due to runoff and scour along the course of the river,
- spatial variation of the coefficients in the specific rates of the processes or of the oxygen saturation concentration.

The models integrating the above processes correspond to a system of ordinary differential equations, thus they could be analytically solved, making the appropriate assumptions and specifying the required boundary conditions.

The application of computers and of numerical analysis methods allowed the solution of models that include partial differential equations and represented a high degree

of temporal and spatial detail in the hydraulic characteristics of the river system, the load of the pollutants, the mathematical expressions of the processes and finally in the predicted concentrations of the modelled constituents (Kawashima et al., 1989; Crockett et al., 1989a; 1989b; Jobson, 1985). These DO-BOD models could be also extended to the description of other important indicators of water quality. Thus these models could be extended to describe the cycle and the relationships between other *conservative* and *non-conservative* constants, such as inorganic nutrients that are responsible for eutrophication, of living organisms-parts of the trophic chain (for example algae and zooplankton) or of toxic materials. A few representative examples of these high-complexity *computerized models* are presented in the following paragraphs.

A series of models was developed by Thomann, O'Connor and Di Toro (O'Connor et al., 1973) for a detailed description of the nitrification process and its effects on the DO concentration. The models include simulation of the variation in phytoplankton population, nutrients and DO concentration, in river and estuarine systems. The finite difference approach has been followed, in which the water body is represented by a multisegment system and can have spatially variable properties and multiple pollutant load along its course. These models have been used for studies of the San Joaquin River in California, and the Delaware and Potomac estuaries (O'Connor et al., 1973).

The DOSAG-I model was developed by the Texas Water Development Board (TWDB) in 1970 and simulates the steady-state CBOD, NBOD and DO variation in a system of a main river and its tributaries under various conditions of flow and temperature. It also computes the flow augmentation required to bring the DO concentration to a specified target level. In the model, the river system is represented by a sequence of reaches and nodes and can accept waste discharge or allow water withdrawal at any of its points. The coefficients in the description of biochemical processes can also vary between

reaches. The model has been applied in studies of the San Antonio River (Texas, USA) and of the Holston River (Tennessee, USA) (Jorgensen, 1983).

The DOSAG-I was modified by Armstrong at the University of Texas (Austin) in 1977 (Jorgensen, 1983). The resulting model, named DOSAGM, describes additional processes, has improved rate coefficients, and also has provisions to reduce the waste loading from point sources by the application of a user-defined treatment factor. Representative applications are the Paraiba River Basin in Brasil (Jorgensen, 1983) and the Holston River in the US (Krenkel and Novothy, 1980).

QUAL I was developed by Masch and Associates in collaboration with TWDB in 1970 (Orlob, 1983). The model simulates the temporal and spatial DO, BOD, conservative minerals and temperature variation, in a one-dimensional, completely mixed branching river system. The system is represented by a series of control volumes with different cross sections and lengths. Waste discharge into these volumes or water withdrawal from them is allowed. The transport of constituents and heat through them is made with advective and dispersive mechanisms. The variation of temperature is accounted for through the heat energy exchange across the air-water interface of the control volume. The simulation of temperature permits the adjustment of the temperature-dependent parameters of the model during unsteady-state simulations.

A modification of the QUAL I model, named QUAL II, was developed by Water Resources Engineers, Inc. (WRE) for the US Environmental Protection Agency (EPA) in 1973 (Orlob, 1983). The model describes the variation of benthic oxygen demand, chlorophyll a, nitrogen forms of ammonia, nitrite, and nitrate and phosphorus, the interactions of those constituents and their impact on the DO-BOD relationship, finally the variation of temperature, coliforms, radioactive material and conservative minerals. QUAL II can be applicable to more complex river systems than QUAL I.

QUAL II was further modified and become available through the EPA Center for Water Quality Modelling (CWQM) as QUAL 2E in 1985. This model was extended to

QUAL 2E UNCAS by EPA and Tufts University in 1987, and can account for uncertainty providing options for sensitivity analysis, first-order error analysis and Monte Carlo simulation.

QUAL I and its subsequent developments are some of the most widely used models. QUAL I was applied to the San Antonio River Basin in Texas, to the Plantation Canal in South Florida, to the Lower Mississippi River; QUAL II to Truckee Carson River System in northern California and Nevada, to the Utah Lake-Jordan River Basin; QUAL 2E-UNCAS to the Santa Ana River in California (Orlob, 1992).

Several other modifications of the above models have been also performed with the purpose to overcome numerical instabilities, eliminate or at least reduce certain of their limitations, and generally improve the applicability and accuracy of the models.

The previously described models are all *deterministic models*. However the complex geophysical and (bio)chemical processes of water and constituents transport, storage and transformation that take place in a river basin, exhibit random fluctuations from year to year as well as periodic and stochastic fluctuations within the same year (Salas et al., 1980). Thus it becomes evident that a *stochastic analysis* may be more appropriate, especially in a management study. A series of stochastic river quality models have been developed (Leduc et al., 1986; Dewey, 1984). These represent usually the major DO-BOD interactions without accounting for the other constituents due to the increased degree of difficulty in their solution and the excessive amount of required data. It should be noted that the calibration of these models requires the comparison and agreement not only of the estimated concentration values with the mean measured data but also of their probability distributions.

Finally, a point worth mentioning is the lack of *identifiability* in predictive river quality and generally environmental modelling (Beck and Halfon, 1991). The identifiability of a model refers to the ability to determine a uniquely best set of estimates for a model's parameters. This ability is achieved when we account for the existence and

propagation of uncertainty that is associated with the structure and the hypotheses in a model, thus with our assumptions for the mathematically described knowledge of a river system behaviour. *Filtering theory* provides the conceptual framework for such an analysis and for the estimation of the propagated errors in the predictions of a river quality model (Beck and Halfon, 1991).

2.3. Principles of Optimization Models for Point-Source Pollution Control

The purpose of optimization models of river quality is to find an optimal scheme of control strategies applicable in a river basin that can ensure an acceptable quality in the river system and can also satisfy certain objectives.

A major objective is the minimization of the total cost (C) that is required for the application of a set of control methods considered with scale S from all individuals or groups (S) that discharge pollutants at various sites (i) in the river system, i.e..

$$\min \left(\sum_i \sum_s C_i^s(S_i) \right) \quad (2.1)$$

To account for the fact that the above cost can be estimated under a constraint of a water quality higher than the minimum allowable and also to provide those who are concerned with waste discharge a motivation to improve the effluent quality, the cost in the objective function is increased with the addition of another component. The extra component corresponds to taxes or subsidies, which are specified on the pollutant quantity that has remained after the reduction measures and is discharged with each effluent. Such a policy defines a balance between the increased cost that would be related with a better wastewater treatment and the decreased cost for taxes that would result from the better

effluent water quality (Loucks in Orlob, 1983).

The variables included in the objective function have to satisfy certain constraints, which define the range of their acceptable values and also determine the solution to the optimization problem. In water quality optimization models, a primitive part of the constraints has as a purpose to ensure that water quality standards are satisfied. Thus they specify the maximum quantity of one or more pollutants that is allowed to be discharged at a river site. Alternatively, constraints require that the water quality should be higher than a specified minimum allowable quality (Loucks in Biswas, 1981).

When applying the above principles in the control strategies described, the following issues should be taken into consideration:

(i) An optimization model for the multiple point-source waste reduction in a basin has as its purpose to find a *degree of removal* for each constituent of interest and for each treatment plant that will minimize the sum of the cost of treatment over all plants and will also satisfy the effluent standards of each plant and the river standards at each river site for all the constituents under study. It should be noted that due to the high interaction of constituents, there is an interdependence in their efficiencies that can be mathematically described and used to simplify the solution of the model (Loucks in Biswas, 1976).

(ii) A model for an optimum waste load allocation in a river system has as its objective function the minimization of the overall *pipng* and *pumping cost* that is related to the transport of waste from all required current sites to other appropriate sites in the whole basin. The total cost for each allocation is a function of the quantity of flow that has to be transported. The minimization of the total cost over the basin is subject to the river water quality limitations that have to be met at all new discharge sites. When the waste that has to be allocated is the effluent of a treatment plant, the effluent quality limitations also have to be satisfied (Loucks in Orlob, 1983).

(iii) An optimization instream aeration model for a river system has to identify a combination of aerators that will operate along the course of the river system and whose number and power capacity will minimize the total associated cost. The location of each one of those aerators should be such that river water quality (especially DO standards) are satisfied prior to the application of aeration at the site, but with aeration applied at all defined upstream sites. The rate of artificial aeration at a specified location is mainly a function of the oxygen deficit at this location and of the power capacity of the corresponding aerator (Loucks in Orlob, 1983).

(iv) Finally, an optimization model of flow augmentation has to find an optimal scheme of increasing the river flow at appropriate sites, with the appropriate augmentation different for each site dependent on the quantity of water released from reservoirs. The objective in the identification of this scheme is the minimization of the net cost that is related to the use of reservoirs for quality management, and additional to their cost for flow regulation in quantity management (School of Civil and Environmental Engineering, Cornell University and IIASA, 1990). The constraints in the solution are imposed by the water quality standards at the river locations of reservoirs that have to be satisfied. It should be noted that the quality at these locations should be estimated based on the augmented flow and its effects on the rate of chemical transformations (Loucks in Biswas, 1976).

2.4 Models for Nonpoint River Pollution Control

Models of river quality or of point source pollution control are mainly *theoretical*. This means that they describe the related processes using physical laws and theoretical principles. On the other hand models for nonpoint source pollution control, can be either theoretical or *empirical*. Such models are usually based on *observed* input-output (named also disturbance-response) relationships without attempting to simulate the processes. The selection of the appropriate type of model depends on the available data of the river basin under study and on the source of nonpoint pollution load.

The pollutant load from the direct deposition of rain or snow on the river surface can be estimated from *input* or *pollutant generation* models. Such models are usually regression relationships between the concentration of an ion of interest in fallout water and the quantity of rain and the dry deposition rate.

The load transported by the subsurface flow is usually estimated with simple empirical relations of *groundwater recharge* which are included in river quality models. Two- or three-dimensional numerical models of groundwater quality have been also developed (Duffy and Lee, 1992). However due to the different time scales in the transport of pollutants in the ground and surface water and in the time lag in the response of a river to the introduction of a pollutant, there is a difficulty in interfacing these numerical models with a river quality model. As a result the theoretical groundwater models are used mainly in groundwater quality control and only to a small extent for river quality control (Biswas, 1976; Thierry, 1989).

The pollutant load transferred with surface runoff can be estimated using *specific area nutrient yield* values (Jorgensen, 1983) along with single and multiple *regression relationships* between the loading rate and factors which affect it. Semi-empirical and theoretical models can also be applied. Those models can be used for water quality control and can also be tools for integrated watershed management support systems. Due

to this reason a more detailed presentation of them will be attempted.

The specific area yield value represents the mass of pollutant resulting from a unit of area in the watershed due to surface runoff in a time period. These values are highly dependent on the land use, the features of the watershed and the type of the pollutant. If they are used for a watershed different from the one for which they were estimated they can give only a gross approximation of the range of expected pollutant yield.

The regression models that have been developed usually express the surface runoff yield of pollutants as a single function of the percentage of cultivated area. More detailed models include distributions of precipitation magnitude, the density of arable land, the fertilizer use, the population density, the ratio of population supported by sewerage and other natural, regional and management conditions (Jorgensen, 1983). Another type of regression model relates the yield rate of area pollutant (or the concentration of the pollutant or the loading rate in the recipient river) with the rate of flow of the recipient watercourse. In order to generalize these relationships, the regression coefficients can be expressed in terms of indices of the natural, regional and management conditions as described above (Jorgensen, 1983; Haith and Shoemaker, 1987).

Semi-empirical relations have been also developed between the amounts of *rainfall* and *runoff* (Loague, 1991; Rankl, 1990). A representative example is the *Soil Conservation Service* (SCS) equation. The rainfall-runoff relation can be also estimated with the *unit hydrograph* method (Bhaskar, 1988; Nix and Tsay, 1988; Hjelmfelt, 1986).

Alternatively it can be quantified with *stochastic (time-series)* models when the extensive amount of required data is available (Salas et al., 1980; Box and Jenkins, 1976). These models relate a runoff at a particular time step with runoff and rainfalls at a *number* of previous time steps. The number of steps defines the order of the model. First- (Markov) or second-order models are the most frequently used and they can simulate daily, monthly or annual runoff fluctuations (Chang et al., 1987; Henderson, 1989; Cover and Unny, 1986; Wang et al., 1991).

The loading rate can be then calculated from the estimated volume of runoff water and literature values of the concentration of pollutants in the runoff. The loading rate can be also estimated using a *unit mass-response function* which can be defined similarly to the unit hydrograph, based on the analogy of the pollutant runoff to the water runoff.

In addition to the above types of models, a wide variety of conceptual models have been developed that simulate the movement, storage and transformation of water in the hydrologic cycle and study the resulting runoff on a quantitative and qualitative basis (Brooks et al., 1991; Chiew and McMahon, 1991; Choudhury and Blanchard, 1983). These models represent the mass balance principle but include experimental considerations as well. Representative models of this type are given below.

(i) A *continuous hydrologic* model named *Hydrocomp (Stanford) Simulation Model (HSM)* was developed by Crawford and Linsley in 1964 (Jorgensen, 1983). The model can synthesize continuous hydrographs of hourly or daily streamflows for watersheds of varying sizes and character, and has been widely used in watershed management studies.

(ii) An extension of this model to include quality simulation was developed by Hydrocomp for the USEPA in 1980 (DeCoursey, 1985). The resulting model - *Hydrologic Simulation Program Fortran (HSPF)* - can simulate the movement of DO, organic matter, temperature, pesticides, nutrients, salts, bacteria, sediment, pH and plankton, from the land surface through a channel and reservoir system including groundwater. HSPF has been the most extensively used general hydrologic model, even though it requires several years of record for the estimation of its empirical parameters (Ng and Marsalek, 1989).

(iii) A model for the simulation of the response to nonpoint pollution in field-size areas was developed by Knisel et al. in 1980 (DeCoursey, 1985). The model, named *Chemicals Runoff and Erosion from Agricultural Management Systems (CREAMS)*, can simulate tillage, crop growth, erosion, sediment yield, plant nutrient and pesticide movement. It has been applied to a large number of field-scale agricultural sites to evaluate alternative

land use practices and observation measures.

(iv) A modification of CREAMS called *Simulator for Water Resources in Rural Basins* (SWRRB) has been developed (Williams et al., 1985). The purpose of this code is to predict the effects of management decisions in water and sediment yield in rural areas. The model gives a long-term simulation of hydrology processes, weather features and sediment yield in subdivided large, complex or small basins with mixed land use, numerous control structures and varied soils. The model estimates have a reasonable accuracy even for ungaged basins (Williams et al., 1985).

(v) SWRRB has been the basis for the hydrology component of *Simulation of Production and Utilization of Rangelands* model (SPUR), that was developed by Wight et al. in 1987 (Wilcox et al., 1989). It includes improved description and facilitates treatment of spatial variability of climatic, hydrologic, plant, animal and economic factors and processes.

(vi) A model for the evaluation of different management schemes on soil loss and water quality in agricultural areas was developed by Yoo and Molnau (1987). The model interfaces the USDHAL-74 Watershed Hydrology Model with a soil erosion model, and is appropriate for areas with main features the sediment and related pollutants generation and transport.

(vii) A continuous simulation model for regional stormwater management quality analysis, named QUALHYMO, was developed by Rowney (1985). The model simulates the rainfall-runoff process and the routing of water through control structures and through the receiving water body. The model is more suitable for large watersheds of mixed urban and rural composition.

A variety of models for urban runoff and pollution control have been also developed (Maksimovic and Radojkovic, 1986). A few representative models of this type are described below.

(i) A *single-event* model that simulates the rainfall-runoff process, named HEC, was developed by the Hydrologic Engineering Center in 1981. The model simulates the outflow hydrograph of the basin or of its subbasins and the routing of flow (both in the watershed and in the sewer channel), and has been used in studies of the urbanization effects in a watershed (Garcia and James, 1988). The model has been also interfaced with an AUTO-CAD-based watershed information system (Cline et al., 1989).

(ii) One of the most widely used models that includes quantity and quality simulation is *Storm Water Management Model* (SWMM) (Gore and Storrie, 1980). The model was developed for EPA by Metcalf and Eddy (1971). Its purpose is to study the pollutant potential in urban watersheds and to compare alternative strategies for the management of storm water runoff and combined sewer overflows. The comparison of these strategies is made in terms of the relative cost and the effects on the receiving water body, which may be a river, a lake or an estuary.

(iii) A decision support system for the planning and evaluation of municipal pollution control services, named PCP-Advisor, was developed by CH2M HILL ENGINEERING LTD. and Hydromantics Inc. in 1993 (Patry, 1990; Zukovs et al., 1991; Gall et al., 1993; Anderson et al., 1993). The system has a wide range of diagnostic and simulation tools for the assessment of sewage collection and wastewater treatment systems and of the receiving water body. The system has been applied to the city of Port Colborne.

Chapter 3

Object-Oriented Programming

The basic principles of object-oriented programming will be introduced and explained. The benefits obtained from the implementation of each principle will be explained along with the types of object-oriented languages. The selection of Smalltalk versus C++ for the implementation of the current project will be reasoned through a comparison of the features, strengths and weaknesses of both languages, keeping in mind the special features of the project.

3.1 Principles of Object-Oriented Programming

Object-Oriented Methodology is a whole philosophy in system analysis and design in software engineering and represents a way of *abstracting* and organizing our knowledge about the real world, using a powerful *metaphor* of *encapsulating* the knowledge into *objects* (Graham, 1994). This is a natural approach appropriate for *simulation modelling*, since the flow of actions in computation reflects the structure of the real problem (Budd, 1991). It is a *paradigm* that allows for the design and implementation of reusable and extensible components that can be assembled or used in various systems. It has been

influenced by principles of data modelling and artificial intelligence theories and has given rise to several methods (Graham, 1994). Object-oriented approach is a field of continuous development and research interest and has been applied to a growing number of commercial systems (Graham, 1994).

The primitive elements of structuring a system in an *object-oriented methodology* are the objects. These can represent physical entities of the real world (like a river or a forest) or concepts that form an abstraction (e.g., a data file). They can be a single entity or a group of them with similar features. Objects are defined by their *features* and their *responsibilities*. The *features* (or *data* or *attributes*) represent what an object *knows* and determine its *state*. The *responsibilities* (or *operations* or *methods* or *services*) represent what an object can *do* and allow it to manipulate its data and modify its state.

The principles of Object-Oriented Approach (OOA) are the ones of *encapsulation*, *information hiding*, *message passing*, *inheritance* and *polymorphism*.

3.1.1 Encapsulation

Encapsulation refers to the inclusion in a single entity (object) of both information and operations that are associated with it. This grouping of knowledge aids in the conceptualization of the system and allows the reuse of entities with specified characteristics in other parts of the same or of another system.

3.1.2 Information Hiding

It is based on the distinction of an object's knowledge in two parts: its *public interface* and its *private representation* by other objects. The public interface consists of

what is *visible* to other objects during their communication and necessary for the performance of their responsibilities for the correct functionality of the whole system. In particular, the interface consists of the attributes of an object whose value can be available and of the services that can be accomplished if *requested* by other objects. However the actual operations performed for the *implementation* of those services are only accessible within the object and consist of its private or *internal* structure.

This feature reduces the *coupling* between separate entities and enhances the need and effectiveness of the design of simple and specific interfaces. Information hiding offers a more abstract and global view of the system. It also aids in the design of easily *maintained, modified* and *extended* software, and has been proven extremely beneficial in complicated systems. An object may apply or test a modified method for the implementation of a service without requiring any changes in the structure of its interacting objects, as long as the interface of the former remains the same. Possible errors introduced by this alteration would be also localized and could be easily corrected (Wirfs-Brock et al., 1990).

3.1.3 Message Passing

Message passing is the only way of communication between objects. Any object has its own responsibilities and is not allowed to have direct access to the properties or behaviour of another object. So when it needs at one step of fulfilling its own responsibilities some information or service that is semantically related with another object, it has to send it a message requesting it. This message consists of a specific name and a possible list of arguments and will be understood by the receiver object only if the latter includes in its behaviour a method with the same name (*selector* of the method). In that case the method will be executed applying all the appropriate steps (internal ones or

message passing to other objects) and it may also return some value to the sender object. The name of the method with the type of its parameters and of the return value is the *signature* of the method. An exact definition of this signature can be of significant help in the design.

3.1.4 Classes and Instances - Object Identity

As noted earlier, objects with similar characteristics are collected and define a new object which is called a *class*. Any object has to be a *member* of a class and is called an *instance* of it. A class has its own attributes which are shared (become known) by all its instances and are called *class variables*. A class has its own operations which can perform on those variables and are called *class methods*. One of the most important responsibilities of a class, common to most classes in any system, is the creation of a new instance of it. A class defines the general features and behaviour that all its instances must have. These characteristics are called *instance variables* and *instance methods*, correspondingly. However the values of those attributes and the result of a method are *unique* for each instance. This reflects that each object has its *unique own identity* that possesses from the moment of its creation through its entire lifetime. This identity may be stored as the value of an ID attribute with which an instance may be distinguished in a collection of instances of the same class. The above practice is extensively used in data management systems (Coad and Yourdon, 1993).

3.1.5 Inheritance-Class Hierarchy

It is a mechanism that allows all properties and operations associated with an

object to be applicable by another object that represents a *distinct abstraction* but which *includes* this knowledge in its definition. It is implemented in terms of classes and actually allows the characteristics of a *generalized* class (and of its instances) to be inherited by any *specialized* class (and its instances), so that the latter will not need to redefine them. In that way inheritance offers a way of *structuring* a basic part of any system by creating *class hierarchies* (Budd, 1991; Booch, 1994). The commonalties between different classes are exploited and are encapsulated in a class which is set at the top of the *hierarchy*. A class whose any instance *is* what the highest class represents but has *extended* its knowledge and generally has some *differences* in what it *knows* and *does*, is set at a lower level. A class which is one level higher than the others is a *superclass* for any one of them and has them as *subclasses* (Masini et al., 1990).

The highest class in a hierarchy may be a *concrete* or *abstract* class. A concrete class can *create its own instances* that can participate in an application and has an exact *specification* of how to *implement* all its services. An abstract class *cannot create its own instances* so it is not directly involved in a system. However, it defines a behaviour that will be shared by all its subclasses. It can also specify the implementation of a part or all of its operations or may declare this as a responsibility of each subclass (Wirfs-Brock et al., 1990).

A class may inherit some of its characteristics from superclasses in the same or in distinct hierarchies. This corresponds to the distinction of *single* from *multiple* inheritance. Multiple inheritance promises a more natural representation but its application has not been completely determined and it is not supported by all languages (Masini et al., 1990). A difficulty arises when superclasses from different hierarchies perform an operation with the same name but apply a different internal algorithm. The algorithm applied in this method when inherited by its subclasses will then have to be determined. Special methods of *resolution conflict* including *renaming* and *redefining* of the method in

the superclass have been proposed. Generally this issue is a subject of current research (Graham, 1994; Singh, 1994).

Implementation of inheritance in structuring a system delivers *reusability of code* and facilitates the *extendibility* of the system. The code that corresponds to the definition of attributes and methods in a superclass should not be rewritten when defining any of its subclasses. The addition of new functionality encapsulated in a distinct subclass requires coding only of this new part.

Modification and maintenance of the system are also supported resulting in a more reliable software. Changes in a method have to be applied to the class at the top of the hierarchy and inherited by all its subclasses. Similarly, errors in the inherited behaviour of a class would have to be corrected once within the behaviour of the highest class in the hierarchy (Wirfs-Brock et al., 1990; Budd 1991).

A consistency in designing a classification structure is also achieved. A partially improved understanding of communication of its objects with other objects is gained as well. These are due to the fact that the public interface of the subclasses in the hierarchy will include the same inherited public interface of their common superclass.

In order to gain the above benefits of inheritance the classification structure should be carefully designed. The general class should have a wide functionality that would allow for potential refinement. At the same time this functionality should be narrow enough to exclude semantically irrelevant knowledge as well as knowledge that should be apportioned in other hierarchies.

3.1.6 Overriding Polymorphism - Static and Dynamic Binding

Polymorphism is an action allowing the definition inside a subclass of a method with the *same* name with an inherited one but with different *meaning* of it. This

redefinition accounts for the distinct way with which individual objects of the same kind, thus instances of different subclasses with a common superclass, respond to the same message. As a result, it offers a better representation of the actual system.

Overriding can be applied as *replacement* or as *refinement*, when the subclass method applies correspondingly to a totally different sequence of operations from the one of inherited method or uses the latter as a part of its own algorithm.

It is actually a type of *polymorphism* as implemented in object-oriented languages. Polymorphism is the ability of a variable or a function to take different forms during the course of execution and is supported in both conventional and object-oriented languages. It is distinguished in *pure* or *universal* and *ad hoc* polymorphism. Pure polymorphism refers to using a single function with arguments of various types while ad-hoc polymorphism refers to using the same symbol (name) for different functions.

Polymorphism can be implemented using two techniques which differ with respect to the *binding time*, i.e., the time at which a name or an expression are associated with the type of value they can hold. This association can be performed at compile time and is called *static* or *early binding*, or can be performed at run time and is called *late* or *dynamic binding*. In object-oriented applications binding refers to the allocation of memory to objects and determination of an object's class.

3.1.7 Whole - Part Structure

The whole-part structure represents the relationship between an object (whole) that is *composed of* or *contains* other objects (parts). These may be of various kinds but are semantically different from the whole object which means that they have different attributes and behaviour from it. Consequently they may belong in one or more distinct hierarchies, which are also distinct from the whole object hierarchy.

Every whole object has to know about its parts, while a part may or may not know about the whole. A composite object will further communicate and cooperate with its part for the achievement of a goal. On the contrary a container object needs only the identity of its elements.

Whole part structures are identified when a "has a" or equivalently an "is part of" relationship can be distinguished among objects. Those structures, when combined with classification structures, result in the representation of a major part of any system.

3.2 Object-Oriented Languages - Selection of Smalltalk

Many object-oriented languages have been developed and can be grouped in two categories: *pure* and *hybrid*. Pure object-oriented languages are based on and support only the principles of object-orientation. Typical examples of them are Smalltalk and Eiffel (Clavel and Veillon, 1991). Hybrid languages combine features of both Structured and Object-Oriented Methodology, and are extensions or modifications of existing conventional or logic programming languages. Some of these developments are C++, Objective C, Object Pascal, and COBOL (Common LISP Object System, CLOS) (Budd, 1991; Graham, 1994).

Languages of both types have been successfully used in large commercial projects, and some of them are available in various platforms. Each type is superior in certain points and has weaknesses in others. The selection of a language to be implemented in a project depends on the features of the project and the strength of the language to facilitate the accomplishment of its goal. The languages considered for the current research were Smalltalk and C++. Selection of Smalltalk was due to the following reasons:

(i) It offers a clear way for implementing an object-oriented design. Smalltalk has a conceptual uniformity due to its principle to define every object as an instance of a class. Even a class is an instance of a *metaclass*. The concept of metaclass may seem complex but it is isolated by the programmer and was introduced in the language to maintain the consistency in the system definition (Goldberg and Robson, 1989). Smalltalk is widely considered as the purest representation of the object-oriented idea and is ideal as a pedagogic tool.

In the contrast, C++ allows definition in the same code of both objects and intrinsic data types. Coding in a way easily understood by others that also keeps its distinct features separate requires a good knowledge of C and C++ itself. Besides, the language is usually used as C with its object-oriented features ignored or in some cases misused (Graham, 1994).

The subject of our application by its nature offers a large potential for extension. Thus coding is required in a way understood by others, in a language that uses as much as possible the benefits of extendibility and reusability of object-orientation.

(ii) Smalltalk has an excellent *programming environment* that allows *rapid prototyping*. A class browser shows all classes and allows access to their code, thus facilitating code reusability (Lalonde and Pugh, 1990). Class definitions can be easily modified and single expressions or methods can be checked so that the application will be incrementally developed. Any syntactic errors are identified by a compiler while defining new objects. Since Smalltalk applies *dynamic binding*, any semantic errors in objects definition and communication occur in run time. An excellent *interactive debugger* provides with their identification and allows for their correction and continuation of the application (Shafer, 1993; Digitalk, 1988; 1992).

C++ offers a basic environment. By default it applies binding at compile time. Therefore it has the potential to execute the code faster but it also requires a much longer

development time (Graham, 1994).

In watershed management there are many subsystems that have to be described and many aspects to be covered. A language that allows rapid prototyping is preferable to a language that could give faster results, supposing that the programmer is able to implement the capabilities of the latter.

(iii) Smalltalk applies *automatic memory management*. A garbage collection collects all objects that were defined during execution but are no longer used, i.e., those that are not referenced by the others, and frees the memory allocated to them. This facility requires a part of the system memory but it is widely considered as a requirement of object-oriented languages because it offers a reliable code and increases the available development time. In languages like C++, the programmer must keep track of all objects and deallocate the unused ones. This practice usually results in memory leaks, as memory allocated to some unused objects is not freed, or in errors due to message passing to objects that have been destructed. The right memory allocation and the identification and correction of errors requires considerable programming effort and consumes valuable time spent in repetitive debugging (Graham, 1994).

In the present project we wanted to concentrate in the description of our application that had a considerable degree of complexity, so saving of debugging time that was offered by Smalltalk was a real benefit.

(iv) Both Smalltalk and C++ have rich libraries of text, mathematical, Boolean, graphics and interface objects, but Smalltalk as noted above offers a global view and access to them directly through its environment.

3.3 Object-Oriented Methods

A variety of object-oriented analysis and design methods have been proposed for the description of a system (Graham, 1994). Methods can be distinguished with respect to their notation in *ternary* and *unary* ones (Graham, 1994).

Ternary methods use three separate notations for the description of the system in terms of *data* (entities), *dynamics* (transition between finite states) and *process* (functions or activities) and are closer to structured methods. Of those methods, the clearest and most complete approaches are considered to be the *Object Modelling Technique* (OMT), the *modified Ptech of Martin and Odell*, and the *Object-Oriented Systems Analysis* methods.

Unary methods use one single notation for the full description of any object and are more consistent with the principle of encapsulation of object-orientation. Of those the best known are the use of *Class, Responsibility and Collaboration* (CRC) cards (a *responsibility-driven* approach described in Wirfs-Brock et al., 1990) and the *Coad/Yourdon* method (Coad and Yourdon, 1993).

There are also hybrid methods which combine features of principles and notations of both types. Representative examples are the *Henderson-Sellers* and the *Semantic Object Modelling Approach* (SOMA) (Graham, 1994).

It should be mentioned that the proposed notations of methods of the same or of different type are generally quite distinct. They correspond to different degrees of complexity, detail in representation and ability of dealing with multiple inheritance hierarchies, real-time systems, or expression of rules included in the system.

Chapter 4

Mathematical Analysis of a Watershed

The principles and mathematical equations of river quality management applied in the present work will be analytically presented. In particular, the estimation of design river flows, optimum discharge windows and travel times will be explained. The toxicity-dilution analysis in the conditions of the point-sources effluent and of the receiving river will be outlined. The objective of the analysis is the compliance of water quality with the toxicity standards. The hydrodynamic model coupled with the water quality model will be described. Finally, the strategies of point-source pollution control that are under consideration will be discussed.

4.1 Design River Flows - Optimum Discharge Windows

4.1.1. Relation of Low Flows with Water Quality - Daily $Q_{t,low}^7$ and $Q_{t,low}^{30}$ Flows

An efficient management or planning study of river water quality has to account

for the occurrence of severe conditions in the factors that influence water quality (Dorfman et al., 1972). The magnitude of river flows is a controlling factor in the ability of a river to absorb waste materials without affecting adversely its ecosystem thus losing its recreational and aesthetic value. Low flows reduce the discharged waste dilution and result in higher concentrations of biodegradable, nutrient or toxic materials. These materials impose a higher DO demand, may result in algae blooms and in anaerobic conditions and generally reduce the appropriateness of water for its multipurpose use (Moss, 1988). Low flows also correspond to low velocities, which concentrate the waste near its discharge point and delay its transport in downstream reaches where the flows are higher and could increase its dilution. Thus the adequacy of streamflows to supply the requirements for the water uses is commonly evaluated in terms of *low flow characteristics* (Riggs, 1972).

On the other hand, a management study based on extreme low flows may result in very conservative estimates of the expected water quality or of the required measures for its improvement to acceptable levels. This is why studies are in practice based on a *critical hydrologic condition* of specified duration and recurrence interval (Dorfman et al., 1972). The maintenance of a minimum water quality under this severe condition ensures the appropriateness of quality for all flows that were higher than this critical flow.

The *critical flow* can be defined in a variety of ways. In this study it has been estimated on a daily basis for durations of seven and thirty days and a recurrence interval of 20 years.

If the variable $q_{t,y}$ represents the measured average streamflow at a river gauge station in day t of year y , then the average daily flow over the 7-day or 30-day period with median day t can be denoted as $Q_{t,y}^7$ and $Q_{t,y}^{30}$ correspondingly, and can be calculated from the equations:

$$Q_{t,y}^7 = \frac{1}{7} \sum_{\tau=t-3}^{t+3} q_{\tau,y} \quad (4.1)$$

$$Q_{t,y}^{30} = \frac{1}{30} \sum_{\tau=t-15}^{t+14} q_{\tau,y} \quad (4.2)$$

The value of $Q_{t,y}^7$ and $Q_{t,y}^{30}$ for the same day t can be calculated for all years using available historical data. These values are *a sample* and can be checked for randomness, homogeneity, stationarity and independence (Salas et al., 1980; Box and Jenkins, 1976). If they satisfy the above principles they can be expressed in a *cumulative probability distribution*. The particular average seven (or thirty) day flow of median day t , that fails to be exceeded or equaled only once within the 20 years, represents the 7-day (or 30-day), 20-yr. low flow of day t and can be expressed as $Q_{t,low}^7$ or $Q_{t,low}^{30}$. In other words, the consecutive average 7-day flow will be less than $Q_{t,low}^7$ at intervals averaging 20 years in length, and the probability that the average 7-day flow around day t in one year will be less than $Q_{t,low}^7$ is 0.05 (Riggs, 1968), i.e.

$$P(Q_{t,y}^7 < Q_{t,low}^7) = (1/20) \quad (4.3)$$

The cumulative probability distribution that can be fit to the $Q_{t,y}^7$ and $Q_{t,y}^{30}$ flows over the years of measured flows is the *asymptotic type III extreme-value distribution* (Gumbel, 1967). This distribution is applied in low flow and generally drought analysis. It has three characteristic parameters: (i) a *lower limit*, which may be zero or a finite value greater than zero, (ii) a *characteristic drought*, which has a recurrence interval of 1.58 years, and (iii) a *shape parameter*, which defines the skewness of the distribution. The above parameters were estimated in this study with the method of *smallest observed drought* (Gumbel, 1967; Raudkivi, 1979).

4.1.2 Optimum Discharge Windows

The time periods in a year during which the assimilative capacity of a river reaches its maximum levels can be considered as the optimum time periods for the discharge of waste from the various pollutant point sources of the river. The assimilative capacity of the river over a time interval $[t_1, t_2]$ at a river site i can be quantified through an implied river volume V_i that is available for dilution at the above interval. This volume is generally estimated from the integral of river flow with respect to time or from its discretized form when there are correspondingly continuous or discrete measurements of river flows $Q_{t,i}$ with $t \in [t_1, t_2]$, i.e.

$$V_i = \int_{t_1}^{t_2} Q_i dt \quad (4.4)$$

or

$$V_i = \sum_{t=t_1}^{t_2} Q_{t,i} \Delta t_i \quad (4.5)$$

In order to have a conservative estimate of the implied volume, the daily measured flows in the above relation are substituted by the daily 7 or 30 day, 20 yr. low flows ($Q_{t,low}^7$ or $Q_{t,low}^{30}$), which are estimated for every median day t with $t \in [t_1, t_2]$. Furthermore the volume is calculated based on the $Q_{t,low}^7$ or $Q_{t,low}^{30}$ flow, which is always equaled or exceeded in the time period $[t_1, t_2]$ and can be notated as $Q_{min,low}^7$ or $Q_{min,low}^{30}$, i.e.

$$V = Q_{min,low}^7 \Delta t \quad (4.6)$$

$$\forall t \in \Delta t, Q_{min,low}^7 \leq Q_{t,low}^7 \quad (4.7)$$

Thus the optimum discharge window for the river site with the available historical data can be estimated as the time subinterval:

$$\Delta t_{opt} = (t_2 - t_1)_{opt} \quad (4.8)$$

of the whole study period of interest Δt_{study} , during which the estimated (using the above procedure) river volume achieves its maximum value V_{max} i.e.

$$\Delta t_{opt} = \left[\Delta t : \Delta t \in \Delta t_{study} \text{ and } V(= Q_{min,low}^7 \Delta t) = V_{max} \right] \quad (4.9)$$

Applying this procedure, optimum discharge windows can be estimated for all the river sites that have valid available data. Then for any given study period a minimum, average, and maximum window applicable in the overall river system can be found, using the starting and ending day of the optimum window for each river site i , $(t_{1,opt})_i$, $(t_{2,opt})_i$ and the following relations:

$$\Delta t_{opt,min} = \max_i(t_{1,opt})_i - \min_i(t_{2,opt})_i \quad (4.10)$$

$$\Delta t_{opt,aver} = \text{aver}_i(t_{1,opt})_i - \text{aver}_i(t_{2,opt})_i \quad (4.11)$$

$$\Delta t_{opt,max} = \min_i(t_{1,opt})_i - \max_i(t_{2,opt})_i \quad (4.12)$$

4.1.3 Minimum Daily Flows - Regional Analysis

From the estimated daily $Q_{t,low}^7$ and $Q_{t,low}^{30}$ flows of each river gauge station and for any study period, the *minimum daily flow* of the station during the minimum, average,

and maximum window of the specified period can be found using eq. (4.7):

$$(Q_{min,daily}^7)_{wType} < Q_{t,low}^7 \quad \forall t \in \Delta_{opt,wType}, wType: min, aver, max \quad (4.13)$$

The minimum daily flow of any river site for a specified study period and discharge window will be considered in the present study as the *design* river flow of the particular site. There is a need to estimate this flow at any gauged or ungauged river point. *Regional analysis* provides a tool for achieving this, by relating through single or multiple regression the minimum daily flow with topographic, geologic and climatic characteristics of the river basin. From the above characteristics, the *drainage area* is the most accurate parameter to quantify. Thus regional analysis usually relates the minimum daily flow at a point with the drainage area above that point, assuming a homogeneity in the other features in the river basin (Riggs, 1973).

Such an analysis has been performed in this study by fitting a linear regression model to the estimated $Q_{min,daily}^7$ flow of the gauge stations and the drainage area (A_D) above them. A measure of the appropriateness of this relation is given by the correlation coefficient:

$$Q_{min,daily} = const. A_D \quad (4.14)$$

If the correlation is not acceptable, then the minimum daily flow at any river point i ($Q_{min,daily}_i$) is found from the corresponding estimated flow at a river gauge station j ($Q_{min,daily}_{g_j}$) and the ratio of the drainage areas of the two river points, i.e.

$$(Q_{min,daily})_i = \frac{A_{D,i}}{A_{D,g_j}} (Q_{min,daily})_{g_j} \quad (4.15)$$

Alternatively, other linear regression models could be checked.

4.1.4 Travel Times for the River System

The travel time for the water body between any two points in the river system is strongly dependent on the velocity variation between these points, thus on the magnitude of flows and the channel geometry for the specified part of the river system. In the present study, the travel times (Δt) are distinctly estimated between any river point and its downstream (at distance Δs) junction node or discharge point. The average velocity (v) over the reach cross section, as calculated at the beginning of the reach, has been used to determine the travel time (*upwind scheme*), i.e.

$$v = \frac{\Delta s}{\Delta t} \quad (4.16)$$

This average velocity can be estimated from a Manning type equation (Burke, 1983). The correlation that was found for our particular case study of the South Nation watershed is

$$v = 0.062 Q^{0.4} \quad (4.17)$$

where v is given in m/s and Q , the volumetric flowrate, in m^3/s (Droste and Gibbons, 1992). By setting appropriate Q values, the travel time of the river water body in the presence or absence of its point sources can be estimated.

The travel time between any two river points is estimated as the sum of travel times from the starting river site to its successive junction nodes or discharge points up to the final river site.

4.2 Water Quality Model

In the present study a *steady-state, node-reach, coupled, non-conservative biochemical* model has been applied for the water quality simulation of a branching river system that receives *point* and *non-point* pollution loads. In particular, the water quality simulation describes the variation and the interrelation of DO, BOD and total ammonia concentration in the river system.

4.2.1 Concentration Change due to Biochemical Processes

The dominant process assumed to be responsible for the concentration variation of DO, BOD and ammonia is the utilization of DO for the oxidation of the organic and inorganic load in the river. The rate of the various forms of oxidation and of the other mechanisms under study is mainly expressed through first-order kinetics with respect to concentration. The specific rate of these reactions is temperature-dependent and is assumed to be given by the following modified Arrhenius equation (Tetra Tech, 1978a; Jorgensen, 1983):

$$k_T = k_{T_{ref}} \theta^{T-T_{ref}} \quad (4.18)$$

where the specific rate k_T at a temperature T can be found from the specific rate $k_{T_{ref}}$ at a reference temperature T_{ref} and from a temperature correction factor θ (dimensionless). The k_T is expressed in d^{-1} and the temperature in $^{\circ}C$. The $k_{T_{ref}}$ can be estimated either from empirical relations that describe its dependence on physical characteristics of the system or from values found in the literature. Similarly θ is found from available estimates in the literature. The above equation gives acceptable results for temperatures

between 5 and 30°C and has been extensively used in river water quality models (Jorgensen, 1983).

It should be noted that there is a wide variety of alternative formulations to those that will be presented here. Higher than first-order kinetics have been suggested to describe better the processes under study. Also the temperature dependence is expressed by different types of functions that account for an optimum process temperature. Finally, the specific rates at a reference temperature are estimated from *in situ* measurements whenever this is feasible, from laboratory experiments after modification of the obtained results, or from the calibration of the whole water quality model.

In the following sections, the overall rate of change in the concentration C of the constituents modelled due to chemical or biological transformation r_C will be briefly presented. This rate will be included in the mass balances in the water quality model.

Given the initial concentration C_0 of a constituent at a time interval $[t_0, t_f]$ and this rate, the final concentration C_f due to chemical transformation, can be estimated from the following equations:

$$r_C = \frac{dC}{dt} \quad \text{where } C: L, N, DOD \quad (4.19)$$

$$C_f = \sum_{t_0}^{t_f} r_c dt \quad (4.20)$$

In the above, L stands for carbonaceous biochemical oxygen demand, N for total ammonia and DOD for dissolved oxygen deficit.

4.2.1.1 Carbonaceous Biochemical Oxygen Demand (CBOD) Variation

The carbonaceous biochemical oxygen demand (CBOD) represents the

requirements of DO for the biodegradation of the carbonaceous organic material in the river; that is, for the oxidation and decomposition of the complex organic compounds into simpler products with the aid of heterogeneous microbial populations which use the waste material as a substrate for their growth. The amount of oxygen required for this oxidation is equal to the reduction in the BOD of the river.

The above implies that BOD is an indirect measure of the biodegradable organic material present in a river system. The removal of this material can result among others from the oxidation of its soluble part, the sedimentation and flocculation of its suspended part or from the reaction of its volatile part with bacteria in the benthos followed by release of methane and hydrogen sulphide gases. The above processes can be *lumped* into an *overall removal or exertion* rate of carbonaceous BOD denoted by r_{BODrem} , and expressed by the following relation:

$$r_{BODrem} = -k_r L \quad (4.21)$$

The specific removal rate of BOD denoted by k_r , depends on the biological growth in river beds, the immediate demand by some waste components, the river flow, the turbulence, the temperature, the type and content of solids and the toxicity of the river water. A typical range of its values at a reference temperature of 20°C is [0.05, 0.50] d⁻¹ (Tetra Tech, 1978a, 1978b; (Jorgensen, 1986). The lower values correspond to potable river water while the higher values correspond to municipal mechanically treated discharged waste. The temperature correction factor of the specific rate is usually set equal to 1.047, a value estimated by Streeter and Phelps (Orlob, 1983) for temperatures between 10 and 35°C.

In the present model the overall exertion of ultimate BOD is assumed to be the main biochemical transformation process that causes changes in its concentration. Thus the rate of change r_L and its integrated form can be described by the following equations:

$$r_L = r_{BODrem} = -k_r L \quad (4.22)$$

$$L_f = L_0 e^{-k_r t} \quad (4.23)$$

The BOD concentration in a river or any other source is usually expressed in terms of the laboratory measured BOD₅ value (C_5), which represents the oxygen demand exerted under specified conditions in a sample of water in a period of 5 days. If the ultimate oxygen demand of the sample is L_u , then the remaining oxygen demand after 5 days L_5 can be estimated using eq. (4.23) substituting L_0 with L_u and L_f with L_5 . In that way the 5-day oxygen demand can be related to the ultimate oxygen demand by the following relation:

$$X_5 = L_u - L_5 = L_u (1 - e^{-k_{r,lab}(5)}) \quad (4.24)$$

Note that the specific removal rate under laboratory conditions will not be the same as in the actual conditions due to differences in temperature and sunlight, the composition and size of bacterial population, the water movement and degree of turbulence. Thus k_r has been also substituted by $k_{r,lab}$ which has a typical value of 0.23 d⁻¹.

4.2.1.2 Total Ammonia Variation

The ammonia present in the river system results mainly from the direct discharge of ammonia nitrogen and the hydrolysis of organic nitrogenous matter. During the nitrogen cycle, ammonia may undergo an autotrophic biochemical oxidation process called *nitrification*. Nitrification occurs under favorable conditions of temperature, pH, and DO

concentration and in presence of specific nitrifying microorganisms. These microorganisms are strictly aerobes, and are distinct from those performing the biodegradation of the carbonaceous organic material. The nitrification reaction proceeds *sequentially* from ammonia through nitrite to nitrate, however its rate is controlled from the first step, the ammonia oxidation, which is the slowest one (O'Connor et al., 1973).

Nitrification can be mathematically expressed through the kinetics of its two sequential reactions or can be represented as a single-stage reaction and be described by a first-order relation (Tetra Tech, 1978a; Jorgensen, 1986):

$$r_{nitr} = -k_n N \quad (4.25)$$

In the latter approach (followed in this work) the parameter values are close to those in the kinetics of the controlling step. The specific rate of total ammonia oxidation (k_n) depends on the temperature, the pH, the suspended particles concentration, the benthos, the nature of waste treatment, and the hydraulic parameters of the river system. When it accounts for the hydrolysis of organic matter in addition to the oxidation of ammonia, its values may be lowered due to the time lag, i.e., the delay introduced by the hydrolysis process. A reported range of values for this specific rate is [0.10, 0.54] d⁻¹. (Tetra Tech, 1978a; 1978b). Such range is valid for temperatures between at 20 and 29°C. A default range of the specific rate for the ammonia hydrolysis ($k_{n,20}$) at 20°C is [0.1, 1.0] d⁻¹ (Droste and Gibbons, 1992). The reported values of the temperature correction factors θ for the rates of the nitrification reactions range from 1.05 to 1.11 (Jorgensen, 1983). The higher of them have been estimated or measured for the case of ammonia oxidation and are considered more appropriate for the temperature dependence of the rate k_N .

In this work the nitrification process is considered as the main transformation process responsible for the total ammonia variation. Thus the overall rate of change due

to reaction r_N and its integrated form can be obtained using:

$$r_N = r_{nitr} = -k_n N \quad (4.26)$$

$$N_f = N_0 e^{-k_n t} \quad (4.27)$$

4.2.1.3 Dissolved Oxygen Deficit Variation

The DO is a necessary constituent for the maintenance of aerobic conditions in a river, hence for the safe use of water and the support of aquatic life. Its concentration, DO , should be ideally at the saturation level, DO_s , for the given conditions. This is why the quantity of DO in a river is usually studied in terms of the difference (*deficit DOD*) of its concentration from the saturation concentration value or in terms of the percentage of DO saturation $\%DO_s$, i.e.

$$DOD = DO_s - DO \quad (4.28)$$

$$\%DO_s = \frac{DO}{DO_s} = \frac{DO_s - DOD}{DO_s} \quad (4.29)$$

Several expressions have been developed relating the DO_s with the water temperature and salinity and to a smaller degree with the river altitude and the corresponding barometric pressure. In this work DO_s has been estimated using the following relation developed by Hyer et al. (Tetra Tech, 1978a):

$$DO_s = 14.6244 - 0.367134 T + 0.0044972 T^2 - 0.0966 S + 0.00205 ST + 0.0002739 S^2 \quad (4.30)$$

where T is the temperature in °C and S is salinity in ppt.

The DO deficit increases during the bacterial decomposition of the carbonaceous organic matter. As can be derived from the CBOD definition, the rate of this increase is equal to the rate of CBOD removal r_{BODrem} .

The deficit further increases due to the process of nitrification. The consumed DO can be estimated as the oxygen equivalent of the oxidized ammonia. The stoichiometry of the nitrification reactions has been described by Hutchinson (1957). Since 3.76 gr of oxygen are utilized per gr of ammonia (O'Connor et al., 1973), the rate of deficit increase will be 3.76 times as much as the nitrification rate.

Another source of DO deficit variation in the river is the benthic oxygen demand. This is mainly required for the aerobic decomposition of the deposited organic material in the upper part of the sediment, as well as the decomposition of the degradable particulates that are conveyed back in the river water. DO is also consumed for the oxidation of gaseous products which arise after the anaerobic decomposition in the sediment (Moss, 1988).

The rate of benthic oxygen demand (r_{bod}) can be described as a function of the areal rate of the benthic oxygen demand (k_{bod}) and of the stream depth (d) using the formula:

$$r_{bod} = \frac{k_{bod}}{d} \quad (4.31)$$

A typical range of values for the areal rate of bod is [0.1, 10] g/m²/d and is narrowed to [0.1, 5.0] g/m²/d for streams not exposed to continuous wastewater effluent discharge (Droste and Gibbons, 1992). The reported values for the temperature correction

factor k_{bod} are in the range of [1.04, 1.15] for temperatures between 5 and 30°C.

The increase in DO deficit that results from the above processes is assumed to be balanced by the replenishment of river with DO through reaeration. The process of reaeration refers to the dissolution of gaseous oxygen in a river and depends on the temperature, the DO deficit, the hydraulic properties of the river, the wind velocity, the turbulence, and the concentrations of surface active agents, suspended solids and dissolved substances.

The rate of reaeration r_{reacr} can be described as a first-order equation with respect to the DO deficit according to:

$$r_{reacr} = -k_a D \quad (4.32)$$

The specific rate of reaeration (k_a) can be estimated by several *direct* and *indirect* methods (Jorgensen, 1986; Orlob, 1983; Tetra Tech, 1978a).

O'Connors and Dobbins

$$k_{a,20} = 1.707 v^{0.500} d^{-1.500}, \quad v \in [0.03, 0.5] m/s \quad d \in [0.6, 13] m \quad (4.33)$$

Churchill et al.

$$k_{a,20} = 2.178 v^{0.968} d^{-1.673}, \quad v \in [0.5, 1.7] m/s \quad d \in [0.6, 13] m \quad (4.34)$$

Owens et al.

$$k_{a,20} = 2.316 v^{0.670} d^{-1.850}, \quad v \in [0.03, 1.7] m/s \quad d \in [0.08, 1.6] m \quad (4.35)$$

In this study the specific rate at 20°C ($k_{a,20}$) is estimated using the *predictive*

formulas (4.33) to (4.35) that relate it with the velocity (v) and depth (d) of the river. The temperature correction factor by this rate is usually taken as $1.024 d^{-1}$.

The processes discussed above are assumed to be the major ones that determine the variation in the DO deficit due to (bio)chemical transformations. When combined they give the overall rate of change (r_D) and its integrated form, as shown below:

$$r_D = -r_{BODrem} - 3.76r_{nitr} + r_{reaer} + r_{bod} = k_r L + 3.76k_n N - k_a D + \frac{k_{bod}}{d} \quad (4.36)$$

$$D = \frac{k_r L_0}{k_a - k_r} \left[e^{-k_r t} - e^{-k_a t} \right] + \frac{3.76 k_n N_0}{k_a - k_n} \left[e^{-k_n t} - e^{-k_a t} \right] + \frac{k_{bod}}{d k_a} \left[1 - e^{-k_a t} \right] + D_0 e^{-k_a t} \quad (4.37)$$

4.2.2 Point and Nonpoint Source Load

The point pollutant sources of a river accounted in this study are the effluents of water and wastewater treatment plants that discharge on it. The pollutant point sources are of major significance in watersheds with a high percentage of urban areas. For a management study, the treatment plants can be restricted to discharge during the optimum discharge windows, Δt_{opt} , of any study period. If a constant effluent rate is also assumed, and the total waste volume allowed to be discharged in the study period is V_w , then the *design waste flow*, Q_w , can be estimated by:

$$Q_w = \frac{V_w}{\Delta t_{opt}} \quad (4.38)$$

The effluent concentration (C_w) can be also assumed constant in the duration of the optimum window, and equal to a typical concentration value expected from the

specified facilities and degree of treatment in the plant. From the above values of Q_w and C_w , the treatment plant loads (I_w) introduced in the river can be estimated.

The tributary inflows of a river transfer the load that was discharged on or drained by them, thus forming an additional pollution source for the river.

The *non-point sources* of a river and its *base load* include the runoff from urban and rural areas of the watershed, the atmospheric pollution and the *internal load* from groundwater infiltration and from benthic (bio)chemical transformations. In the present study, the contribution of the above non-point sources to the flow in a part of the river between two points A and B has been estimated as the difference in the *minimum daily flow* of those two points, $\Delta Q_{np}^{A,B} = Q_{min,daily}^B - Q_{min,daily}^A$.

The concentration of the pollutant constituents in the non-point source flow has been considered constant for the whole river system and for all study periods and windows, and equal to the *background concentration* (C_b) of the constituents. The background concentration of each constituent has been derived from statistical analysis applied to the data of each river quality station for each study period. In particular it has been defined as the average of the percentile values of certain order that are calculated in each of the above samples. It should be noted that concentration measurements reflect the influence of both point and non-point sources, while the background concentration should refer only to the non-point sources. The approach followed is appropriate when the point source load has a small contribution to the overall river load.

From the values of $\Delta Q_{np}^{A,B}$ and C_b , the non-point source load in the part of the river between points A and B ($I_{np}^{A,B}$) can be estimated using:

$$I_{np}^{A,B} = \Delta Q_{np}^{A,B} C_b = (Q_{min,daily}^B - Q_{min,daily}^A) C_b \quad (4.39)$$

4.2.3 Coupled Water Quantity - Quality Model

In the present model, the configuration of the river system is represented by a node-reach network. The beginning of each river or tributary corresponds to a headwater source (*head node*), while the junction point of any two or more rivers is represented by a *junction node*. The part of the river between two successive interconnected junction nodes is represented by a *reach*. The site on a river reach that accepts the load of a point pollutant source is defined as a *discharge point* of the system. The part of a reach between two successive discharge points that may be defined on it or between a discharge point and its successive or prior junction node is a *subreach* of the system. A subreach is considered to have uniform hydraulic, physical and biological characteristics, thus its length depends on the specific river system under study. In the present case study of the South Nation River system, the length of a subreach varies from one to twenty kilometers.

The simulation of water quality in the river system has been performed applying the *conservation of mass principle* for all modelled constituents successively to all (sub)reaches in the direction of flow in the system. The *assumptions* determining the form and solution of the mass balance equation, and the river system conditions during simulation are presented below.

Assumptions

The water quality simulations have been undertaken in each study period for the time intervals of the corresponding optimum discharge windows, the latter been estimated using eqs. (4.10) to (4.12). During these intervals the *base river flow* has been considered *constant* and equal to the corresponding *design minimum daily flow* that is estimated by eqs. (4.14) or (4.15). The discharge flow of each point source has been also considered *constant* and estimated by eq. (4.38) for each specified optimum window and study period.

Under the above considerations, steady-state conditions have been assumed. In reality, the variation of water quality in a river is a transient phenomenon since the flows, velocities, turbulence, temperature, point-source discharge and nonpoint-source runoff vary with time. However steady-state models are simple and have been widely used in long-term planning of river water quality (Oriob, 1983; Loucks in Biswas, 1976).

Applying the steady-state assumption the accumulation term in the mass balance equation has been neglected. The transport of a constituent by dispersion has been also assumed negligible. The velocity at each subreach has been assumed constant and equal to the velocity at the beginning of the subreach. The concentration of the constituents modelled has been also assumed uniform throughout each cross section in the river.

The change in concentration due to (bio)chemical reactions has been quantified using eqs. (4.22), (4.26) and (4.36). Other sources and sinks of a constituent in the (sub)reach have been represented by the non-point source pollutant load. This load has been considered to enter the (sub)reach just prior its starting node. The point source load which is transferred by the treatment plants effluent (w), or any tributary inflow ($trib$), or the inflow of any joining river (jn), similarly enters the system in the river node just before the beginning of the reach.

The above point and non-point source loads have been accounted for in the flow and mass balances that are performed at the starting node of the reach and influence the initial (in space) flow and concentration of the reach. This concentration has been considered as the boundary condition for the differential form of the conservation of mass, hence it has been also included in its integrated form over the reach length.

Schematic representation

The above can be better understood by referring to Figure 4.1. The reach $R_{j,j+1}$ is defined between a discharge point (dp) denoted by j and a junction node (jn) denoted by $j+1$.

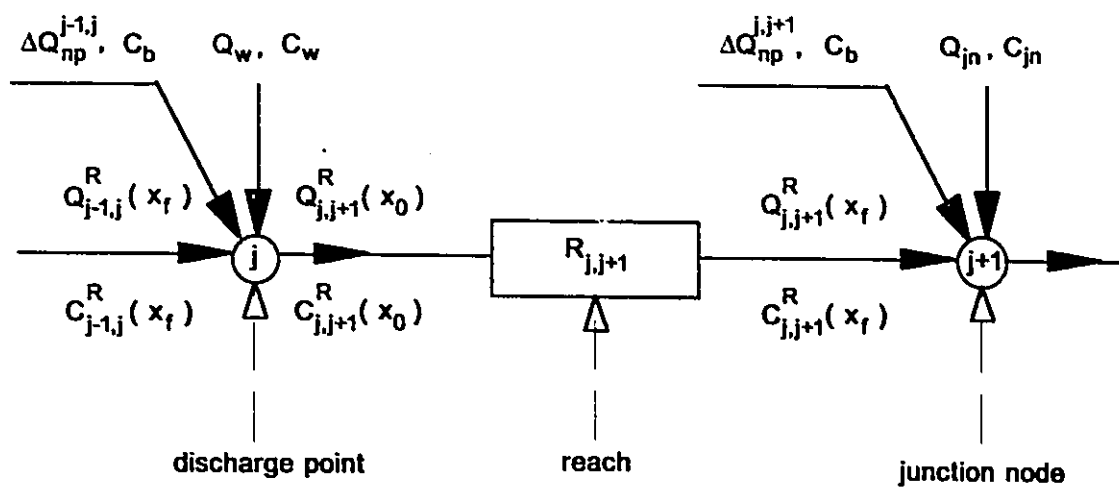


Figure 4.1: Transport of pollutants in node-reach model.

At the beginning of the reach, three flow streams are considered to be mixed:

- (i) the flow stream at the end of the previous reach $R_{j-1,j}$ with features $Q_{j-1,j}^R(x_f)$ and $C_{j-1,j}^R(x_f)$,
- (ii) the contribution of the nonpoint sources between nodes $j-1$ and j with features $\Delta Q_{np}^{j-1,j}$ and C_b , and
- (iii) the treatment plant effluent stream with features Q_{side} and C_{side} (where *side: w*).

The conditions at the beginning of the reach can be estimated using:

$$Q_{j,j+1}^R(x_0) = Q_{j-1,j}^R(x_f) + \Delta Q_{np}^{j-1,j} + Q_{side} \quad (4.40)$$

$$Q_{j,j+1}^R(x_0) C_{j,j+1}^R(x_0) = Q_{j-1,j}^R(x_f) C_{j-1,j}^R(x_f) + \Delta Q_{np}^{j-1,j} C_b + Q_{side} C_{side} \quad (4.41)$$

where "side" can be in general *w*, *trib* or *jn*.

The flow stream after the mixing, with features $Q_{j,j+1}^R(x_0)$ and $C_{j,j+1}^R(x_0)$, will travel until the end of the reach being subject to the (bio)chemical reactions. The calculated $C_{j,j+1}^R(x_0)$ for the biochemical oxygen demand, total ammonia, and DO deficit, are the corresponding L_0 , N_0 and D_0 in eqs. (4.23), (4.27) and (4.37). From those equations the concentration $C_{j,j+1}^R(x_f)$ at the end of reach $R_{j,j+1}$ can be calculated for all constituents modelled, substituting also the selected values of specific coefficients and the value of time. It should be noted that the travel time has been calculated from eqs. (4.16) and (4.17) using the length of the reach and the $Q_{j,j+1}^R(x_0)$ flow.

A similar sequence of calculations can be applied successively to all reaches, thus complete the water quality simulation for the whole river system.

4.3 Water Toxicity - Dilution Requirements

Toxicity is one of the primary biological indicators of the quality of river water and wastewater effluent (Krenkel and Novothy, 1980; Ricklefs, 1990). It can be caused by human activities (mainly through the discharge of contaminated wastewater or application of practices that increase the nonpoint load of toxic material), as well as by natural biological processes that take part in the river (the major one being the anaerobic decomposition of organic material). Toxicity may have short or long term destructive effects on the assimilative capacity of a river, on the aquatic ecosystem and on users of its water. Thus the control of the toxicity level in a river is a necessary part of the surface water quality management.

Theoretically, every compound can be toxic, depending only on its concentration and the tolerance of the biota in the aquatic environment to a particular toxicant (Krenkel and Novothy, 1980). However certain compounds like heavy metals, oils, chlorinated hydrocarbons, pesticides, radionuclides, and a series of organic compounds are toxic even at very small concentrations (Orlob, 1983).

The present study focuses on two toxic substances which cause unpleasant taste and noxious odor in water and have even lethal effects above certain concentration limits. These substances are the undissociated ammonia $(NH_3)_u$ and the undissociated hydrogen sulphide $(H_2S)_u$. The degree of dissociation of the corresponding total ammonia $(NH_3)_{tot}$ and total hydrogen sulphide $(H_2S)_{tot}$ is strongly dependent, among other factors, on the temperature and pH of the water under study.

For the case of $(NH_3)_{tot}$ dissociation, the following correlation has been used.

$$\frac{[(NH_3)_u]}{[(NH_3)_{tot}]} = \frac{10^{[pH-10.08+0.042T^{0.918}]}}{1 + 10^{[pH-10.08+0.042T^{0.918}]}} \quad (4.42)$$

This correlation describes the ratio of free and coupled ammonia as a function of pH and temperature and is based on the equilibrium model (Droste and Gibbons, 1992). In the above relation all concentrations are expressed in mg/m³ and the temperature in °C.

For the case of $(H_2S)_{tot}$ two-step dissociation, the following correlation has been used.

$$\frac{[H_2S_u]}{[H_2S_{tot}]} = 1 + \frac{6.348 \times 10^{-4} \exp\left(-\frac{2616}{T}\right)}{[H^+]} + \frac{2.993 \times 10^{-28} \exp\left(\frac{4565}{T}\right)}{[H^+]} \quad (4.43)$$

This correlation has been derived by combining the dissociation reactions, the Van'tHoff relationship and the extended Debye-Huckel and Guntelberg approximations (Droste and Gibbons, 1992). In the above relation all concentrations are expressed in mg/l and the temperature in K.

Equations (4.42) and (4.43) can provide an estimate of the toxic substances concentrations in the effluent of a point source or in the river point downstream of a point discharge. These concentrations can be compared directly with specified water quality standards and assist in the evaluation of the river water quality and its suitability to safely supply its uses.

A significant physical mechanism for the reduction of toxicity occurring from point discharges in a river is the dilution of the added load that takes place in the river water. The adequacy of the present dilution or the need for increase can be quantified with the use of the *dilution factor* D , which is defined for a discharge point as the ratio of the river flowrate Q_r at that point immediately prior the discharge to the point-source flowrate Q_w , i.e.

$$D = \frac{Q_r}{Q_w} \quad (4.44)$$

Under the assumption of steady-state conditions both Q_r and Q_w flowrates are constant hence the dilution factor of a discharge point is constant as well.

Under the assumption of *perfect mixing conditions* immediately after the discharge point in a river, thus considering as negligible the mixing zone effects, the dilution factor can be expressed in terms of the river concentration before and after the mixing (C_r and C_m) and of the point source effluent concentration, C_w , i.e.

$$D = \frac{C_w - C_m}{C_m - C_r} \quad (4.45)$$

Substitution of the present values of Q_r and Q_w flowrates in eq. (4.44) provides an estimate of the *available dilution* at a river point. Substitution of the toxicity standards in the C_m or C_w concentration in eq. (4.45) gives an estimate of the *required dilution* in order to satisfy the river or point source effluent limits in toxic substances. A comparison of the above two dilution factors provides a means to identify the discharge points that require a higher dilution capacity and the magnitude of this requirement in order to prevent the exertion of harmful toxic effects.

4.4 Strategies of Water Quality Control

The ability of treatment plants effluent and of river water quality to meet the concentration limits of various important constituents, the requirement for establishing quality control methods, and the evaluation of those methods can be studied in two cases. *In the first case (scenario A)* the river system accepts the effluent of *each* treatment plant individually in a study period. *In the second case (scenario B)* it accepts the effluent of *all* treatment plants discharging simultaneously at the same study period.

The results of the *first scenario* may be used for a preliminary identification of the plants and sites where control is required, as well as for narrowing the range of suggested strategies. This analysis includes the following points:

- A1:** The measured concentrations of $(NH_3)_{tot}$ and $(H_2S)_{tot}$ in the plants effluent can be compared to the maximum allowed concentrations, for the given conditions of temperature and pH, that correspond to nonlethal limits of the toxic compounds, $(NH_3)_u$ and $(H_2S)_u$. If the limits are exceeded, there is an indication that there is a need for an advanced or an extra treatment facility for the reduction of the concentration to acceptable levels.
- A2:** The maximum dilution factor at each discharge point can be estimated for all study periods and for all of their optimum windows, assuming in each case the discharge of the total annual waste volume of the corresponding plant. Significantly higher D_{av} values for one study period could suggest this period as the most appropriate for discharge. However, comparable values of D_{av} for both periods may indicate that the total waste volume could be apportioned not necessarily equally and discharged in both of them. High differences in the D_{av} values for various windows could also suggest the corresponding stagger of the discharge in these windows.
- A3:** The available dilution factor at each discharge point can be compared with the corresponding required dilution factors for the toxic materials and for the current conditions in the river and in the plants. Insufficiency of the available dilution to provide the required dilution might show the need for extra facilities for the removal of the toxic substance, similarly to case A1. Alternative types of

treatment can be checked through comparison of their dilution requirements with the existing D_{av} .

A4: The value of D_{req} can be also set equal to the value of the corresponding D_{av} for a period that was suggested as comparable, but less appropriate for discharge, from point **A2**. The maximum waste volume allowed to be discharged in this period can then be estimated. If it is a significant portion of the total annual waste volume, it will strengthen the feasibility of this period for discharge; otherwise it may show that this option should be rejected from further consideration.

For the estimation of the D_{av} in the above analysis, the flowrate Q_r of the discharge point is set equal to the corresponding $Q_{min,daily}^7$ or $Q_{min,daily}^{30}$ flowrate. For the estimation of the D_{req} , the concentration C_r of the discharge point is set equal to the background concentration of the river.

After the preliminary analysis, the *second scenario* with all plants discharging simultaneously can be studied. Prior to any test for application of a control strategy, simulations are performed that predict the distribution of the constituents modelled in the river for all study periods (time intervals during a year). These simulations result in the identification of the river nodes whose water quality does not meet the defined standards. They also aid in the selection of the study period, which is more appropriate for the discharge of all plants of the watershed.

The possible forms of water quality management are then studied. The objective of the optimization procedures is the *appropriate utilization of the assimilative capacity of the river*, and the *minimization of the required relocation of the present discharge points* as well as *of the degree of plants upgrade* that would result in an *increased cost*.

The methods of water quality control that have been considered are the following:

B1: *Staging of discharge in the estimated optimum windows.* With this method it is evaluated whether a treatment plant can discharge its waste volume during one of the optimum discharge windows, without causing a violation of the standards at the site of the receiving river.

For the achievement of the above, the maximum, average and minimum windows are successively examined. These have respectively the earliest, average and latest starting date, and are expected to correspond to gradually higher river design flows. The river sites are assigned the design flows which correspond to the window currently examined. The treatment plants which were allowed to discharge in an earlier window are expected to satisfy the quality criteria under the new conditions of higher river flowrates. So they maintain their already determined discharge rate. The treatment plants which were allowed to discharge in a later window are assigned a zero discharge rate during the window currently examined.

Under these conditions the maximum waste volume that can be discharged from the treatment plant under study, without violating the limits of the constituents modelled, is estimated in a way similar to step A4. This volume is compared with the annual waste volume of the treatment plant. If it is higher it suggests that the window is appropriate for discharge for the given plant with respect to the defined limits. Thus the discharge of the plant is determined based on the annual waste volume and the window duration. In the opposite case the next window is examined. However, if the current one is the minimum window, it is concluded that one of the following control strategies should be tested for the plant under study.

B2: *Relocation of the discharge point.* This method estimates the required relocation of the discharge point of a plant to a river site where the water quality after the discharge is compliant with the river standards. For that purpose the available and the required dilution at the site of discharge currently examined are estimated and compared. If the available dilution is higher than the dilution required, then the river site becomes the discharge point of the treatment plant under study. Otherwise the current discharge point is relocated to a downstream river site, i.e., to a point slightly downstream of the following river node, where the flow (thus the dilution capacity) is expected to be augmented. The above procedure is then repeated.

When the relocation point tested is at a significant distance from the community, the cost required for the waste transfer will be high as well. In that case the discharge point is transferred to another river closer to the community or with higher flows, and the above procedure is repeated.

B3: *Upgrade or replacement of the treatment facilities.* The current treatment is replaced with more advanced processes which have different operating conditions and result in improved effluent concentration for the constituents modelled. The processes which are gradually examined are (i) the *Lagoon* treatment, (ii) the *Extended Aeration* and (iii) the *New Hamburg*. A process is considered as appropriate for a treatment plant, if the water quality at the sites immediately after the discharge and immediately before the following node is maintained above the defined river quality standards.

For the estimation of the D_{av} and D_{req} in the above analysis, the flowrate Q_r and the concentration C_r of the discharge point are the ones that result from the mixing of the flow stream at the end of the reach prior to the discharge point, with the nonpoint source

load between the discharge node and its previous connected node (these are the flow streams (i) and (ii) described in section 4.2.3).

The above methods are combined as described in the following, to suggest an *optimal discharge scheme* for the plants of the watershed.

The optimization is initially applied to the first point-source discharges in the various branches of the river system. These correspond to the plant effluents of the *remote communities* of the watershed and influence the quality of all downstream sites. The optimization is then applied sequentially to the following nodes of the system until it reaches its final node. A control method is applied to a river node, only if the nodes in the upstream system satisfy the water quality limits.

The staging of discharge is the first strategy to be tested at any point source discharge, since it is not related with any cost. If it does not result in compliance of the water quality with the corresponding limits, it is followed either by the relocation of the discharge point or by the upgrade of the treatment plant. When the application of the first method, **(B2)** results in a high relocation distance, the upgrade of the treatment plant at the location of the initial discharge point is applied, and if required, the relocation procedure is repeated. Similarly when none of the processes considered during the application of method **(B3)** results in an acceptable river water quality, the relocation of the discharge point of the plant is applied, and if required, the upgrade of the treatment plant is again considered. It should be noted that prior to the application of a new trial step of methods **(B2)**, **(B3)** or of their combination, the staging of discharge in the optimum discharge windows is tested.

When two discharge points are located on two reaches with ending node the same junction node and the relocation of both discharge points has to be applied, the sequence of the relocated discharge points after the junction node is defined from the pollutant loadings of the plants effluent and the distance of relocation of each discharge point.

Chapter 5

Object-Oriented Analysis of a Watershed

The object-oriented analysis and design of river quality management that has been developed and implemented in the present work will be described. The major components of the analysis will be explained and also schematically represented. Major points for the reasoning of the present analysis will be also described. The implementation of this analysis will be then discussed and the resulting software tool will be presented.

5.1 Object-Oriented Analysis of River Quality Management

An object-oriented analysis (OOA) of water quality management in a river system has as purpose to identify and define the key objects along with their relationships that correspond to an abstraction of the structure and the water quality analysis and control of a real-world river system. In other words, it has as a purpose the development (*synthesis*) of a model, through objects, for the description of the above specified domain.

At a *high level of abstraction* the major portions of such a model are represented according to the Coad/Yourdon method by the *subjects* (Coad and Yourdon, 1993); the latter correspond to the *class categories* of Booch's methodology (Booch, 1994). A

subject contains usually five to nine related objects (classes) and has a minimum and well defined interdependence with the other subjects.

At a *lower level of abstraction*, thus at a level of increased detail, the objects of the model are identified and organised in classification and composition structures, and their attributes and services are defined. It is noted that the *layering* of objects in subjects can be performed for the initial decomposition of the domain or for the organization of objects at the end of the analysis. However, it is mainly used as a guide in the presentation of a model.

An object-oriented analysis is followed by an object-oriented design (OOD), which is related to the *architectural modelling* of software and is the intermediate step prior to the implementation of the analysis. In the Coad/Yourdon method the design consists of the development of four components. These are (i) the Problem Domain Component (PDC), (ii) the Human Interaction Component (HIC), (iii) the Data Management Component (DMC), and (iv) the Task Management Component (TMC). Their development should be kept distinct in order to facilitate changes in reuse in each one of them, and should be performed by applying the strategies of object-oriented analysis in the corresponding fields.

Some representative subjects that have been included in the present OOA model of the river water quality management are the Watershed, RiverWaterQualitySimulation and RiverWaterQualityManagement. These have been further refined during the PDC design and have been also integrated with the RiverNetworkView and the WatershedInputOutput subsystems, which correspond to the HIC and DMC parts of the design. A schematic presentation of these subjects is shown in Figure 5.1, while their internal organization will be presented in the following.

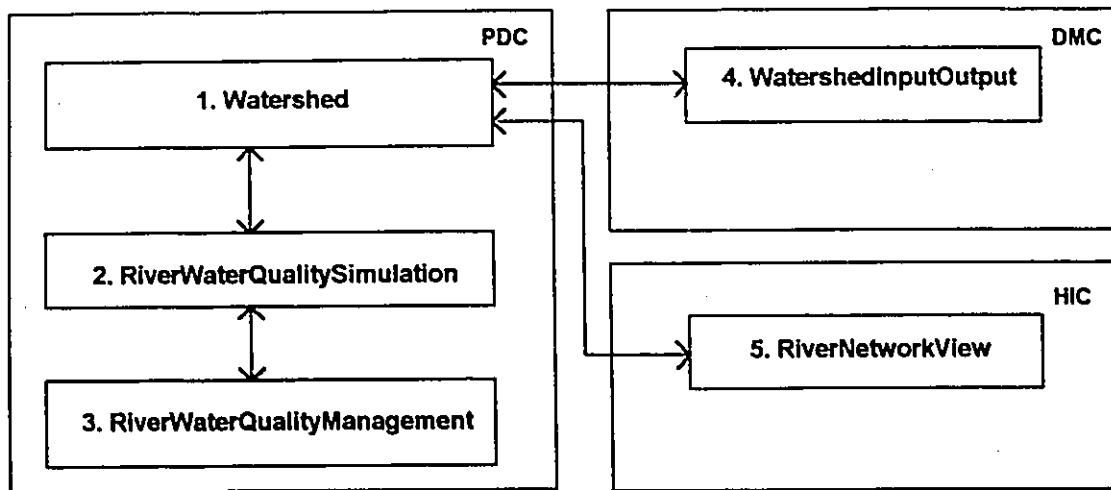


Figure 5.1: Major subjects in the object-oriented analysis for river water quality management.

5.1.1 Representation of a Watershed and of a River System

As has been explained in chapters 1 and 2, the human activities and the physical processes that take place in a watershed have a significant impact on the quality of the receiving river water. Thus a starting object for the present analysis has been the watershed itself. Given the specifications of this study to the management of river water quality with respect to its point-source load from the wastewater treatment plants, the watershed is considered as a *composite object*, as a system which consists of a river system, of communities, industries, and of treatment plants (Figure 5.2). As can be seen from the schematic representation, the watershed *knows about* its river system but it may or may not know about a number of the other *part* objects, depending on its features, on the degree of its urban or rural character and of its development. Each part object also knows about the *whole* object (Watershed). In the same representation, the association between the TreatmentPlant and its source of water or wastewater as well as between the RiverSystem and its point sources of pollutant load can be distinguished. In a refinement of the model, the direct mapping of the communities and industries with the RiverSystem could be added to describe the direct point or diffused discharge of the former in the receiving water.

If the study was extended to the detailed quantification and management of other point-sources and of the nonpoint source load of the river, or to the management of the watershed as a whole, then the parts of the watershed object would be similarly extended. In that case objects like landfills or stormwater sites as well as rural areas, or in more detail, forests and agricultural areas, would be also parts of the watershed.

If the study was extended to the management of river water quality in a higher, *regional* basis, then the watershed object could be a component of the new model, and its features and functionality could be reused.

From the watershed parts, the river system has a major importance for the

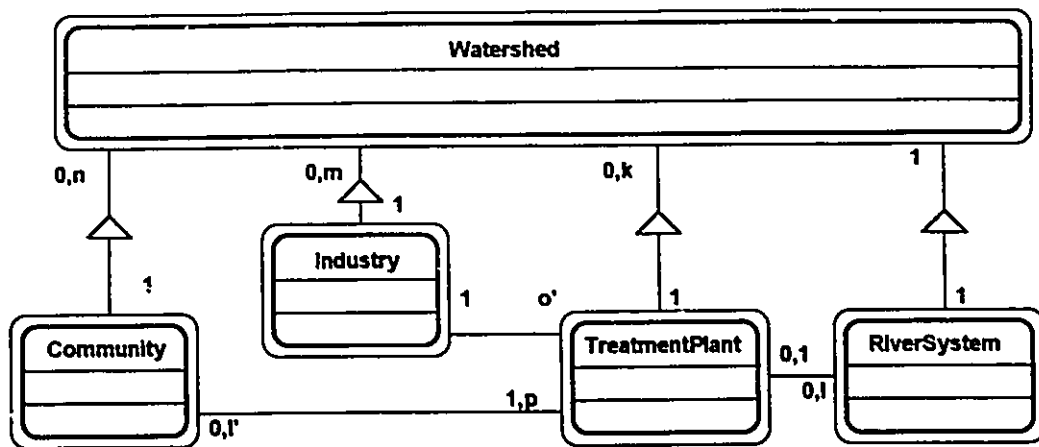


Figure 5.2: The composition structure of the Watershed object. All symbols are explained in Appendix A.

whole study, and that is why it exists as a subsystem on its own. The RiverSystem includes the main river in which the watershed drains and all its tributaries within the area under study. This system has been represented as a network of interconnected RiverNodes and RiverReaches. Hence several of its features are estimated from the *collaboration* of these two components (Wirfs-Brock et al., 1990).

The RiverNode object is an abstract class which defines a part of the behavior that all the nodes must have. Hence it is also the most general class that defines a whole hierarchy of nodes with extended or specialized attributes and responsibilities. Among them there are boundary nodes that define the starting of a tributary and generally the source of water, and are called HeadNodes, or the FinalNode of the system, which does not have a defined downstream node. There are also the internal nodes of the system which have a complete connectivity, i.e., both upstream and downstream node(s), and can be further specialised to nodes where flow and mass balances can be applied. In particular, there are the JunctionNodes which are the points of confluence between any two or more rivers, or the point sources of pollutant load like the TreatmentPlantDischargePoint, where the mixing principle can be applied. Similarly, there could be separation points or diversion sites where one river could give rise to two or more other streams in different directions of flow. Finally, in the present analysis, data acquisition river points have been defined, which are distinguished between RiverGaugeStations, where flow measurements are obtained, and WaterQualityStations where flow and also measurements of various indicators of water quality are obtained.

All the river points described satisfy the *is kind of* relationship, since they are kind of RiverNodes but have differentiated their features in agreement with the above scheme (Figure 5.3). They are also *associated* with different objects, so they are involved in various, distinct steps of the water quality analysis and management in a river. Thus their main functionality is that they define the configuration of the river system. Furthermore, depending on their position in the classification structure, they may collaborate in the

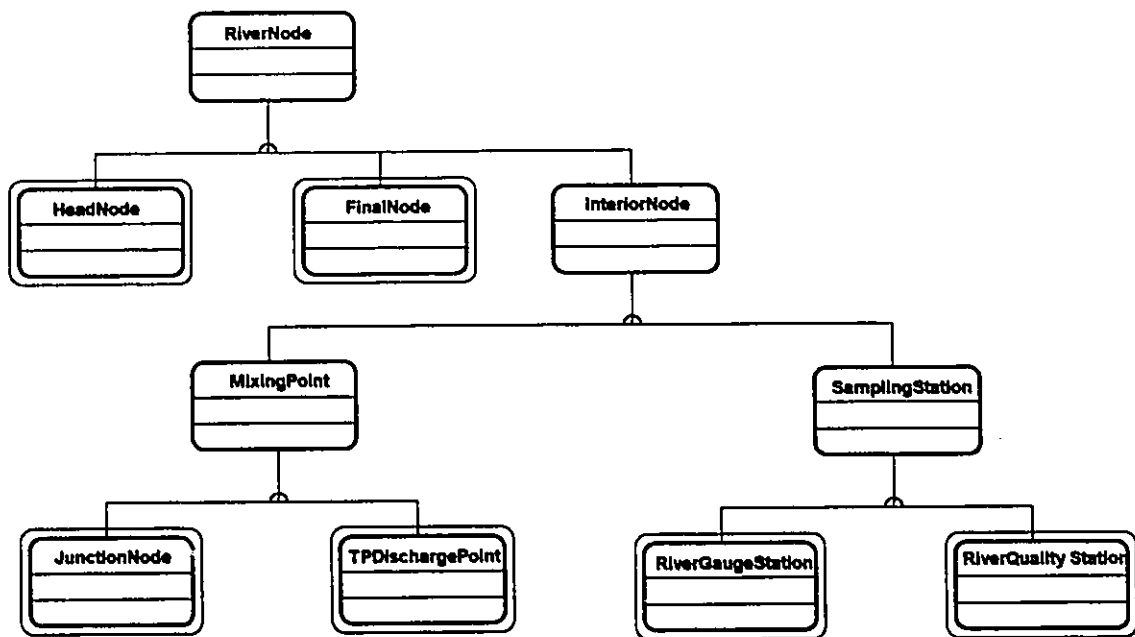


Figure 5.3: The classification structure of the RiverNode object.

simulation of the water quality or the performance of the flow analysis.

The RiverReach class is the second component of the RiverSystem and represents the part of the river between any two junction and/or discharge nodes. The main functionality of the reaches is to define the geometry of the river system. Thus they are also expected to collaborate for the estimation of hydraulic parameters and hydrodynamic quantities, and for the transport of pollutants during the water quantity and quality simulations in the river.

5.1.2 Description of a WastewaterTreatmentPlant System

A wastewater treatment plant can apply a series of processes of various degrees of treatment for the achievement of an acceptable quality in the effluent stream and the reduction of the pollutant load of the influent stream. Due to the inherent complexity and the width of the field, the required degree of abstraction has to be specified prior to the object-oriented analysis of a treatment plant.

In the present study, a wastewater treatment plant knows the *source* of its wastewater (Figure 5.4), so it can *communicate* with it and be informed of the future estimates of the inflow flowrate and wastewater quality. Thus the plant can then inform its effluent about the expected flowrate in the same future period. In that way various scenarios on the rates of growth of the communities or of the industries can be considered and their effects on the plant effluent and receiving water quality could be evaluated. In this analysis, a treatment plant also knows the *type of the final process* that is applied prior to the discharge of the treated wastewater. So it can know the typical expected features of the effluent of this process. In addition, the historical data of water quality of the effluent can be compared with quality standards that have been specified for the plant. Thus the actual quality of the treated water from the plant can be evaluated.

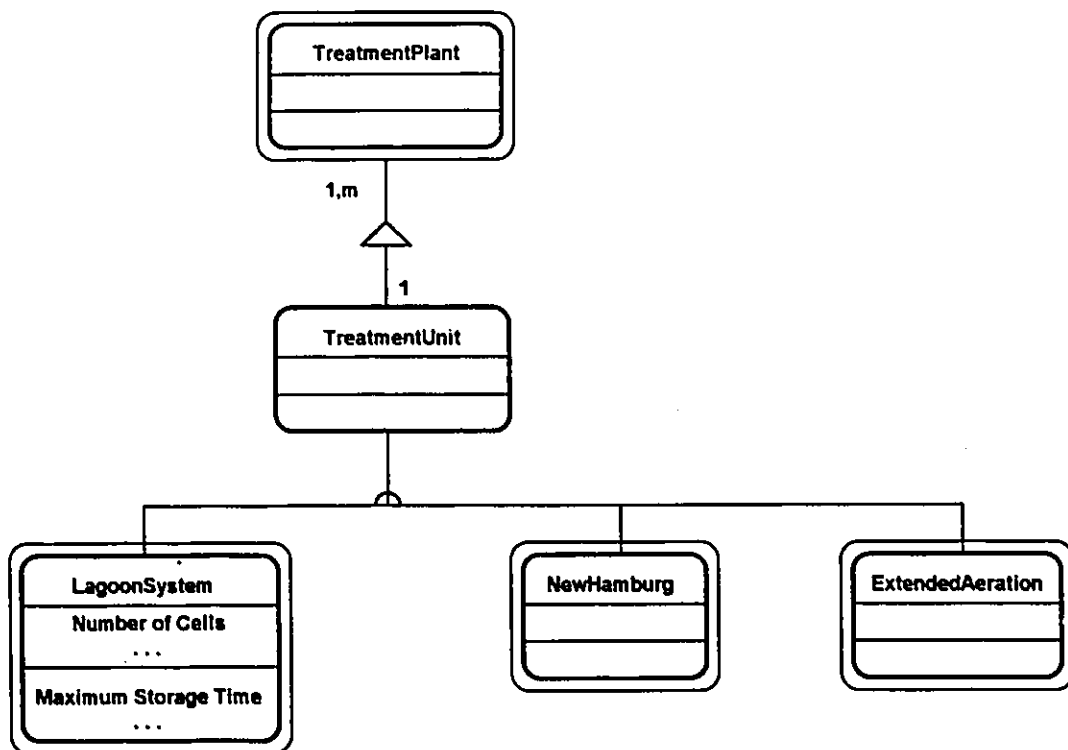


Figure 5.4: Treatment plant system.

It should be noted that a treatment unit-process is represented in this study as an abstract class. Some of its concrete classes include the LagoonSystem, New Hamburg and ExtendedAeration processes.

5.1.3 Description of the WaterQualitySimulation Subsystem

The WaterQualitySimulation subsystem is based on the collaboration of the RiverSystem with several classes that have been defined to describe the hydrodynamic and quality component of a water quality model, and the conditions at the specified time of simulation. The constituents modelled and the processes that describe their variation have been represented by objects and have been combined in the synthesis of the quality component of the model. The hydrodynamic component and the solution of the coupled model depend on the configuration of the river system so they are related to the RiverSystem object. In the present study the solution of the model, that is, the estimation of the constituents variation along the system, starts from the boundary nodes of the RiverSystem (HeadNodes), and when certain conditions are satisfied, it proceeds sequentially towards the final node. Thus the computation requires an initialization of the objects and is then performed through their collaboration, giving the sense of simulation.

5.1.4 Statistical Analysis

This class is responsible for the estimation of several statistical properties of a sample and the application of several methods on its values. Some of these properties and methods include the mean, the variance, the standard deviation, the skewness coefficient, the percentile values of various orders, the minimum, average and maximum value of the

sample. Some of the methods that have been applied include the interpolation technique on the sample values and the numerical single integration using Simpson's technique. A specialization of this class resulted in two other classes for the regression analysis and the fitting of Gumbel III distribution on a sample of data (Figure 5.5). In the latter case the parameters of the distribution are estimated using the *observed drought method* and the *method of moments*.

5.1.5 Graphical Representation of the RiverSystem along with its Point Sources

For the graphical representation of the river system along with its point sources a separate subsystem of objects (classes) has been defined. Their responsibilities concern the display of the parts of Watershed object on a graphical output device. Thus they are associated with the latter objects but they are clearly independent of them. This separation of concerns increases the likelihood of reuse of both systems.

A key object in the present analysis has been an abstract class which defines that each one of its instances should know among others its location on the display medium, its geometric pattern, its colour, the width of its line, and its associated objects. It should also know how to draw itself and generally handle display-related tasks. A series of concrete classes can then be defined for the display of the parts of the RiverSystem and of the TreatmentPlants. It is noted that each drawing object is informed of its connections through its association with the problem domain object. The connectivity, the upstream and downstream nodes were defined as properties of the river nodes because they are also required in other services.

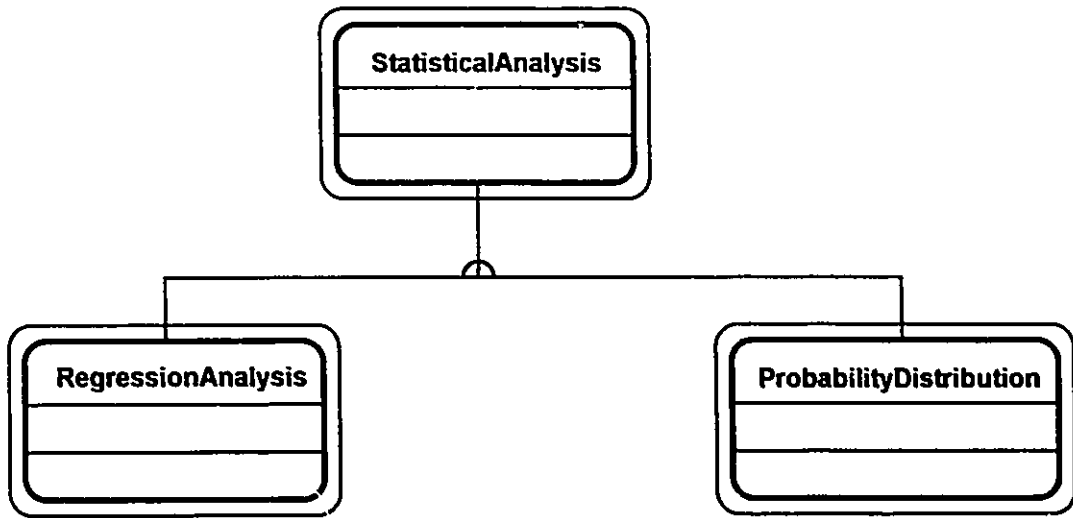


Figure 5.5: The StatisticalAnalysis Hierarchy.

5.1.6 Data Management Component

The design of that component has as its purpose to define how the problem domain objects will be able to store and restore their values. The analysis that has been applied includes the definition of a hierarchy of classes that correspond to the above problem domain objects and know the format with which to store and restore the attributes and the connections of those objects. That component is of particular importance in a water quality management study due to the large amount of data that have to be handled and organized.

5.2 Description of the Developed Software Tool

5.2.1 Organization of Input Data

The required data for the application of the management study are read from input files. These data are associated to the appropriate objects, as was described in paragraph 5.1.6. The required data are the following:

1. Information on the river system configuration. This information includes the code-names and the screen locations of the head nodes, junction nodes, treatment plants, discharge points and river stations. The connections and the downstream river name of each river node are also needed.
2. Daily streamflow measurements ($q_{t,y}$). The present analysis requires data obtained at each river gauge station over a sequence of years.
3. River water quality measurements. Data of the river flowrate and of the *DO*, *BOD₅*, *NH₃-N* concentrations are required. The data should be given on a daily or

monthly basis for a sequence of years and for each river quality station. Alternatively, estimates of the DO , BOD_5 , NH_3-N mass rates can be used. Temperature and pH measurements from each quality station are also needed on a daily or monthly basis.

4. Information on the treatment plants. The temperature, pH, flowrate and water quality data are needed for the effluent of each treatment plant. Information on the type of the last treatment process that is applied by the plant is also of great value. Finally, information on the source of the influent wastewater is required. If the source is a community, population data and projected population estimates are needed.

5. Water quality standards. The water quality standards for the modelled constituents (DO , BOD_5 , NH_3-N) and for the toxic constituents (undissociated ammonia and hydrogen sulphide) are required for in-stream conditions. The water quality standards for the toxic constituents are also needed for the "in-pipe" conditions of the treatment plants effluents.

6. Expected effluent water quality from various treatment processes. In particular, the expected effluent quality from lagoons, New Hamburg processes and extended-aeration processes are needed..

5.2.2 Development of the Interface

The permanent interface of the software tool consists of a main window with three parts and of a series of multi-level menus (Figure 5.6). The first level of menus is presented in Figure 5.7. The parts of the window are used for the presentation of the river system along with its point sources, while the menus are used for the definition or alteration of the system configuration and the selection of the step of the analysis to be performed. In more detail, the window consists of the following panes:

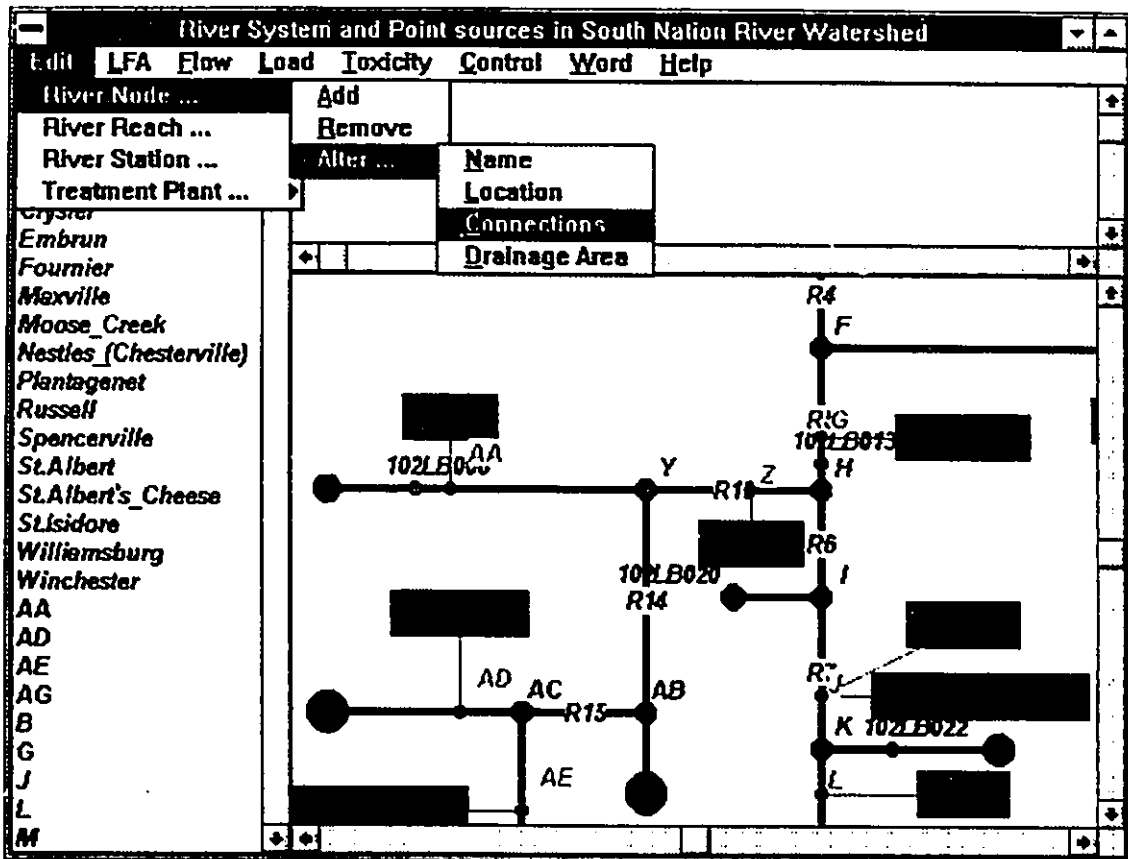


Figure 5.6: River network representation in the software tool.

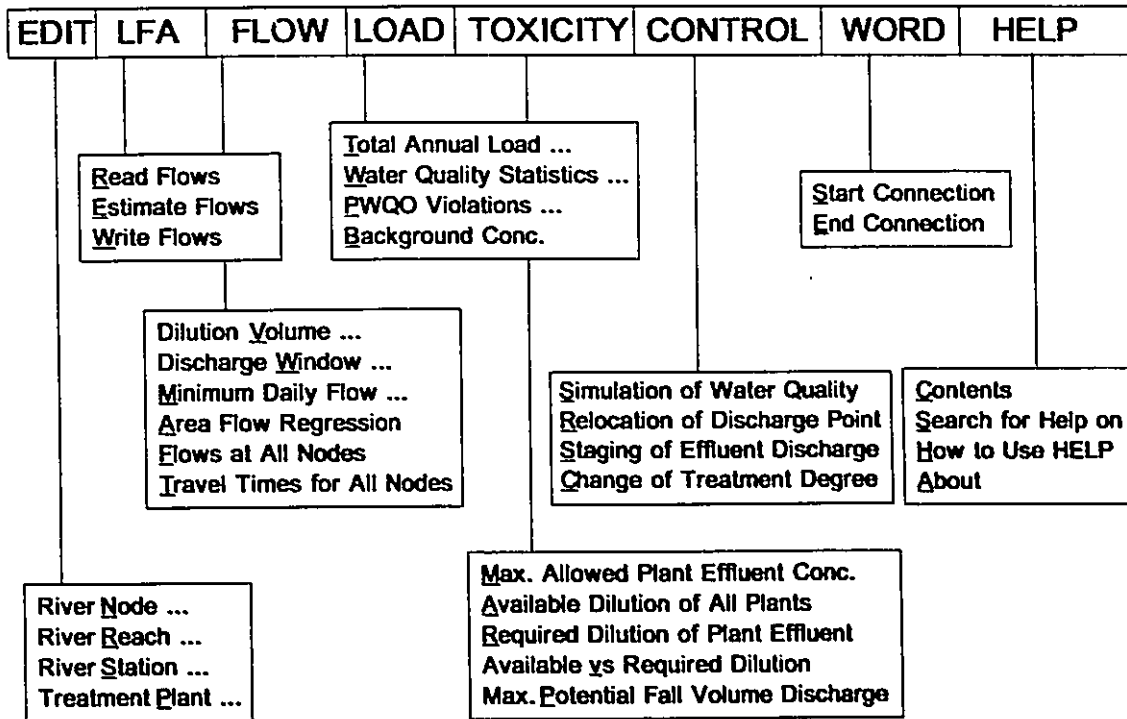


Figure 5.7: First level of menus in the software tool.

- A list with the names of all the parts of the river system along with its point-sources. The list consists sequentially of the treatment plants, the discharge, head and junction nodes, the reaches, and the river gauge stations. All the items in each of those groups are presented in alphabetical order and the whole list can be browsed.
- An area containing in text form information on the properties of each object that is also presented in the list.
- A display area (it is actually a scrolling window), where the system is graphically represented using different geometric patterns for each different kind of object. Thus the head nodes are represented as circles of colour dark cyan, with their name in them. The junction nodes as circles of the same colour but of smaller diameter with the node name right and on the top of the circle. The reaches as dark blue thick lines with the reach name in the middle of the line. The treatment plants as green rectangles with the name in them. The discharge nodes as small green circles linked with the corresponding plants and with the name right and above the circle but in green colour. Finally, the river gauge stations are represented as small circles of cyan colour. The graphically present network of the above elements is clickable.

These three panes communicate with each other, so that whenever the user selects an element from the list or clicks once at an element in the display area, the related information is presented, and correspondingly the selected element in the display area changes its colour to dark pink or the clicked element is also selected in the list (Figure 5.8).

The configuration of a river system can be altered to any extent by activating

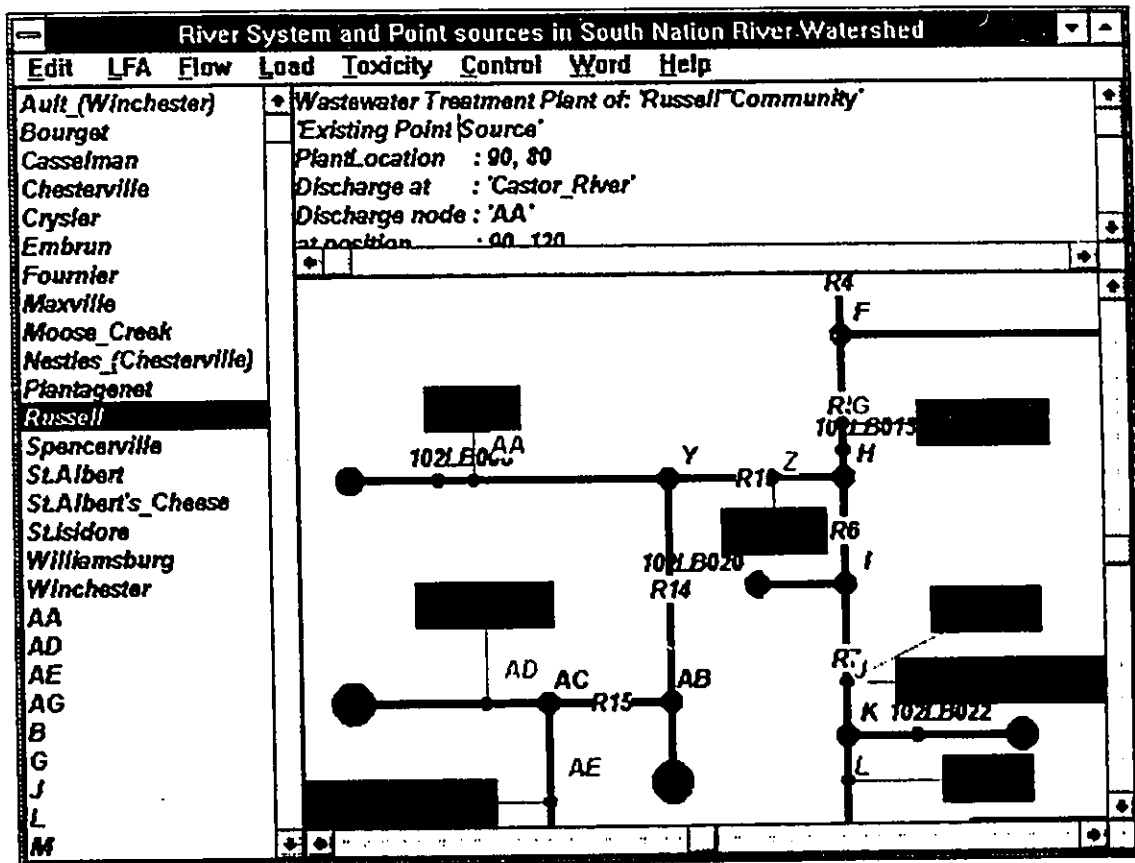


Figure 5.8: Communication between panes in the main window of the software tool.

the Edit menu on the top of the window. A detailed presentation of its options is shown in Figure 5.9. As can be seen an object of each kind of the above groups can be added or removed or its properties can be changed. The ability to change the structure of the network allows the interactive definition of the system configuration. More important it allows the update of the system representation in the window, after any reallocations that may be suggested from the optimization strategies of water quality. Before exiting, any changes that may have been made can be saved in a file specified by the user. This file can be the initial input file or a new one with a structure similar to the initial from which the application can be restarted.

The optimization strategies as well as any other step of the analysis can be performed by activating the corresponding options in the Flow, Load, Toxicity and Control menus that have been built in the interface. The sequence of steps that is followed in the actual analysis is also represented in the sequence of options in the above menus. The results of each of those steps are presented in detail so that the user can check them and derive any possible useful information from them.

The user can perform gradually all the steps that are involved in the analysis or ask for the results at any intermediate step of it. This is because the software identifies whether at any certain stage of the analysis all previous required steps have been performed. If this is the case, the user can select to continue the analysis with the current values for all variables. Alternatively, a backward procedure is followed and the user can specify the point after which all steps should be repeated and all variables should be re-estimated. This feature offers a high flexibility in the program and allows the testing of several different scenarios in a reduced total time, since only a user-specified part of the overall calculations would have to be re-estimated each time.

The parameters that are required for the execution of the steps of the analysis are asked from the user through prompters. These prompters are associated with a default value specified for each parameter with the appropriate format, thus they

EDIT

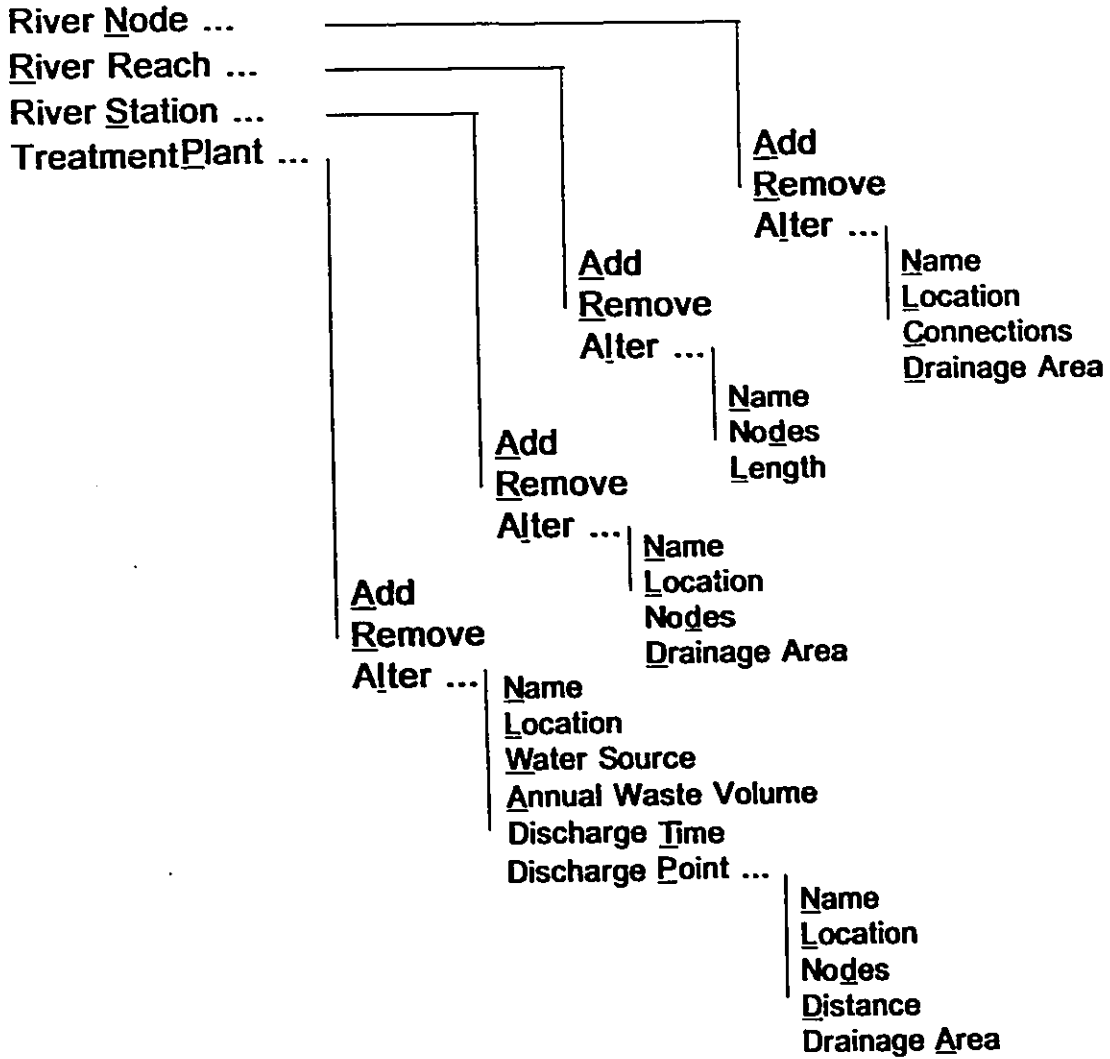


Figure 5.9: Levels of Edit menu in the software tool.

aid in the acceptance and wider use of the software. The input values are checked, and if they are not in the expected range, error messages, explanations, and suggestions are provided to the user through notification messages. In that way a more efficient use of the software is achieved and a continuity in the calculations can be ensured.

5.2.3 Presentation of Output Results

The results that are obtained throughout the analysis are presented in WORD in the form of custom-made tables which are embedded in a document. The *dynamic linking* of the software with WORD has been implemented and allows the activation of the latter from our application. The *Dynamic Data Exchange* between these two sources has been also used by defining a Client/Server relationship. The WordBasic language of WORD has been also used for the specification of the format in tables. The linking and the starting of conversation between the two sides is asked by the user with the activation of the corresponding option in the Word menu of the software. This action has to be made prior to the initialization of the overall analysis. Similarly at the end of the analysis the conversation between the two parts will have to be terminated.

On-line documentation has been introduced in the software tool with the purpose to aid in the more efficient use of it (Figure 5.10). In particular, this documentation will ultimately: (i) provide the user with information on the theoretical principles that have been applied within the software, (ii) elaborate on the technical points that will guide him in the interactive definition of the configuration of the river system, (iii) explain the options in the menus, (iv) specify the expected values by the user, and (v) provide default values. The addition of this feature in the software was considered as essential for the acceptance of the tool by the users and for the easier maintenance and future extension of the code.

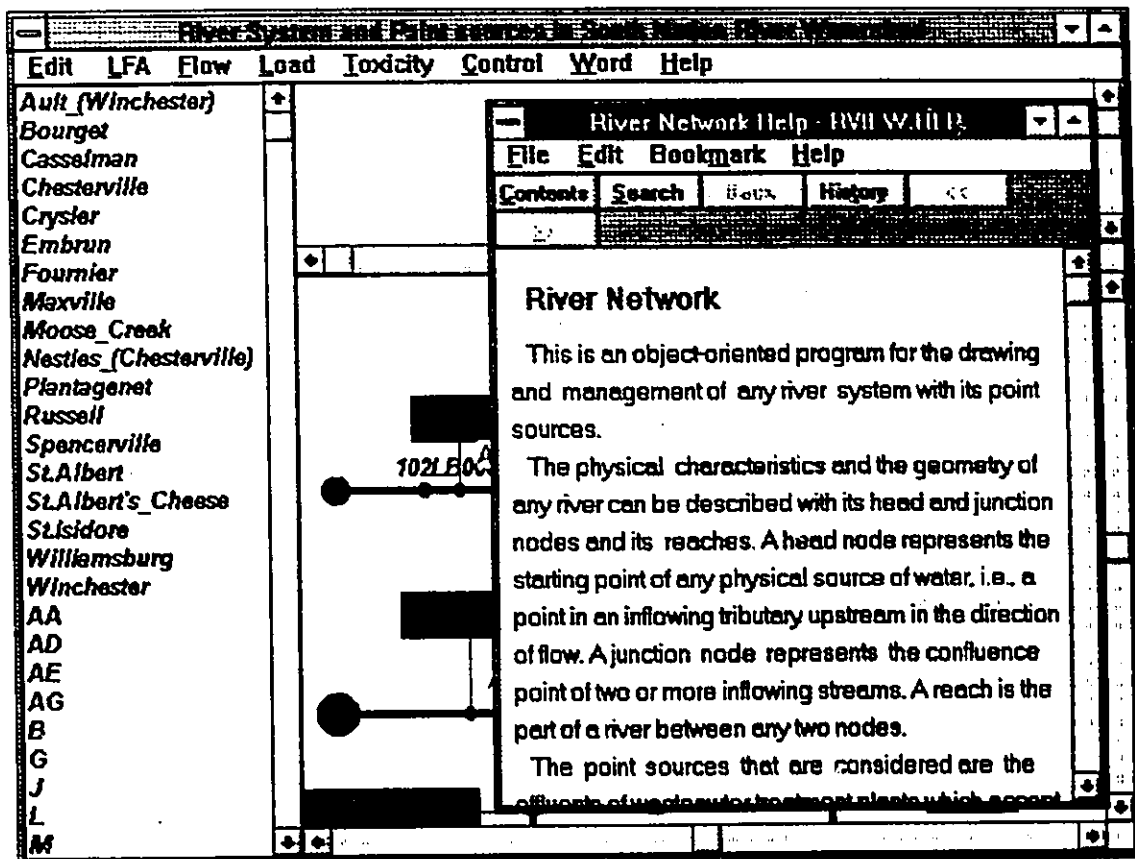


Figure 5.10: On-line documentation in the software tool.

Chapter 6

Case Study: The South Nation River

The application of the previous theories and developments to the case study of the South Nation River system will be discussed. The available data, the model parameters, and the obtained results will be presented.

6.1 The South Nation River Basin

The South Nation River is in the Province of Ontario in Canada. Its length is around 140 km and it has a drainage area of approximately 3900 km². The river rises from the region near the municipality of Donville and the St. Lawrence River and flows in a north-easterly direction, joined by the Hess Creek, the Payne River, the Castor river system, which includes the Henderson and Annable Drains, the Moose Creek and the Scotch River, up to the Paxton Creek near the community of Fournier. It then turns north-west up to the Ottawa River where it discharges (Figure 6.1). A dendrogram of the South Nation River system along with its point sources of pollutant load is presented in Figure 6.2 (Droste et al., 1992).

The river accepts the pollutant load that results from extensive agricultural and

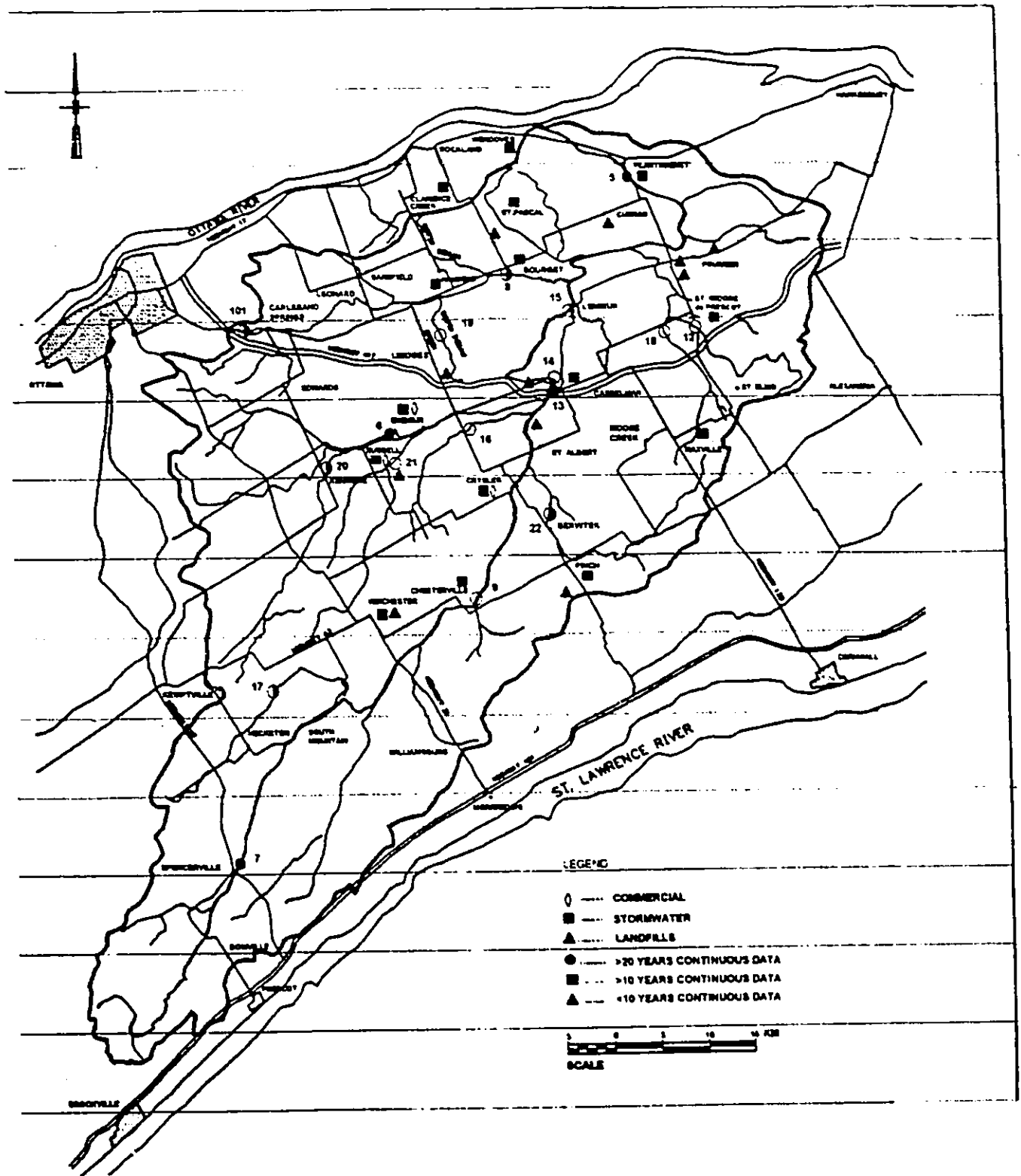


Figure 6.1: Map of the South Nation River Watershed (Droste and Gibbons, 1992).

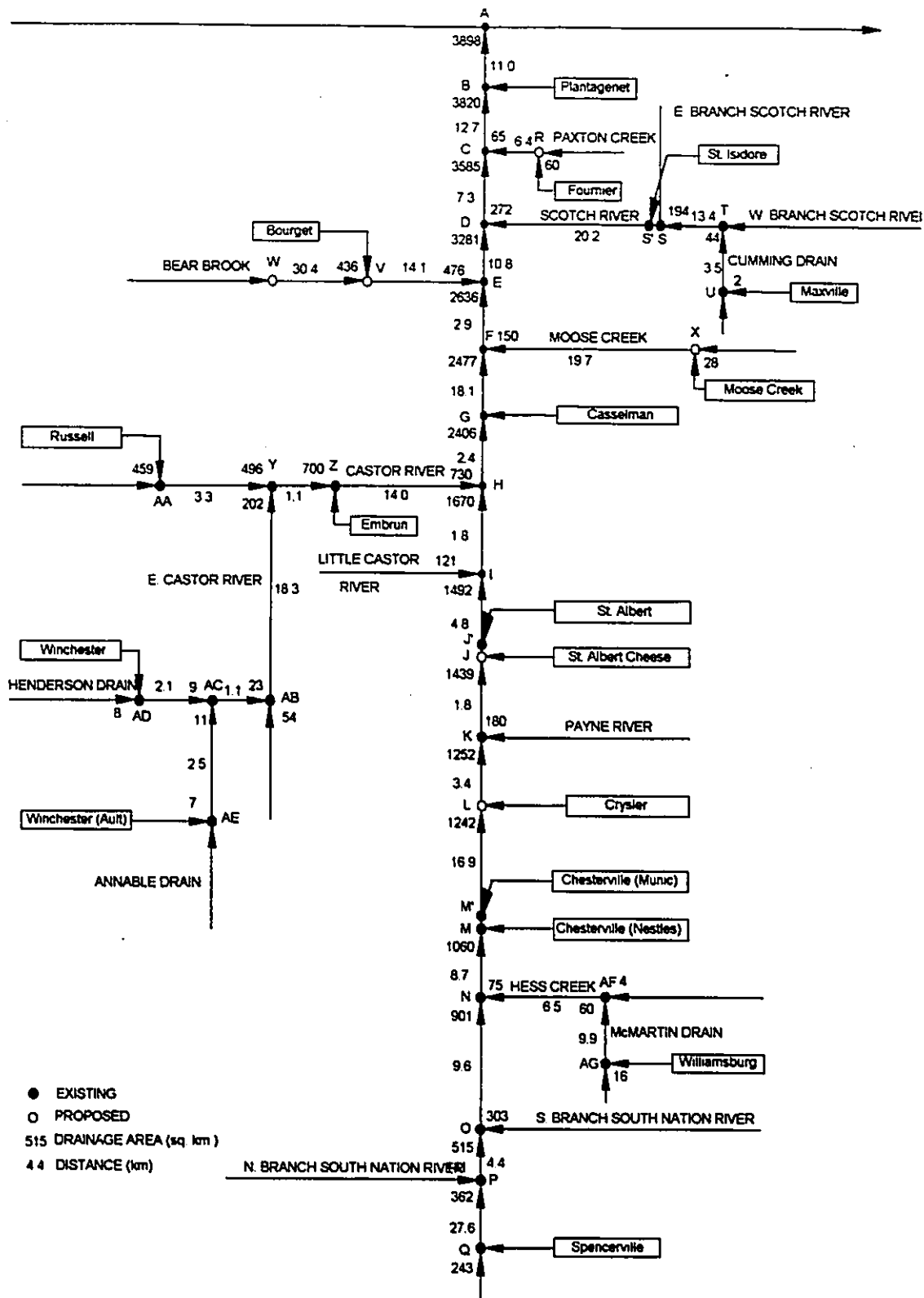


Figure 6.2: River Network in the South Nation River Watershed (Droste and Gibbons, 1992).

animal husbandry activities, from several municipal discharges as well as from a few industries effluent, that take place or are located within its watershed.

In particular the main nonpoint pollution sources of the agricultural land use and livestock grazing include erosion and runoff processes, malfunctioning septic tanks on farms, routing of untreated milkhouse wastes and liquid barnyard wastes to field tile drains, livestock access to streams and drains and manure spills, and runoff from manure piles. According to previous studies the resulting pollutant load constitutes the highest percentage of the total load.

The point source pollutant load from the municipalities wastewater is in most cases treated through storage in lagoons or directed to the river through septic tanks. The municipalities range in size in the watershed from 230 to 3500 people. Those that have treatment facilities and the corresponding rivers or streams to which they discharge are the ones of Casselman, Chesterville, Plantagenet and Spencerville on the South Nation River, of Embrun and Russell on the Castor River, of Maxville and St. Isidore on the West and East branch of the Scotch River, of Williamsburg to McMartin drain and of Winchester to Henderson drain. The municipalities that are expected to have sewage systems and treatment facilities and the corresponding rivers to which they discharge are those of Cryslar, Fournier and St. Albert on the South Nation River, Bourget in Bear Brook and Moose Creek in Moose Creek.

The effluent of the industries is treated in sophisticated, custom-designed treatment plants. The existing industries are Ault and Nestles which are located correspondingly in Winchester and Chesterville with discharges to Anabel drain and on the South Nation River, respectively. In the future, the St. Albert Cheese industry in St. Albert is proposed to install a treatment plant and discharge on the South Nation River.

Finally, there is a series of potential pollutant point sources of the South Nation River system, which include stormwater outfalls, landfill sites and commercial establishments.

The design of the existing and proposed sewer systems for the municipalities and the corresponding determination of the allowed operation and discharge conditions for the treatment plants are based on the design population size of each community and on the loading limits of the receiving streams. In 1990 some communities had already reached their design population size showing higher rates of growth from the estimated ones. In the future and in particular in the planning period of the following twenty years these rates are expected to decline. Most communities are expected to have an arithmetic or geometric growth rate which ranges in the watershed from 0.5 to 14 per annum and from 0.5% to 2% per annum, correspondingly. As an exception, the growth rates of Embrun and Russell are estimated on an arithmetic basis as 461 and 230 per annum and the growth rates of St. Albert and Casselman are estimated on a geometric basis as 7.6% and 5%, respectively.

So there was a need to evaluate the pollutant load in the river system at the new conditions, to consider the possible requirement of load redistribution or other control measures, and also to estimate the effects of additional loading on the assimilative capacity of the river. A study with the above objectives was required by the Ontario Ministry of Environment (MOE), and was undertaken at the time of reference by Gore and Storrie (Droste et al., 1992). The information that was collected during that study and the results that were obtained will be used in the present work for the validation of the code of the software tool developed.

6.2 Organization of Data - Requirements and Specifications

The quantity and quality measurements and the specifications on the operation of its treatment plants that are required for the accomplishment of this study are the

following:

1. Daily streamflow measurements from various flow gauge stations in the river system over a sequence of years. These correspond to the $q_{t,y}$ of eqs. (4.1) and (4.2) and are the primitive data for the low flow analysis. The existing river flow gauge stations in the South Nation River Watershed and their measurements were obtained from the Water Survey of Canada (WSC). The gauge stations whose flows were used in the present study, the corresponding streams and locations of them, finally the years of their record and their overall tributary drainage areas are presented in Appendix B, Table B.1.

2. Measurements of water quality from several sampling stations in the river system, over a sequence of years. These are used in the estimation of the background concentration of the modelled constituents in the river. The sampling stations of river water quality in the South Nation River Watershed and their measurements were obtained from MOE. A considerable number of measurements of the parameters of interest was available for the stations presented in Appendix B, Table B.2. Measurements of the river temperature and pH were further taken on a daily basis by the operators of the treatment plants at Plantagenet and Casselman, starting respectively from 1980 and 1986 up to 1991. This information was also accounted in the analysis.

3. Measurements of flowrate and of the water quality parameters of interest, in the effluent of all the discharging treatment plants in the watershed. This information is used in the simulation and optimization models and in the estimation of the point-source loads in the river system. In the watershed under study, the required data were available for all treatment facilities from MOE records.

Similar measurements in the influent of the plants and information on the features of the facilities can help in the evaluation of the appropriateness of the degree of treatment and in the estimation of the required cost for the treatment plant upgrade or change. The influent flowrate in a plant is also related with the population and the use of water in the community whose waste is treated, thus with the degree of growth of this community. A

part of the above data for the South Nation River Watershed is presented in Appendix B, Table B.3.

4. Standards of river and plant effluent water quality. They are used in the evaluation of water quality and are a basic constraint in all optimization models. In the present study the minimum acceptable quality was defined from the Provincial Water Quality Objectives, PWQO, set by MOE (Droste and Gibbons, 1992). These specify the in-stream limit of minimum DO saturation, which is set by considering the river as a warm water stream, and the non-lethal levels of un-ionized ammonia and un-dissociated hydrogen sulphide. They also specify the non-lethal levels of the two last constituents for in-pipe conditions of the plants effluent. The PWQO do not include a value for the maximum acceptable 5-day BOD concentration. The limit that was considered in the present study was based on Prati's classification system of surface water and the characterization of a slightly polluted stream (Droste and Gibbons, 1992). The above standards are presented in Table 6.1.

5. Constraints in the operation of each treatment plant, as specified in its corresponding certificate of approval. These constraints refer to the allowed loading rate, the effluent concentrations and the discharge periods of the plants. In the South Nation River Watershed, they are set by MOE and are required for a detailed evaluation of the effluent quality of each individual plant.

The discharge periods in particular are restricted for most of these point-pollution sources either in spring or in spring and late fall, due to the significantly low flows that are exhibited by the river in the summer and the start of the fall periods. Thus the time periods for the study of the South Nation River Watershed are defined in Table 6.2.

Table 6.1: The PWQO standards of river and plant effluent water quality (Droste and Gibbons, 1992).

Conditions	In-Stream					In-Pipe Effluent	
Parameter	<i>DO</i>	<i>BOD₅</i>	<i>BOD_u</i>	<i>(NH₃)_u</i>	<i>(H₂S)_u</i>	<i>(NH₃)_u</i>	<i>(H₂S)_u</i>
Standard (mg/l)	4.0	6.0	8.78	0.02	0.002	0.1	0.02
%	47% <i>DO_s</i>						

Table 6.2: Study Periods of Water Quality in the South Nation River Watershed (Droste and Gibbons, 1992).

Index	Code	Starting Date - Ending Date
1	Spring	March 1 - June 30
2	Fall	September 1 - December 30

6. The expected effluent concentrations for various treatment processes. These are required for the solution of the simulation and optimization models and the testing of changes or upgrades to the existing facilities as a means to improve the receiving water quality. The expected effluent concentrations for the facilities considered in this study and for the industries of the watershed are presented in Table 6.3. Additionally, the expected effluent DO concentration from the lagoons and the industries is 4.0 mg/l.

Table 6.3: Expected effluent concentrations from the treatment processes under consideration in the South Nation River Watershed (Droste and Gibbons, 1992).

Process	[BOD_5] (mg/l)	[$(NH_3)_{tot}$] (mg/l)	[$(H_2S)_{tot}$] (mg/l)	pH
Lagoon (Spring Discharge)	30.0	April: 14.0 May: 6.0	April: 3.0 May: 1.0	8.5
Lagoon (Fall Discharge)	20.0	2.0	0.0	8.5
New Hamburg	2.4	4.0	0.0	8.5
Extended Aeration, Activated Sludge	15.0	Winter: 5.0 Summer: 1.2	0.0	6-8
Ault Industry	5.0	6.1	-	-
Nestles Industry	15.0	6.1	-	-
St. Albert's Cheese Industry	15.0	6.1	-	-

6.3 Daily 7-Day and 30-Day, 20-Yr. Low Flows

The daily 7-day and 30-day, 20-yr. low flows $Q_{t,low}^7$ and $Q_{t,low}^{30}$ were estimated for all river gauge stations presented in Appendix B, Table B.1 and for the two study periods (shown in Table 6.2) using the procedure that was developed in section 4.1.1. The flows $Q_{t,y}^7$ and $Q_{t,y}^{30}$ that were estimated for each day t in the study periods over the sequence of years of record were assumed to be random, independent, homogeneous and stationary.

The results of the analysis for the station 02LB005 at Plantagenet are presented in Figure 6.3. It should be noted that this station is the furthest downstream in the river system, so it has the highest flows among all the other stations. However, the remarks that will be discussed below were found to be applicable to all other stations of the river system as well.

As can be seen, the daily $Q_{t,y}^7$ and $Q_{t,y}^{30}$ flows in spring are significantly higher, around one order of magnitude, than the corresponding flows in fall. This implies that the assimilative capacity of the river is higher in the spring, so this period should be studied in more detail for its appropriateness for the plants discharge.

It can be also seen that the daily $Q_{t,low}^7$ flows in spring are almost half of the corresponding daily $Q_{t,low}^{30}$ flows. Similarly in fall they are one third to one fourth of the $Q_{t,low}^{30}$ flows. Thus, an analysis based on the $Q_{t,low}^7$ conditions would result in more conservative estimates of the river water quality and would require the application of a more costly wastewater treatment scheme in the watershed in order to sustain the water quality above the specified standards.

However, the selection of the appropriate duration and recurrence interval for the analysis is a subject of further research and judgment, especially when the estimated treatment cost for certain flow conditions is considered very high in relation to the loss associated with violation of the standards (Loucks in Dorfman et al., 1972). In this

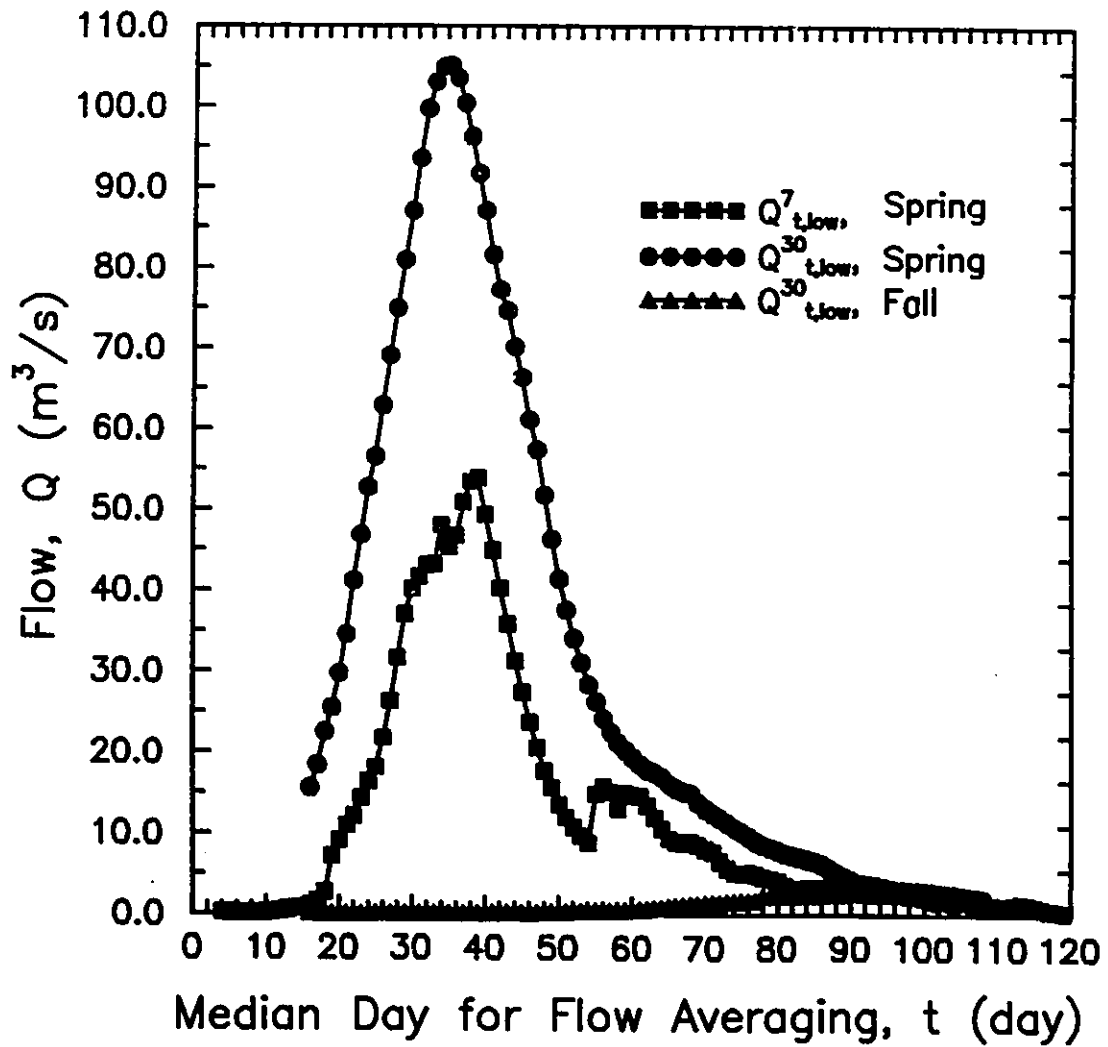


Figure 6.3: Daily low flows (of an average period of 7 and 30 days and a recurrence interval of 20 years) at the gauge station 02LB005 of Plantagenet in study periods 1 and 2.

study the evaluation of water quality control methods was based on both 7-day and 30-day flows.

6.4 Optimum Discharge Windows

From the estimated daily $Q_{t,low}^7$ and $Q_{t,low}^{30}$ flows, the corresponding optimum discharge windows for the site of each river gauge station in each study period were estimated, following the approach that was presented in section 4.1.2. Note that for the estimation of $Q_{min,low}^7$ and $Q_{min,low}^{30}$ flows, eq. (4.7) was applied for eight equal subintervals of each study period.

The application of eqs. (4.6) to (4.9) for the station 02LB005 and for $Q_{t,low}^{30}$ flows is graphically shown in Figures 6.4 and 6.5. The detailed results for all stations are given in Appendix B, Tables B.4, B.5, B.6 and B.7. The optimum minimum, average and maximum discharge windows for the whole river system were finally estimated by using eqs. (4.10) to (4.12) and are presented in Tables 6.4 and 6.5.

It should be noted that the optimum windows of station 02LB020 for $Q_{t,low}^7$ and $Q_{t,low}^{30}$ flows respectively, in the study period of fall, cover almost the whole study period and are wider than the ones of the other stations for more than a month. Their extent results from the variation of $Q_{t,low}^7$ and $Q_{t,low}^{30}$ flows at the site of station 02LB020 over the second study period. The latter is not represented by a "hydrograph type" curve with a rising part, a peak and a recession part, but has an irregular shape with many peaks of considerable height. Thus the above windows were not considered as representative of the conditions in the whole river system and were not accounted for in the estimation of overall optimum windows for the fall period.

The $Q_{t,low}^7$ or $Q_{t,low}^{30}$ flows of other gauge stations presented variations of the above type as well. Such a case is shown in Figure 6.3 where the spring $Q_{t,low}^7$ flows of

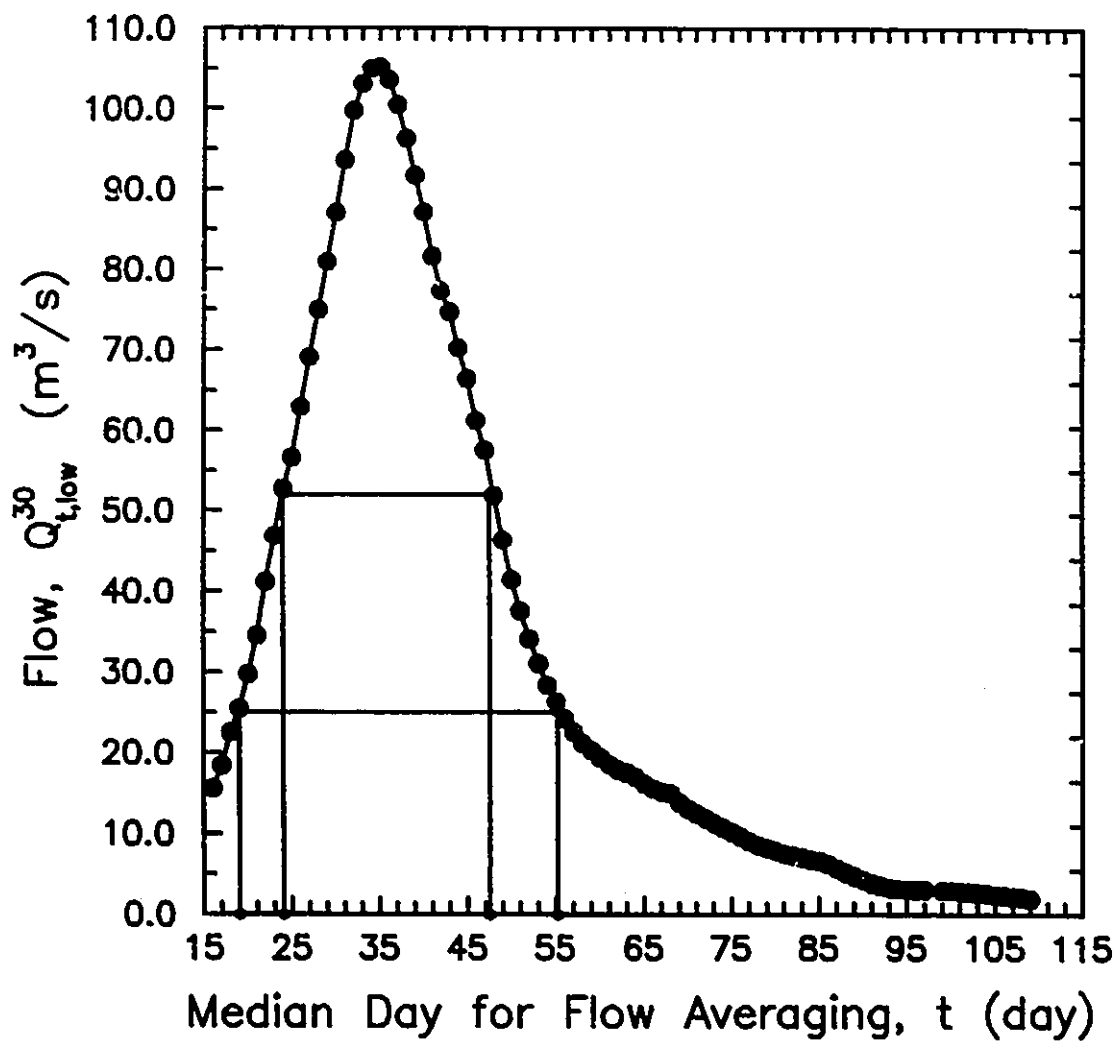


Figure 6.4: Estimation of the $Q_{min,low}^{30}$ flows for various windows at the gauge station 02LB005 of Plantagenet based on the $Q_{t,low}^{30}$ daily low flows of the station in study period 1.

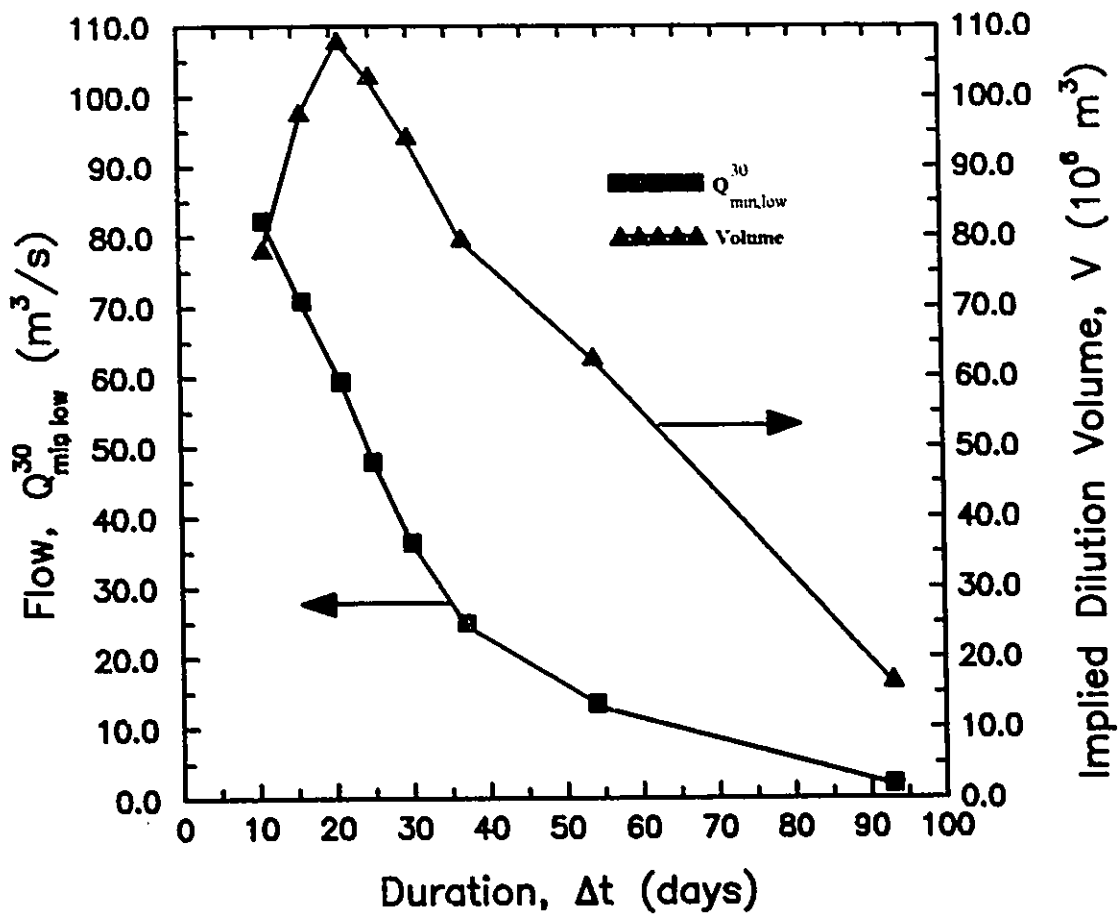


Figure 6.5: Estimation of the optimum discharge window at the gauge station 02LB005 of Plantagenet based on the $Q_{t,low}^{30}$ daily low flows of the station in study period 1.

02LB005 station present a second peak of small height for the median days in the end of April. However due to the small extent and magnitude of those variations the corresponding optimum windows of the stations were comparable to the windows of the other stations in the river system. Hence they were included in the estimation of overall optimum windows.

Table 6.4: The optimum discharge windows for $Q_{t,low}^7$ and $Q_{t,low}^{30}$ flows in the spring study period.

Window	Starting Date - Ending Date	
	daily 7-day, 20-yr. flows	daily 30-day , 20-yr. flows
Minimum	March 28 - April 13	March 28 - April 14
Average	March 24 - April 26	March 20 - April 19
Maximum	March 19 - May 10	March 16 - April 24

Table 6.5: The optimum discharge windows for $Q_{t,low}^7$ and $Q_{t,low}^{30}$ flows in the fall study period.

Window	Starting Date - Ending Date	
	daily 7-day, 20-yr. flows	daily 30-day , 20-yr. flows
Minimum	December 11 - December 13	November 27 - December 12
Average	November 18 - December 20	November 16 - December 16
Maximum	November 9 - December 28	November 4 - December 17

6.5 Minimum Daily Flows - Regional Analysis

Based on the daily $Q_{t,low}^7$ and $Q_{t,low}^{30}$ flows of both study periods and the optimum discharge windows for the whole river system, the corresponding minimum daily flows of all river gauge stations were estimated using eq. (4.13). The graphical application of this equation is shown in Figure 6.6 for the case of $Q_{t,low}^{30}$ conditions of station 02LB005 in study period 1. As can be seen the value of minimum daily flow increases inversely to the duration of the optimum discharge window.

The results of the analysis for all stations are presented in Appendix B, Tables B.8 to B.11. A regression analysis between these flows and the drainage area of the gauge stations was then performed as suggested in eq. (4.14). The regression coefficients and the corresponding correlation coefficients for all study periods, types of flow and windows are summarized in Table 6.6. In agreement with the behaviour of minimum daily flows, the regression coefficients increase as we move from the fall to spring study period, from 7-day to 30-day low flow conditions, and from the maximum to average and minimum optimum discharge window. The two last observations are graphically shown in Figure 6.7, which refers to study period 1.

The correlation coefficients in the regression analysis were generally higher than 0.9. In those cases it was assumed that the corresponding minimum daily flow at any node with known upstream drainage area could be adequately estimated using eq. (4.14). In the opposite case, like under the 7-day low flow conditions in the fall study period, the minimum daily flows were estimated using eq. (4.15) and the flows of the most downstream station in the river system, station 02LB005. This station was selected due to the higher flows it should have with respect to those of upstream stations and due to the availability of a more complete set of measured daily flows. By applying appropriately eqs. (4.14) and (4.15), the minimum daily flow at all junction nodes and discharge points in both study periods were estimated and are presented in Appendix B, Tables B.12 and

B.13. The variation of these flows along the South Nation River nodes for minimum window conditions are also shown in Figure 6.8. In the same figure, the contribution of the Castor River in the South Nation River flows is distinguished through a rise in the minimum daily flows at a distance of 65 km from node A.

6.6 Travel Times for the River System

The travel times of the river water body from any junction node or discharge point in the river system to its subsequent node were estimated for both study periods and all optimum discharge periods using eqs. (4.16) and (4.17). The flowrates considered were the estimated $Q_{min,daily}^7$ and $Q_{min,daily}^{30}$ flows at the starting node of each reach, so they represented the flow of water in absence of the point sources discharge. The total travel time from each node to the most downstream node A in Ottawa river for the above conditions are presented in Tables 6.7 and 6.8. As expected, the lower river flows of the fall study period of the higher duration windows and of the of 7-day low flows conditions result in higher travel times in the river system. The travel times along the South Nation River for average window conditions are shown in Figure 6.9.

It should be noted that the present results are crude estimates of the travel times in the river system. In order to reach more valid and representative estimates, detailed information on the channels geometry should be available, and detailed hydrodynamic models like HEC should be used (Droste and Gibbons, 1992).

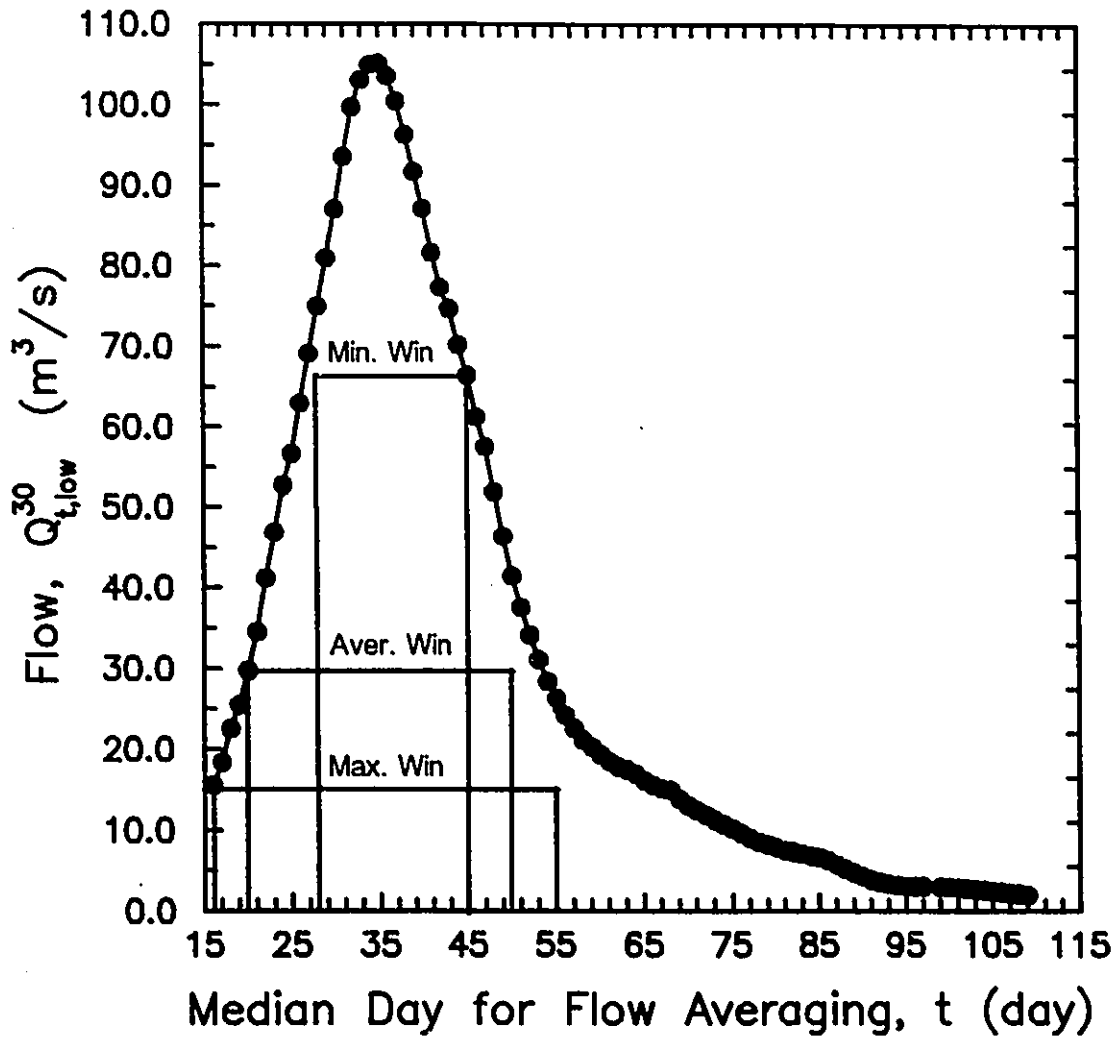


Figure 6.6: Estimation of the $Q_{min,daily}^{30}$ minimum daily flows at the gauge station 02LB005 of Plantagenet during all optimum discharge windows of study period 1 based on the $Q_{t,low}^{30}$ daily low flows.

Table 6.6: Regression coefficients and correlation coefficients in the minimum daily flow-drainage area relation for all types of flow, windows and study periods. The values in parentheses are the correlation coefficients.

Window	<i>const. (m³/s/km²) (r)</i>			
	Study Period 1		Study Period 2	
	<i>Q_{min,daily}⁷</i>	<i>Q_{min,daily}³⁰</i>	<i>Q_{min,daily}⁷</i>	<i>Q_{min,daily}³⁰</i>
Minimum	0.00770 (0.994)	0.01561 (0.979)	0.00037 (0.836)	0.00063 (0.963)
Average	0.00382 (0.998)	0.00821 (0.993)	0.00020 (0.642)	0.00055 (0.943)
Maximum	0.00193 (0.995)	0.00519 (0.914)	0.00010 (0.479)	0.00026 (0.921)

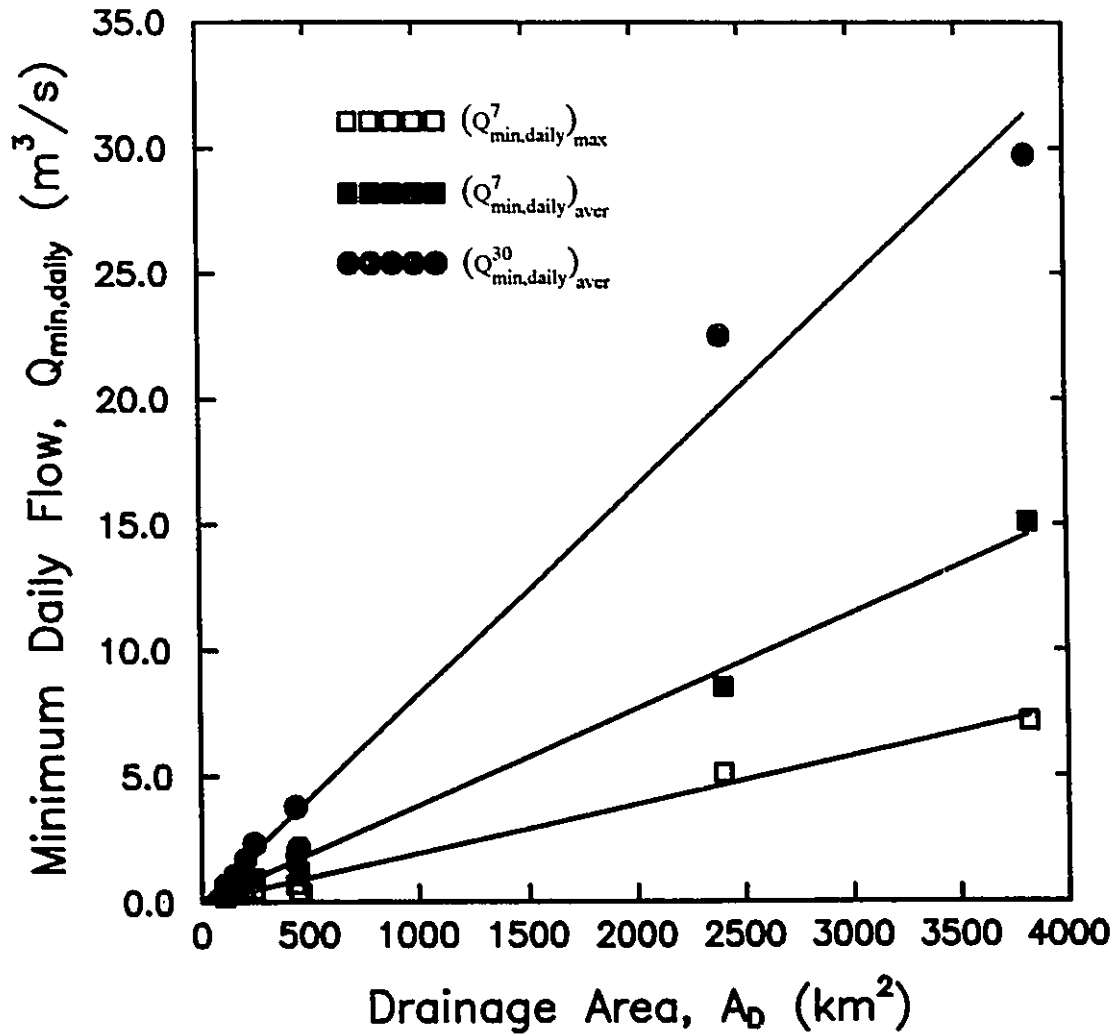


Figure 6.7: Results of the regression analysis between the minimum daily flows and the drainage area of the river gauge stations for river flows of different averaging intervals (7 and 30 days) and during different windows (maximum and average) in study period 1.

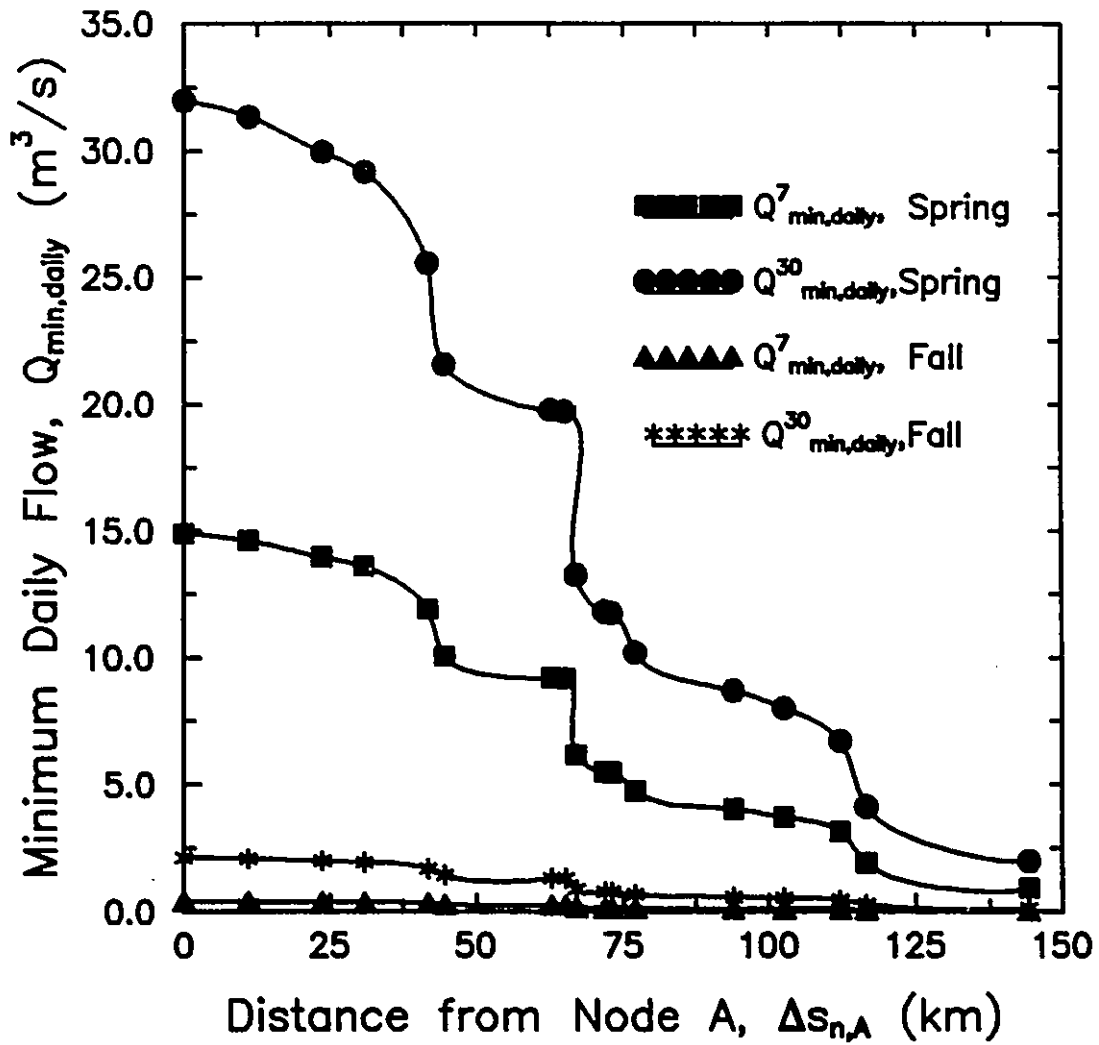


Figure 6.8: Estimated minimum daily flows $Q^7_{min,daily}$ and $Q^{30}_{min,daily}$ at all nodes in the South Nation River during the average optimum windows of study periods 1 and 2.

Table 6.7: Travel time from all junction nodes (jn) and discharge points (dp) of the river system to node A of Ottawa river during all optimum windows in study period 1.

Node	Type	Travel Time (days)			Travel Time (days)		
		$Q_{min.daily}^7$			$Q_{min.daily}^{30}$		
		Min.	Aver.	Max.	Min.	Aver.	Max.
A	jn	0.00	0.00	0.00	0.00	0.00	0.00
B	dp	0.53	0.70	0.92	0.40	0.52	0.62
C	jn	1.15	1.53	2.01	0.87	1.13	1.35
D	jn	1.52	2.01	2.64	1.14	1.48	1.78
E	jn	2.08	2.76	3.62	1.57	2.03	2.44
F	jn	2.25	2.97	3.91	1.69	2.19	2.63
G	dp	3.30	4.36	5.74	2.49	3.21	3.86
H	jn	3.44	4.55	5.98	2.59	3.35	4.03
I	jn	3.56	4.71	6.19	2.68	3.47	4.17
J	dp	3.89	5.16	6.77	2.94	3.80	4.56
J	dp	3.90	5.16	6.79	2.94	3.80	4.57
K	jn	4.03	5.33	7.01	3.04	3.93	4.72
L	dp	4.29	5.68	7.46	3.23	4.18	5.02
M	dp	5.64	7.47	9.81	4.25	5.50	6.61
M	dp	5.65	7.48	9.83	4.26	5.51	6.62
N	jn	6.37	8.44	11.09	4.81	6.21	7.47
O	jn	7.23	9.57	12.58	5.45	7.05	8.47
P	jn	7.71	10.21	13.41	5.81	7.52	9.03
Q	dp	11.72	15.51	20.39	8.84	11.43	13.73
U	dp	12.05	15.95	20.96	9.08	11.75	14.11
T	jn	8.58	11.36	14.92	6.47	8.36	10.05
S'	dp	4.71	6.23	8.19	3.55	4.59	5.51
S	jn	4.72	6.25	8.21	3.56	4.60	5.53
AD	dp	10.71	14.18	18.63	8.08	10.44	12.55
AE	dp	11.02	14.59	19.17	8.31	10.74	12.91
AC	jn	9.52	12.60	16.56	7.18	9.28	11.15
AB	jn	9.08	12.02	15.80	6.85	8.86	10.64
AA	dp	5.24	6.94	9.12	3.95	5.11	6.14
Y	jn	4.87	6.45	8.48	3.67	4.75	5.71
Z	dp	4.77	6.31	8.30	3.60	4.65	5.59
AG	dp	12.25	16.22	21.32	9.24	11.95	14.35
AF	jn	7.98	10.57	13.89	6.02	7.78	9.35
R	dp	2.78	3.68	4.84	2.10	2.71	3.26
V	dp	3.70	4.90	6.44	2.79	3.61	4.34
X	dp	9.04	11.96	15.72	6.81	8.81	10.59

Table 6.8: Travel time from all junction nodes (jn) and discharge points (dp) of the river system to node A of Ottawa river during all optimum windows in study period 2.

Node	Type	Travel Time (days)			Travel Time (days)		
		$Q_{min.daily}^7$			$Q_{min.daily}^{30}$		
		Min.	Aver.	Max.	Min.	Aver.	Max.
A	jn	0.00	0.00	0.00	0.00	0.00	0.00
B	dp	2.03	2.96	4.64	1.45	1.53	2.06
C	jn	4.41	6.43	10.10	3.14	3.33	4.49
D	jn	5.79	8.45	13.27	4.13	4.37	5.90
E	jn	7.95	11.60	18.21	5.67	6.00	8.10
F	jn	8.57	12.51	19.64	6.11	6.47	8.73
G	dp	12.58	18.36	28.82	8.98	9.50	12.82
H	jn	13.11	19.14	30.04	9.35	9.90	13.36
I	jn	13.58	19.82	31.11	9.69	10.25	13.84
J'	dp	14.86	21.69	34.04	10.60	11.22	15.14
J	dp	14.89	21.73	34.11	10.62	11.24	15.17
K	jn	15.38	22.44	35.23	10.97	11.61	15.67
L	dp	16.36	23.87	37.48	11.67	12.35	16.67
M'	dp	21.53	31.41	49.31	15.36	16.25	21.93
M	dp	21.56	31.46	49.38	15.38	16.27	21.96
N	jn	24.32	35.49	55.72	17.35	18.36	24.77
O	jn	27.60	40.27	63.22	19.69	20.83	28.11
P	jn	29.42	42.93	67.40	30.99	22.21	29.70
Q	dp	44.72	65.26	102.45	31.90	33.76	45.55
U	dp	45.97	67.08	105.31	32.79	34.70	46.83
T	jn	32.74	47.77	74.99	23.35	24.71	33.35
S'	dp	17.96	26.21	41.14	12.81	13.56	18.29
S	jn	18.02	26.30	41.28	12.86	13.60	18.36
AD	dp	40.88	59.65	93.65	29.16	30.86	41.64
AE	dp	42.05	61.36	96.32	29.99	31.74	42.83
AC	jn	36.32	53.00	83.20	25.91	27.42	36.99
AB	jn	34.67	50.58	79.41	24.73	26.17	35.31
AA	dp	20.02	29.21	45.85	14.28	15.11	20.39
Y	jn	18.60	27.14	42.60	13.27	14.04	18.94
Z	dp	18.20	26.55	41.69	12.98	13.74	18.54
AG	dp	46.76	68.24	107.12	33.35	35.30	47.63
AF	jn	30.47	44.46	69.79	21.73	23.00	31.03
R	dp	10.62	15.49	24.32	7.57	8.01	10.81
V	dp	14.14	20.63	32.39	10.09	10.67	14.40
X	dp	34.49	50.33	79.02	24.60	26.04	35.13

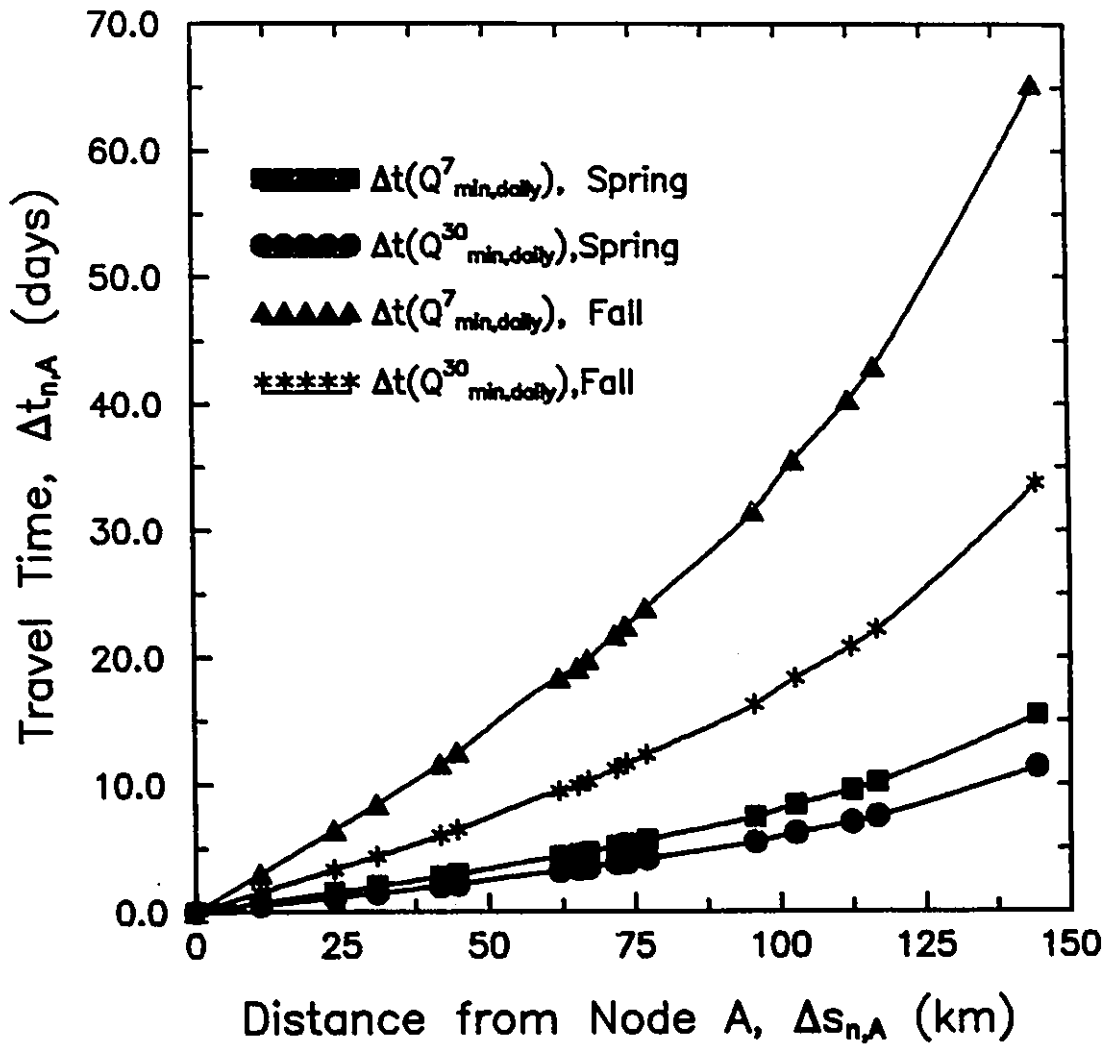


Figure 6.9: Estimated travel times from each node in the South Nation River to node A in Ottawa River with the $Q^7_{min,daily}$ and $Q^{30}_{min,daily}$ river flows during the average optimum windows of study periods 1 and 2.

6.7 Background Concentration of Modelled Constituents

A simple statistical analysis was performed on the water quality measurements of each sampling station in the river system during the months of optimum discharge windows. It included the calculation of the minimum, average and maximum values, as well as of the 25, 50 and 75 percentile values of the parameters of interest. The results of the analysis for the modelled constituents, the temperature and the pH in the data obtained during the months of March to May and October to December in years 1980 to 1991 are presented in Appendix B, Tables B.14 and B.15. In particular, the 75 percentile of 5-day BOD and of ammonia nitrogen and the 25 percentile value of the DO in each station were considered as reasonable and conservative estimates of the background concentration of the constituents in the location of the corresponding stations in the system (Droste et al., 1992). The average of those "local" percentile values over the stations was assumed to represent an estimate of the background concentrations in the whole river system for both study periods of spring and fall. The concentrations that were obtained with the above approach are given in Table 6.9. It is noted that the ultimate BOD background concentration in the Table was obtained from the corresponding 5-day BOD value using eq. (4.24) with $k_{r,lab}$ equal to 0.23 d^{-1} (as suggested in section 4.2.1.1). Finally, the concentration of $(H_2S)_{tot}$ was assumed to be equal to zero due to the absence of any available measurements from the river sampling stations.

Table 6.9: Background concentrations of the modelled constituents in the river.

Constituent	DO	BOD ₅	BOD _u	(NH ₃ -N) _{tot}	(NH ₃) _{tot}	(H ₂ S) _{tot}
<i>C_b</i> (mg/l)	8.707	2.348	3.436	0.095	0.115	(0.000)

6.8 Toxicity and Dilution Requirements - Potential for Spring and Fall Discharge

A preliminary analysis for the identification of sites that require quality control was presented in part A of paragraph 4.4. This analysis was based on the standards of the toxic $(NH_3)_u$ and $(H_2S)_u$ compounds and the dilution requirements. It considered the discharge of each treatment plant effluent distinctly in absence of the discharge from other plants. It also considered the expected changes in the discharge conditions within the planning period of twenty years. More specifically it accounted for the expected communities population and the annual effluent volume of the treatment plants in years 1991 and 2011. The related information was given in section 6.1 and Table B.3 of Appendix B. The analysis implementation in the South Nation River Watershed along with the results obtained are presented in the following:

A1. The ability of the wastewater treatment process under consideration (Table 6.3) to provide an effluent quality that meets the in-pipe toxicity standards was evaluated from the effluent stream conditions using eqs. (4.42) and (4.43). The analysis was performed for all months of the optimum discharge windows.

The effluents of the treatment plants were assumed to be at the same temperature as the receiving water body. The values of river temperature that were accounted for were the daily values monitored at the locations of Plantagenet and Casselman by the operators of the corresponding plants. The minimum of the 25 percentile values and the maximum of the 75 percentile values that were obtained from the statistical analysis in the two river "samples" for each month of discharge were considered to give a representative range of the river system and thus of the plant's effluent temperature.

The effluent pHs were considered to be in the range of 7 to 8. For treatment in lagoons, the lower values were expected in the winter and early spring while the higher values were expected in the late spring and the fall. Generally, higher pH values in the

lagoons are related to intensive algae growth rates that occur under favorable conditions of temperature, radiation and nutrients, and result in the removal of the CO_2 of the water (Peavy et al., 1985). This influence of algae activity on pH is stronger when the storage time of the wastewater in the lagoon is high. In that case the treatment system becomes equivalent to a maturation pond, and the pH can rise above 10. However, in the South Nation River Watershed the relatively low fall temperatures were expected to retain pH values below 8.

Using the above ranges of values and the in-pipe standards presented in Table 6.1, the maximum allowable concentrations of the measured $(NH_3)_{tot}$ and $(H_2S)_{tot}$ in the effluents from the plants were estimated and are shown in Table 6.10. A comparison of these limiting values with the expected concentrations in the effluents of the treatment processes shows that in the spring period the discharge of lagoon effluents would result in violation of the $(H_2S)_u$ standards under any pH conditions and also in violation of the $(NH_3)_u$ standards if the pH was greater than 7. As a result, a post-aeration cell would be required to strip and oxidize the $(H_2S)_u$ and ensure the prevention of its non-lethal effects.

The discharge of lagoons in the fall period as well as the direct discharge from treatment plants that apply the New Hamburg or extended aeration process would satisfy both the $(H_2S)_u$ and $(NH_3)_u$ standards. However, in the South Nation River Watershed the effluent of any plant would have to be stored in ponds. Thus its discharge in the spring period could result in violation of the $(NH_3)_u$ non-lethal standards, if the pH in pipe conditions would reach or exceed the value of eight.

Table 6.10: Maximum allowable concentrations of $(NH_3)_{tot}$ and $(H_2S)_{tot}$ in the plants effluent in order to meet the in-pipe effluent standards of $(NH_3)_u$ (0.1 mg/l) and of $(H_2S)_u$ (0.02 mg/l).

Month	Temp (°C)	pH	$(NH_3)_{tot}$ (mg/l)	$(H_2S)_{tot}$ (mg/l)
March	1.00	7.00	109.24	0.03
	1.00	8.00	11.01	0.11
	3.00	7.00	92.33	0.03
	3.00	8.00	9.32	0.12
April	6.00	7.00	72.95	0.03
	6.00	8.00	7.39	0.13
	10.00	7.00	54.08	0.03
	10.00	8.00	5.50	0.14
May	13.00	7.00	43.51	0.03
	13.00	8.00	4.44	0.16
	17.00	7.00	32.76	0.04
	17.00	8.00	3.37	0.17
October	10.00	7.00	54.08	0.03
	10.00	8.00	5.50	0.14
	13.00	7.00	43.51	0.03
	13.00	8.00	4.44	0.16
November	4.00	7.00	85.22	0.03
	4.00	8.00	8.61	0.12
	7.00	7.00	67.61	0.03
	7.00	8.00	6.85	0.13
December	1.00	7.00	109.24	0.03
	1.00	8.00	11.01	0.11
	3.00	7.00	92.33	0.03
	3.00	8.00	9.32	0.12

A2. The available dilution factor was estimated at all discharge points in the river for both study periods using eqs. (4.44) and (4.38). The conditions that were considered included both $Q_{min,daily}^7$ and $Q_{min,daily}^{30}$ river flows (Tables B.12 and B.13) and the discharge of the total annual waste volume of the plants in years 1991 and 2011 (Table B.3) during all optimum discharge windows (Tables 6.4 and 6.5). The resulting estimates are presented in Appendix B, Tables B.16 to B.19.

Observing these estimates we note that under the spring $Q_{min,daily}^7$ river flows and discharge conditions of both 1991 and 2011, the higher values of the available dilution factor were obtained for discharge in the minimum and then for the average and maximum window. Under the same conditions in the fall, the dilution factor decreased as we moved from the average to minimum and maximum discharge windows. The same order of decrease was observed under the spring $Q_{min,daily}^{30}$ river flows and discharge conditions of both 1991 and 2011. Finally, the order of decrease for the corresponding fall conditions changed from the average to maximum and minimum windows. The above remarks could be useful in the optimization procedure of staging the effluent discharges.

From the results obtained it can be also observed that the available dilution under $Q_{min,daily}^{30}$ flows in spring was between 30 and 400 and was twice as high as the dilution under $Q_{min,daily}^7$ spring flows. The dilution factor under $Q_{min,daily}^{30}$ flows in fall was between 0 and 24 and was around nine times as high as the dilution under $Q_{min,daily}^7$ fall flows. Thus the dilution in spring could be one to two orders of magnitude higher than the dilution in fall, which reinforced the option of discharge in this period. Finally, the dilution factors under the discharge conditions of year 2011 were estimated as equal to one third or one fourth of the corresponding values under the discharge in year 1991. This ratio implied that the assimilative and dilution capacity of the river would be reduced in the future. It also remarked the need for a reduction in the pollutant load of the river in order to prevent the exertion of lethal effects in water use.

A3. The dilution that was required at each discharge point of the river system to meet the in-stream standards of the toxic $(NH_3)_u$ and $(H_2S)_u$ compounds was estimated for each month of discharge using eq. (4.45).

The river concentrations before mixing (C_r) were estimated from the background river concentrations in $(NH_3)_{tot}$ and $(H_2S)_{tot}$ (Table 6.9), after their transformation to concentrations of $(NH_3)_u$ and $(H_2S)_u$ through eqs. (4.42) and (4.43). The discharged wastewater concentrations (C_w), were estimated from the expected effluent concentrations from lagoons (Table 6.3), after their similar transformation to the concentration of undissociated toxic compounds. The ranges of temperature and pH considered were defined from the minimum between the two 25 percentile values and the maximum between the two 75 percentile values of temperature and pH that were obtained from the analysis of daily river data at the locations of Plantagenet and Casselman. The river concentrations after mixing (C_m), were set equal to the instream standards of $(NH_3)_u$ and $(H_2S)_u$ (Table 6.1).

The resulting values of required dilution are presented in Appendix B, Table B.20. As can be seen the dilution factor for the $(NH_3)_u$ standards was in the range of 0 to 32 for discharge in spring and in the range of 0 to 2 for discharge in the fall. However, the corresponding estimates for the $(H_2S)_u$ standards under conditions of spring discharge were one or two orders of magnitude higher. Hence the dilution factor was estimated in the range of 50 to 400 or of 140 to 1100, considering correspondingly the lower or higher expected concentrations of total hydrogen sulphide in the lagoon effluent.

The maximum required dilution factors for each month of discharge were then compared with the maximum (over the optimum discharge windows) available dilution factors for $Q_{min,daily}^7$ and $Q_{min,daily}^{30}$ flows for each study period and for the discharge conditions in years 1991 and 2011. The results of this comparison based on the compliance of river water quality with the $(NH_3)_u$ standards under conditions of spring discharge are presented in Table 6.11.

Table 6.11: Adequacy of the maximum available dilution factor at the locations of point-sources to supply the maximum required dilution for the compliance with the river standard of $(NH_3)_u$ (0.02 mg/l) under conditions of spring $Q_{min,daily}^7$ and $Q_{min,daily}^{30}$ flows and lagoons discharge in years 1991 and 2011.

Plant	1991 Conditions					2011 Conditions				
	D_{av}		$D_{av} > D_{req}$			D_{av}		$D_{av} > D_{req}$		
	$Q_{min,daily}^7$	$Q_{min,daily}^{30}$	Mar.	Apr.	May	$Q_{min,daily}^7$	$Q_{min,daily}^{30}$	Mar.	Apr.	May
		1.4	9.9	31.8			1.4	9.9	31.8	
Bourget	-	-	-	-	-	21.4	45.9	Y	Y	N7
Casselman	74.6	160.0	Y	Y	Y	28.8	61.8	Y	Y	N7
Chesterville	40.6	87.0	Y	Y	Y	37.7	80.8	Y	Y	Y
Crysler	-	-	-	-	-	64.2	137.7	Y	Y	Y
Embrun	16.7	35.8	Y	Y	N7	5.1	11.0	Y	N7	N
Fournier	-	-	-	-	-	13.4	28.7	Y	Y	N
Maxville	0.4	0.8	N	N	N	0.4	0.8	N	N	N
Moose Creek	-	-	-	-	-	3.1	6.7	Y	N	N
Plantagenet	192.0	411.7	Y	Y	Y	170.2	365.0	Y	Y	Y
Russell	26.9	57.7	Y	Y	N7	6.0	12.9	Y	N7	N
Spencerville	53.8	115.4	Y	Y	Y	39.7	85.0	Y	Y	Y
St. Albert	-	-	-	-	-	117.8	252.7	Y	Y	Y
St. Isidore	17.4	37.3	Y	Y	N7	12.5	26.8	Y	Y	N
Williamsburg	2.8	6.9	Y	N	N	2.6	5.5	Y	N	N
Winchester	0.1	0.3	N	N	N	0.1	0.3	N	N	N
Ault	0.4	0.9	Y	N	N	0.2	0.4	Y	N	N
St. Albert Cheese	-	-	-	-	-	719.6	1543.4	Y	Y	Y
Nestles	53.1	113.9	Y	Y	Y	53.1	113.9	Y	Y	Y

Y: The available dilution for $Q_{min,daily}^7$ and $Q_{min,daily}^{30}$ flows exceeds the required dilution.

N: The available dilution for $Q_{min,daily}^7$ and $Q_{min,daily}^{30}$ flows is smaller than the required dilution.

N7: The available dilution for $Q_{min,daily}^7$ flows is smaller than the required dilution.

As can be seen, the available dilution at the locations of effluent discharge from the plants of Maxville and Winchester was not sufficient to provide a water quality that would meet the PWQO standards, for any of the conditions considered. The available dilution at the discharge points of all other plants would meet the dilution requirements, if the plants discharged in March. However, if the plants discharged in April, then the dilution at the discharge points of the plants in Ault industry and Williamsburg community would result in violation of the standards. If the discharge period was transferred furthermore to a later interval in May, then additional violations would occur at the discharge points of the plants in Embrun, Russell and St. Isidore, under $Q_{min,daily}^7$ flows in year 1991 and under both $Q_{min,daily}^7$ and $Q_{min,daily}^{30}$ flows in year 2011.

A similar analysis as above was also performed in order to evaluate the adequacy of the river dilution to meet the PWQO $(H_2S)_u$ standard in spring. However, it was found that the dilution requirements at all discharge points in the river system were much higher than the available dilution. Thus an advanced treatment process or a post-aeration cell would be required by all plants that discharge in spring, in order to prevent the exertion of lethal effects to the users of river water.

Note that in the fall period the expected $(H_2S)_{tot}$ effluent concentration from a lagoon was zero, so there was not an expected deterioration of river quality with respect to the $(H_2S)_u$ levels. To estimate the river quality in the fall with respect to the $(NH_3)_u$ standards, and mainly to evaluate whether the plants, whose discharge in spring resulted in violation of these standards, could discharge a part of their waste volume in the fall, the following analysis was performed.

A4. The maximum potential waste volume of each point source that could be discharged in the fall period without violating the in-stream $(NH_3)_u$ standards was estimated for $Q_{min,daily}^7$ and $Q_{min,daily}^{30}$ river flows and during all optimum windows, by equating the available with the required dilution factor at each discharge point. The required dilution factor as well as the river flow and discharge window were calculated as

described in A2 and A3 but for fall conditions. The $(NH_3)_{tot}$ concentration in the lagoon effluents was considered as 4 mg/l and in the effluent from the treatment plants of the industries as 6.1 mg/l.

The results obtained for the discharge schemes of years 1991 and 2011 are presented in Appendix B, Tables B.21 to B.24.

Observing those values we note that the plants of the communities of Maxville and Winchester would not be allowed to discharge any part of their waste volume, so they would have to upgrade their degree of treatment or apply another control method. Similarly, the plants of Ault industry or of Williamsburg would be allowed to discharge a zero or practically zero percentage of their total waste volume, correspondingly. However, the plants at the communities of Embrun, Russell and St. Isidore could discharge during the average fall window of 1991, 12% to 20% and 60% up to 90% percentage of their annual volume under $Q_{min,daily}^7$ and $Q_{min,daily}^{30}$ river flow conditions, respectively. The corresponding percentages for the year 2011 were generally one-third of the above values. It is also noted that from the plants that were expected to operate after 1991, the ones at Chrysler and St. Albert Cheese could discharge a significant percentage and all of their annual waste volume respectively, in the fall period.

6.9 Water Quality Simulation for Present and Future Design Conditions

Simulations of water quality were performed for the river system of the South Nation River Watershed using the model described in sections 4.2.1. and 4.2.3. The variation of DO deficit, ultimate BOD and total ammonia along the river system was estimated distinctly for the duration of each optimum discharge window (Tables 6.4 and 6.5) in each study period of spring and fall (Table 6.2). The base river flow at any river

node was assumed to be equal to the corresponding minimum daily flow, $Q_{min,daily}^7$ and $Q_{min,daily}^{30}$, presented in Tables B.12 and B.13, while its concentrations in the modelled constituents were considered equal to the estimated background concentrations of Table 6.9. The effluent flowrate of each treatment plant was derived considering the discharge of the total annual waste volume of the plant within the optimum window and study period of each simulation.

From the parameters of the water quality model, the specific rates of BOD removal ($k_{r,20}$) and of total ammonia oxidation ($k_{n,20}$) at the reference temperature of 20°C were considered to be in the ranges of [0.05 0.50] d⁻¹ and [0.1 1.0] d⁻¹ as suggested in sections 4.2.1.1 and 4.2.1.2. The specific rate for reaeration ($k_{a,20}$) could be estimated from the formula of O'Connors and Dobbins (eq. 4.33). The flows in the river system, as shown in Tables B.12 and B.13, varied between 0.01 and 59 m³/s, and the corresponding velocities, estimated from eq. (4.17), varied between 0.01 and 0.32 m/s. The depth of the South Nation River varied between 0.6 and 1.3 m. The resulting values of $k_{a,20}$ were in the range of 0.1 to 2.1 d⁻¹. However a range of 0.8 to 4.0 d⁻¹ was considered as more representative for the river system under study (Droste and Gibbons, 1992), because it has been derived using more accurate estimates of the river velocity. The rate of benthic oxygen demand ($r_{bod,20}$) was considered significant only in simulations of the fall water quality and was estimated from eq. (4.31) using a range of 0.1 to 5.0 g/m²/d for the rate $k_{bod,20}$ and the variation of the South Nation River depth. The resulting values of $r_{bod,20}$ were in the range of 0.17 to 8.33 g/m³/d. The temperature correction factors of the above rates were assumed to be in the ranges suggested in sections 4.2.1.1 to 4.2.1.3, and in particular, were all set equal to the default values that are given in the manual of QUAL2E (Droste et al., 1992).

Based on the above ranges of the model parameters, two sets were formed and were used for the simulations and the application of quality control methods in the South Nation River system. These sets are presented in Table 6.12. The combination of the

parameter values in set 1 represented high rates of oxygen consumption and low rates of its replenishment. So their application in the given model was expected to result in the most conservative estimates of dissolved oxygen and BOD for the ranges of the parameters considered. Similarly, the parameters in set 2 represented low rates of nitrification that would allow a significant part of the total ammonia to be in its undissociated form. So they were expected to result in the most conservative estimates of water quality with respect to ammonia.

Table 6.12: Sets of values of the model parameters used in the simulations.

Set #	$k_{r,20}$ (d ⁻¹) ($\theta_r=1.047$)	$k_{n,20}$ (d ⁻¹) ($\theta_n=1.083$)	$k_{a,20}$ (d ⁻¹) ($\theta_a=1.024$)	$r_{bod,20}$ (mg/m ³ /d) ($\theta_{bod}=1.072$)
1	0.50	1.0	0.8	0.0 (Spring) 8.3 (Fall)
2	0.50	0.1	0.8	0.0 (Spring) 8.3 (Fall)

The evaluation of the river water quality that was estimated by the model was based on the in-stream PWQO standards presented in Table 6.1. The predicted DO deficit was expressed by eqs. (4.29) and (4.30) in terms of the percentage of saturation and was compared with the corresponding limit. The predicted total ammonia concentration was compared with the maximum allowable concentration in the river that would be in agreement with the PWQO limit of undissociated ammonia. The maximum allowable $(NH_3)_{tot}$ level that was considered for each window was estimated from eq. (4.42), using the maximum values of temperature and pH among the corresponding 75 percentile values

at the stations of Plantagenet and Casselman. The resulting limits in $(NH_3)_{tot}$ values are presented in Table 6.13.

Table 6.13: Maximum allowable concentration of total ammonia in the river during the optimum discharge windows of study period 1.

Parameter	$Q_{min.daily}^7$			$Q_{min.daily}^{30}$		
	Min. Window	Aver. Window	Max. Window	Min. Window	Aver. Window	Max. Window
T ($^{\circ}C$)	8.0	9.0	13.0	8.0	8.0	8.0
pH	7.7	7.9	7.9	7.75	7.7	7.7
$(NH_3)_{tot}$ (mg/l)	2.519	1.483	1.112	2.250	2.519	2.519

Among the obtained simulation results, the major interest has been in the variation of DO and BOD_u as were predicted with set 1, and in the variation of $(NH_3)_{tot}$ as was predicted with set 2 of the model parameters. As explained above, these estimates were the most conservative ones of the corresponding modelled constituents.

The results obtained for the South Nation River in the spring period, under all the conditions of river flows and waste discharge considered, are presented in Figures 6.10 to 6.13 for the percentage of DO saturation, 6.14 to 6.17 for BOD_u and 6.18 to 6.21 for $(NH_3)_{tot}$. The results obtained for all the river system in the spring period and for average window conditions are given in Appendix B, Tables B.25 to B.28.

The predicted variation of the percentage of DO saturation along the South Nation River in study period 2 under $Q_{min.daily}^{30}$ flows and for all windows is presented in Figure 6.22. As can be seen, the percentage of DO saturation was lower than the corresponding limit of 47% for a significant part of the river length. However, the corresponding values

in study period 1 ranged between 70% and 90% (Figure 6.11), suggesting that the discharge of the treatment plants would be more appropriate in study period 1 with respect to the *DO* constraints. The discussion that follows is based on the results of the analysis in the spring study period.

A study of the impact of the base river flow on the predictions of water quality showed that generally the estimated pollutant concentrations under $Q_{min,daily}^7$ base river flows were at higher levels than they were under $Q_{min,daily}^{30}$ flows, and they also covered a wider range of values, with one of their boundaries being closer to the violation of the PWQO standards. Thus, under the conditions of $Q_{min,daily}^{30}$ flows and the discharge schemes of both years 1991 and 2011, the predicted percentage of *DO* saturation in the South Nation River varied over all windows from 70% to 90 %, the BOD_u level from 1.0 to 4.5 mg/l, and the $(NH_3)_{tot}$ level from one eleventh to one half of the maximum allowable $(NH_3)_{tot}$ concentration. However, under the conditions of $Q_{min,daily}^7$ flows the values of percentage of *DO* saturation over all windows were in the wider range between 60% to 95%, the BOD_u level was between 0.5 to 6.0 mg/l, and the $(NH_3)_{tot}$ level under the conditions of the maximum and average windows was between one-fifth to a value twice as high as the maximum allowable $(NH_3)_{tot}$ concentration.

An analysis of the concentration of modelled constituents in the South Nation River as well as in the other rivers of the whole system (Tables B.25 to B.28) showed that the %*DO_s* and BOD_u levels satisfied the corresponding PWQO standards for the most part of the river system, while the $(NH_3)_{tot}$ levels resulted in significant violations of the total ammonia limit in some river sites.

However, severe violations of the limits of all the modelled constituents were predicted by the model mainly in the tributaries of the South Nation River. In some of these streams the predicted %*DO_s* level was even reaching negative values. This might imply that a modified set of the model parameters or a different estimate of the background concentration at those streams would be more representative of their

conditions. In the model applied the parameters were assumed to be constant for the whole system and their values were estimated from the literature. At a higher level of detail, a calibrated or even validated model with constant or variable parameters could be applied for the specific case study. However (i) the inadequacy of the available data for calibration, (ii) the occurrence of negative values to only a small number of sites with respect to the total number of river nodes in the system, and (iii) mainly the fact that the current form of the model had to be applied to allow for immediate comparison with results obtained from previous studies (Droste and Gibbons, 1992), resulted in considering the model as adequate for the purposes of this study.

The most severe violations in the whole river system were predicted for the Castor River and the drains near to it. In particular a non-acceptable water quality was predicted for the Annable and Henderson Drains due to the low streamflows and the excess of the discharging load from the plants of Winchester community and Ault industry correspondingly. Furthermore violations were predicted for the main Castor River due to the discharge of the Russell treatment plant and the increased pollutant load that was transferred upstream of the Embran discharge point.

The simulation also showed that PWQO violations occurred in the streams of the McMartin Drain, Moose Creek and Cumming Drain. These streams had low streamflows and accepted the loads from the plants of Williamsburg, Moose Creek and Maxville communities, respectively. The water quality in the above streams had also influenced the quality in the South Nation River and created critical reaches of increased pollutant concentrations mainly between nodes M, L and H, F.

The above results were compared with the results of the dilution analysis (paragraph 6.8, A3), that had considered the discharge of each plant in absence of any other plant discharging at that time in the river system. The agreement found in these two sets of results reinforces the requirement for the identified communities and industries to apply a quality control method in order to avoid violations of the PWQO standards.

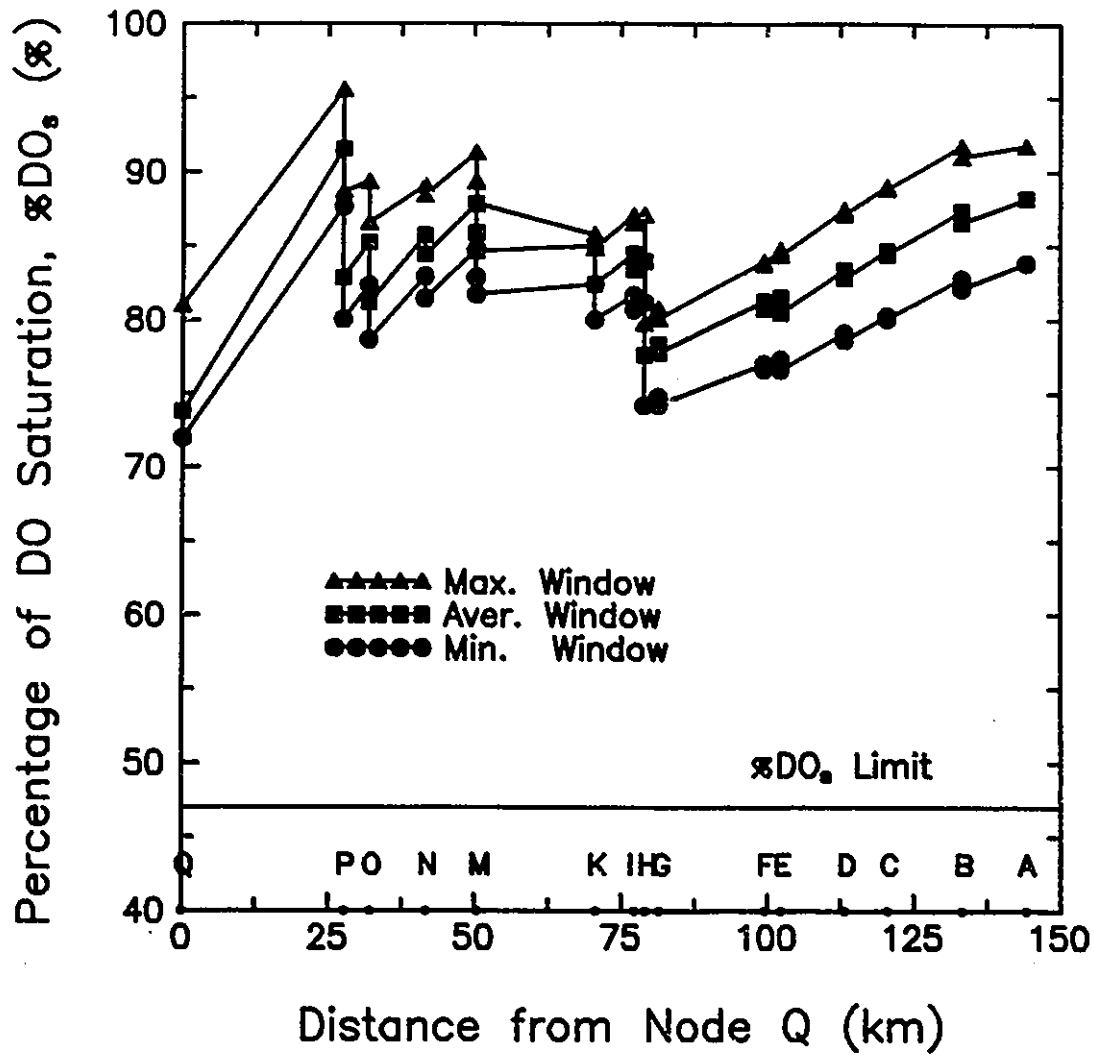


Figure 6.10: Predicted variation of the percentage of DO saturation along the South Nation River in study period 1 for $Q_{min,daily}^7$ river flows and the waste load of the plants in 1991 using the rates of set 1.

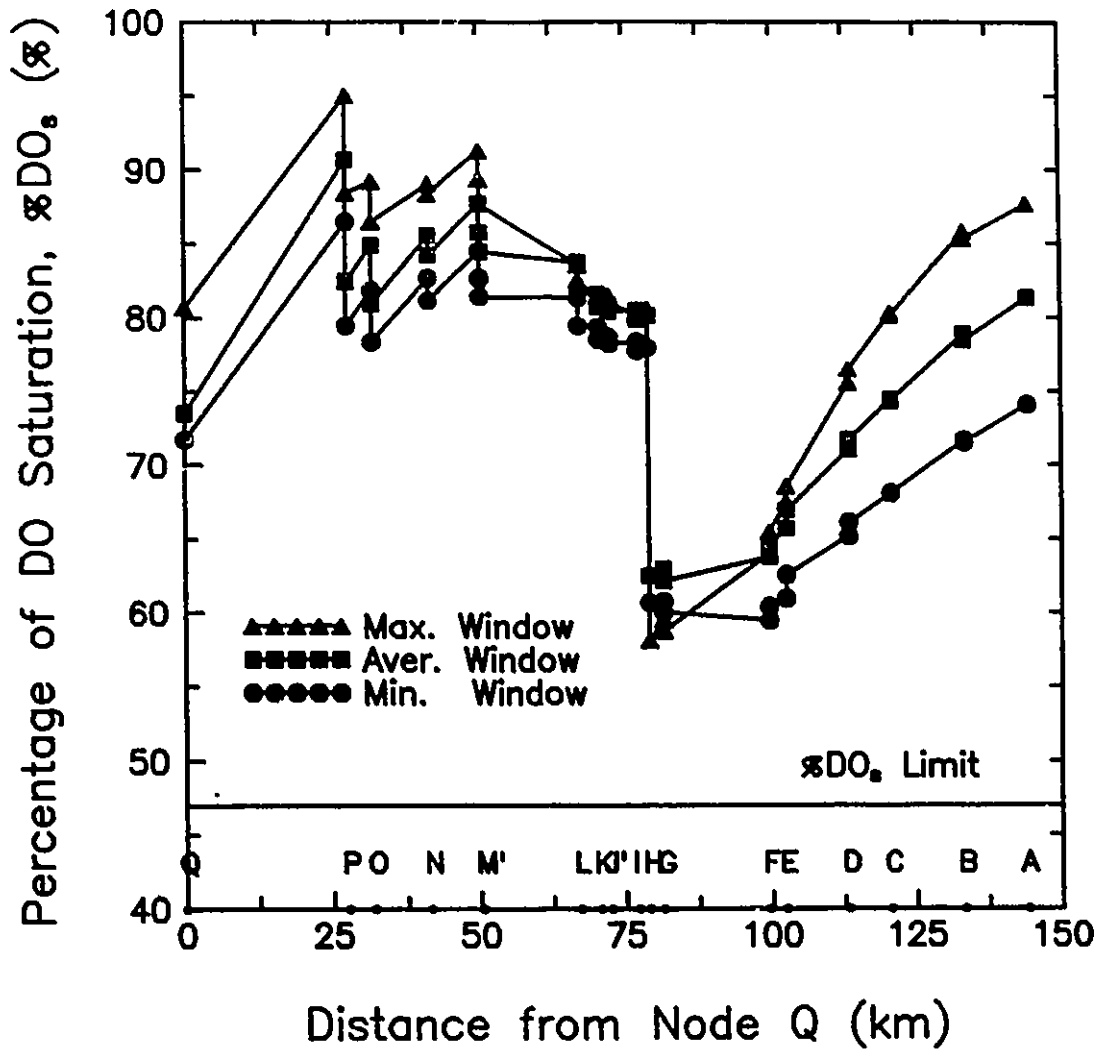


Figure 6.11: Predicted variation of the percentage of DO saturation along the South Nation River in study period 1 for $Q_{min.daily}^7$ river flows and the waste load of the plants in 2011 using the rates of set 1.

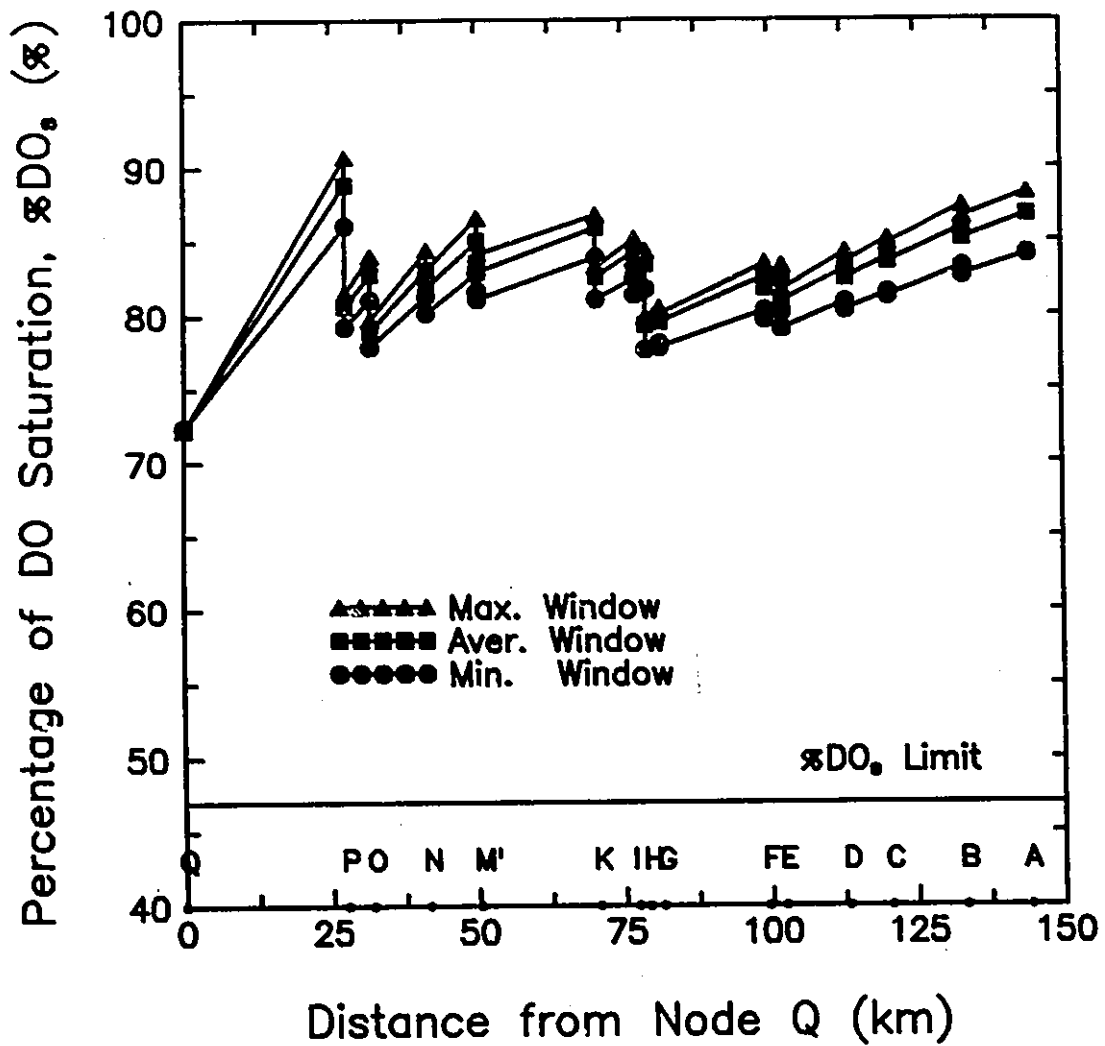


Figure 6.12: Predicted variation of the percentage of DO saturation along the South Nation River in study period 1 for $Q_{min,daily}^{30}$ river flows and the waste load of the plants in 1991 using the rates of set 1.

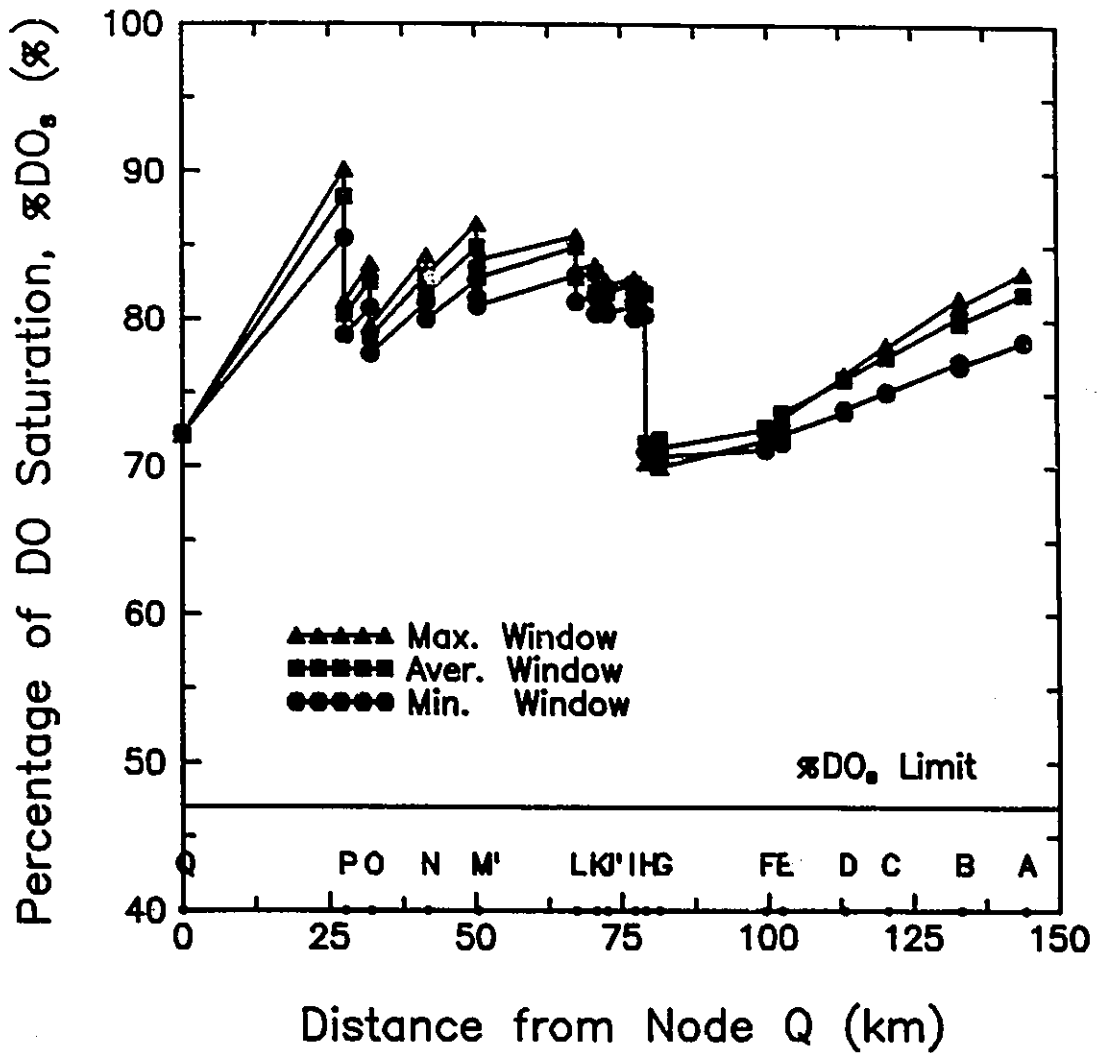


Figure 6.13: Predicted variation of the percentage of DO saturation along the South Nation River in study period 1 for $Q_{min,daily}^{30}$ river flows and the waste load of the plants in 2011 using the rates of set 1.

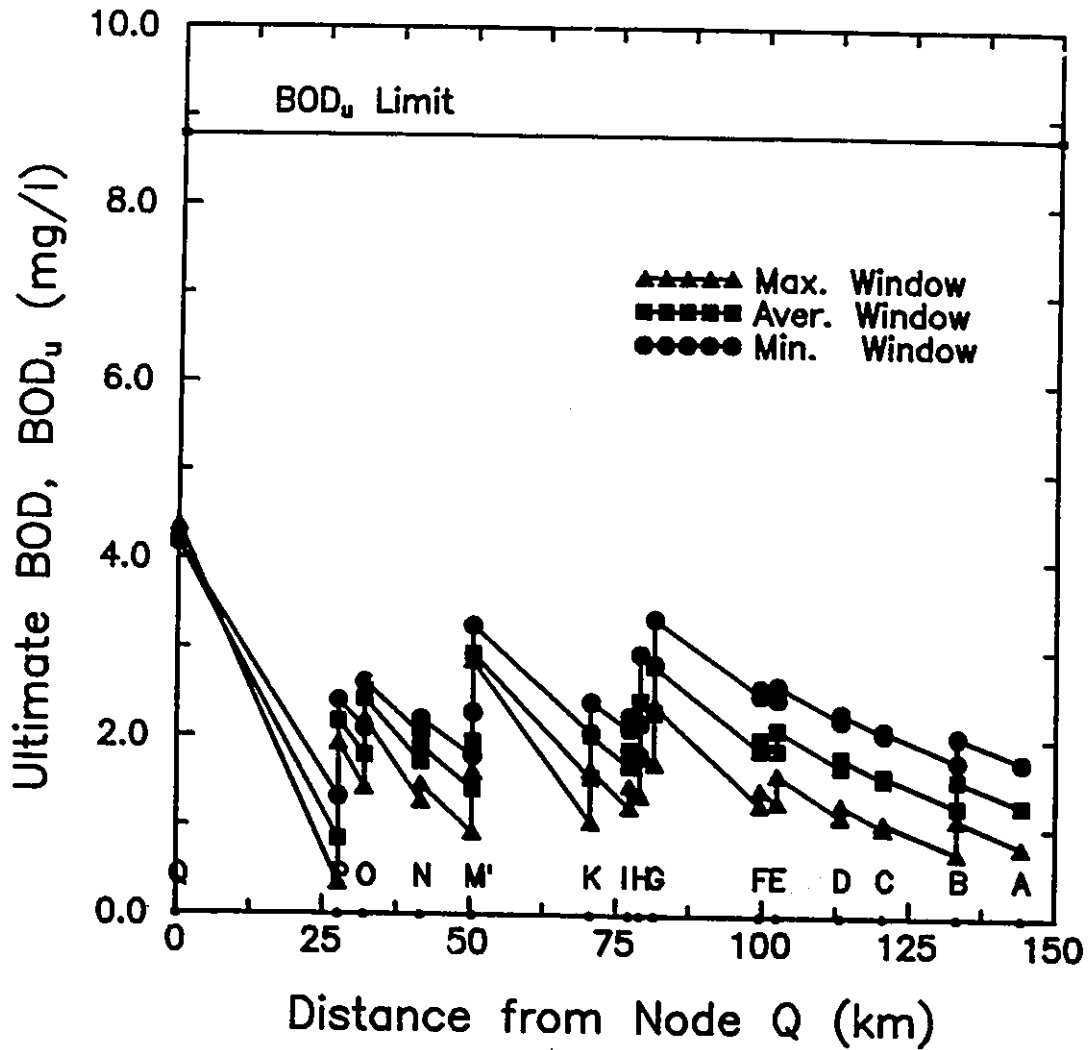


Figure 6.14: Predicted variation of the ultimate BOD concentration along the South Nation River in study period 1 for $Q_{min,daily}^7$ river flows and the waste load of the plants in 1991 using the rates of set 1.

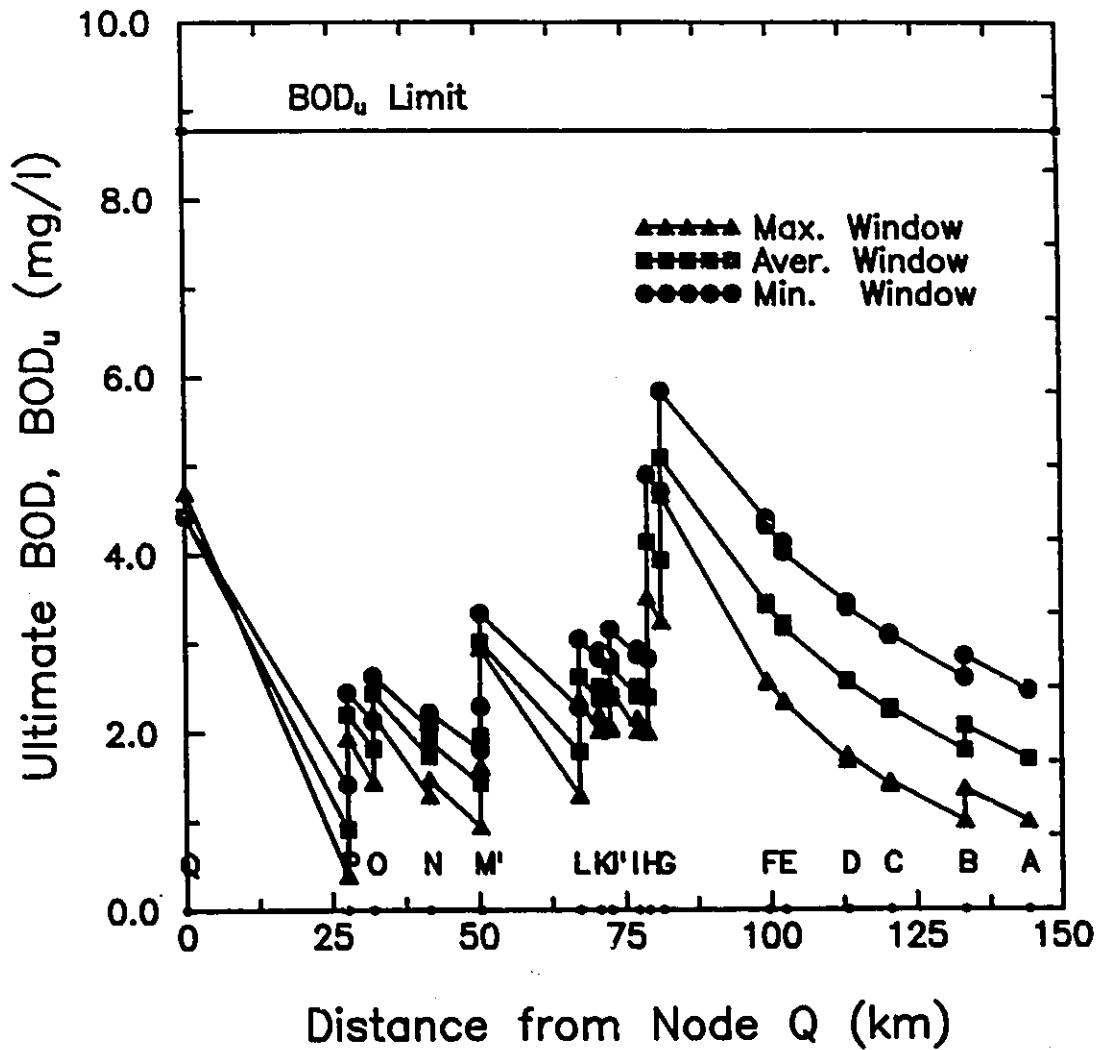


Figure 6.15: Predicted variation of the ultimate BOD concentration along the South Nation River in study period 1 for $Q_{min,daily}^7$ river flows and the waste load of the plants in 2011 using the rates of set 1.

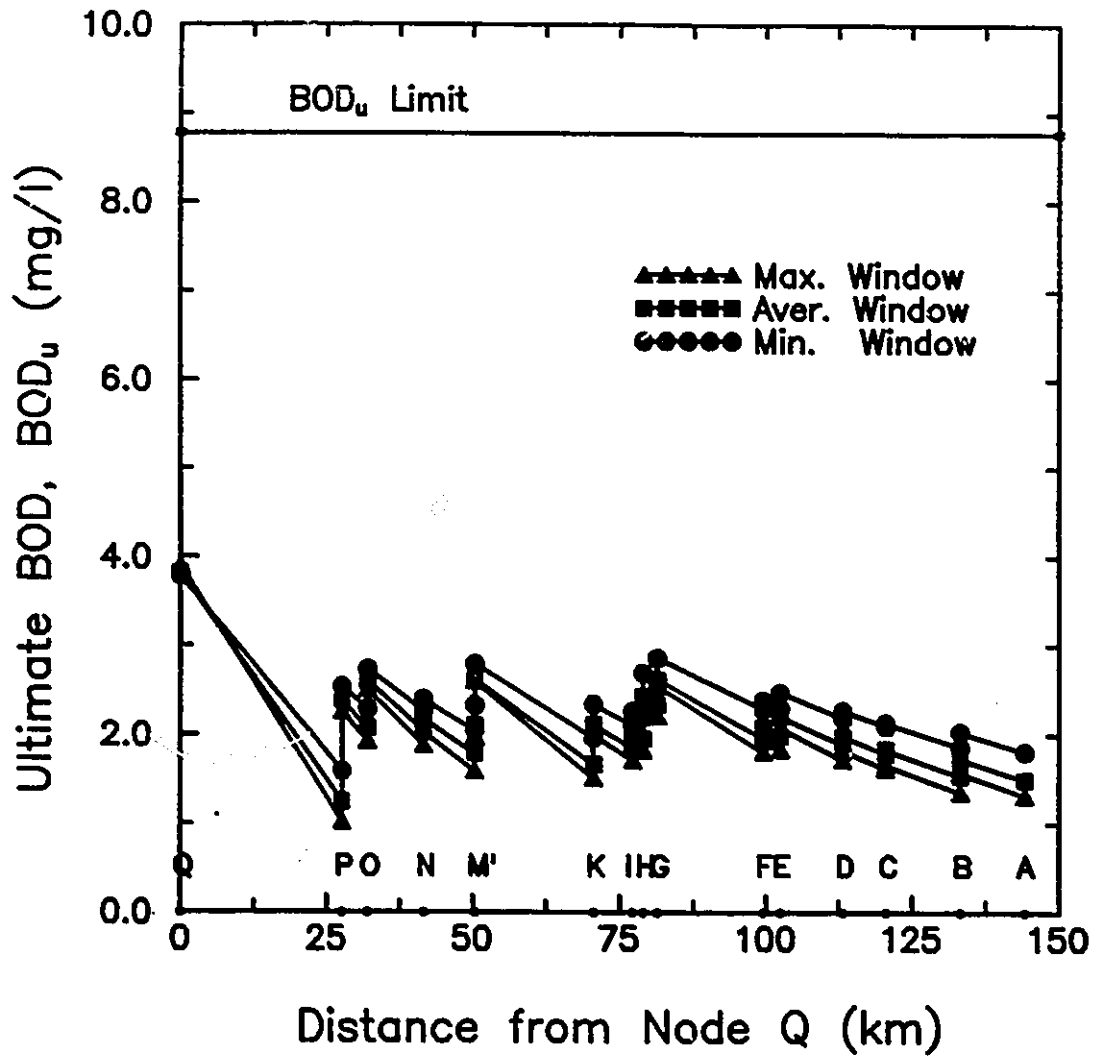


Figure 6.16: Predicted variation of the ultimate BOD concentration along the South Nation River in study period 1 for $Q_{min,daily}^{30}$ river flows and the waste load of the plants in 1991 using the rates of set 1.

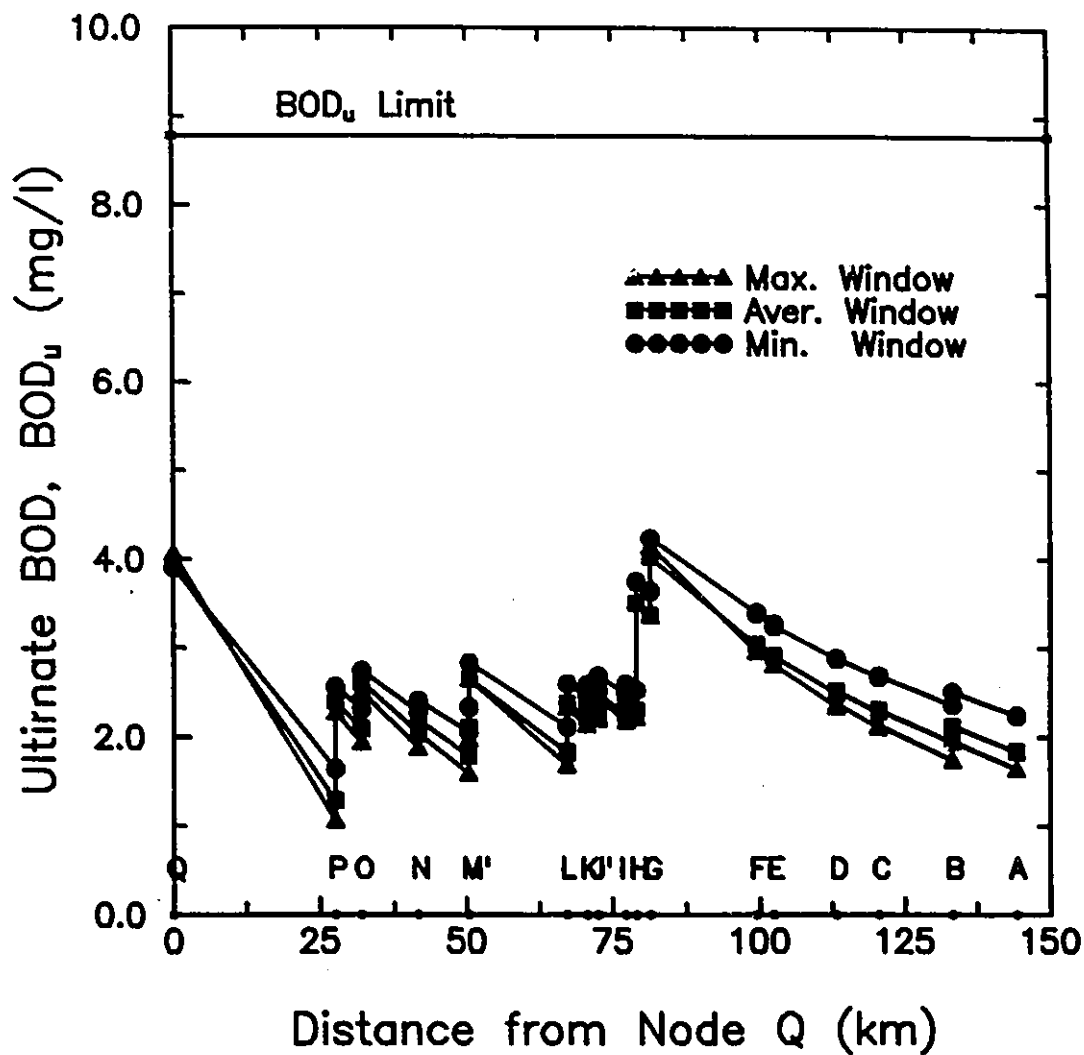


Figure 6.17: Predicted variation of the ultimate BOD concentration along the South Nation River in study period 1 for $Q_{min,daily}^{30}$ river flows and the waste load of the plants in 2011 using the rates of set 1.

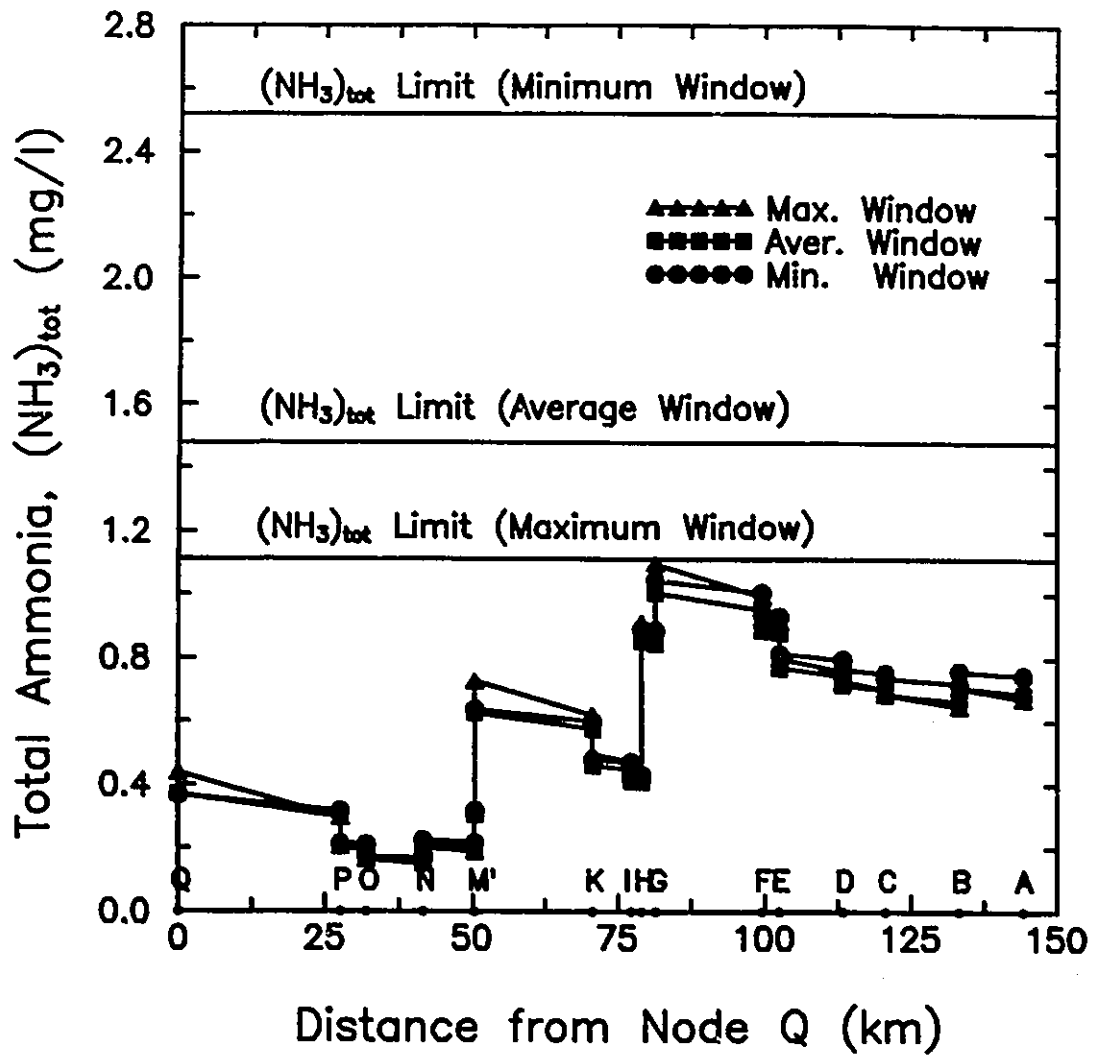


Figure 6.18: Predicted variation of total ammonia concentration along the South Nation River in study period 1 for $Q_{min.daily}^7$ river flows and the waste load of the plants in 1991 using the rates of set 2.

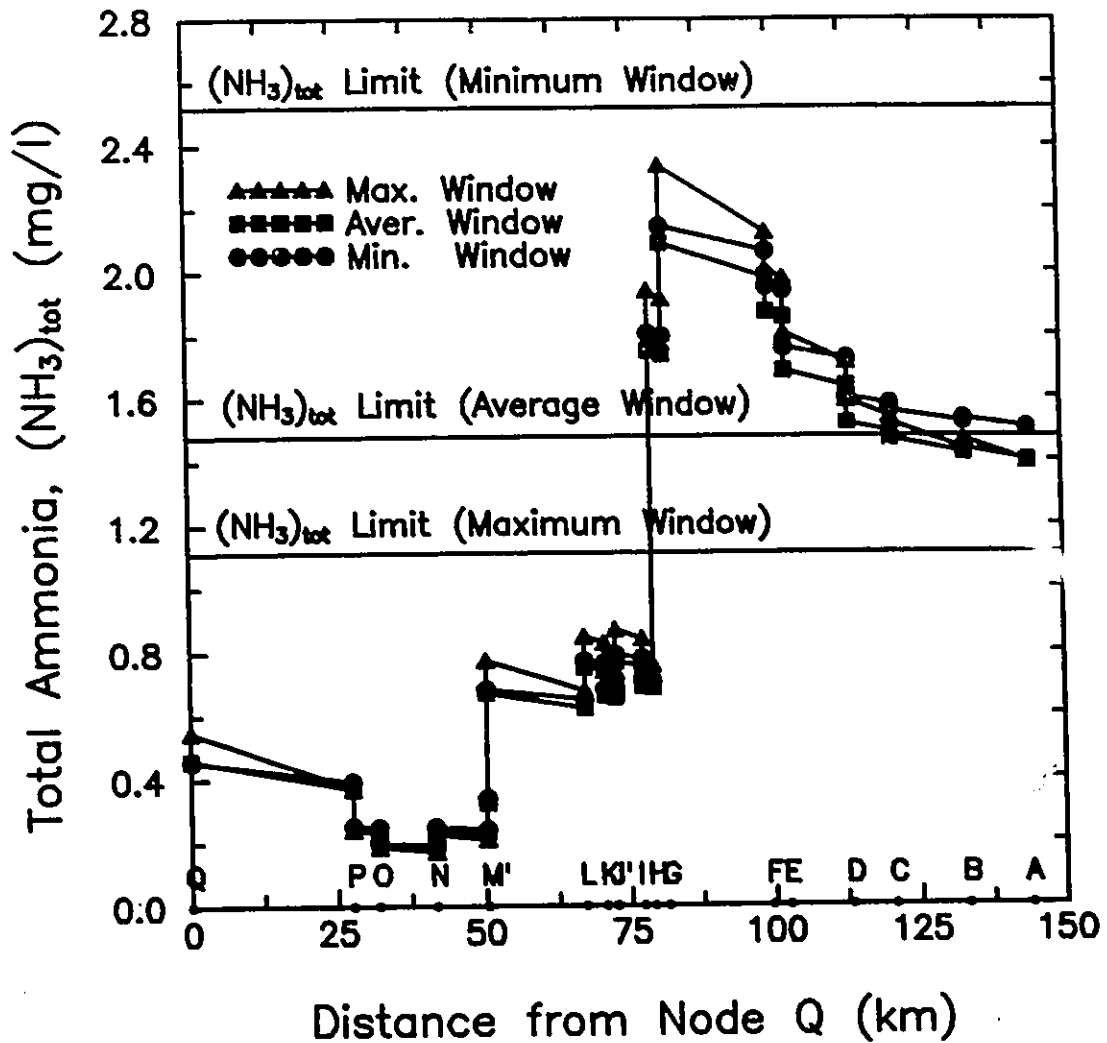


Figure 6.19: Predicted variation of total ammonia concentration along the South Nation River in study period 1 for $Q_{min,daily}^7$ river flows and the waste load of the plants in 2011 using the rates of set 2.

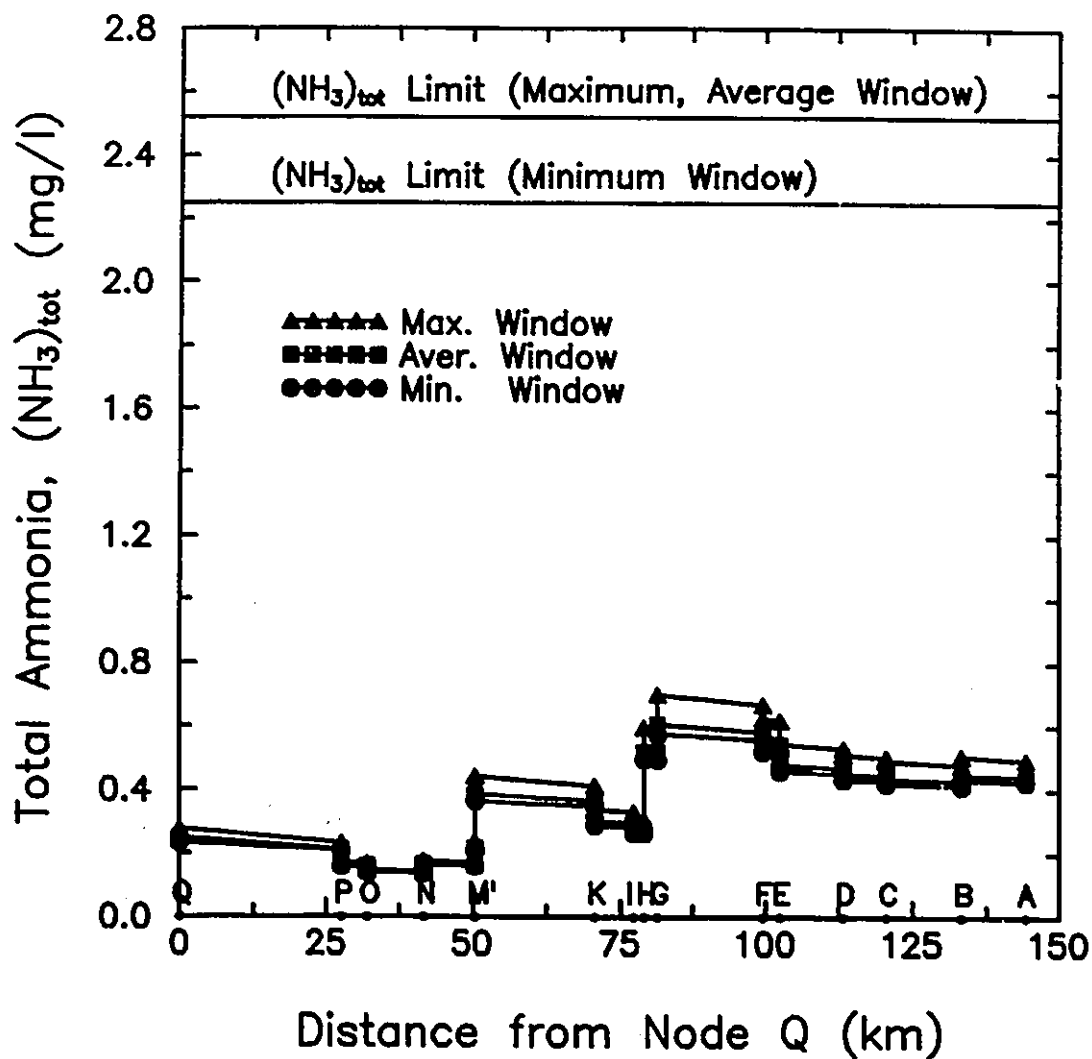


Figure 6.20: Predicted variation of total ammonia concentration along the South Nation River in study period 1 for $Q_{\text{min,daily}}^{30}$ river flows and the waste load of the plants in 1991 using the rates of set 2.

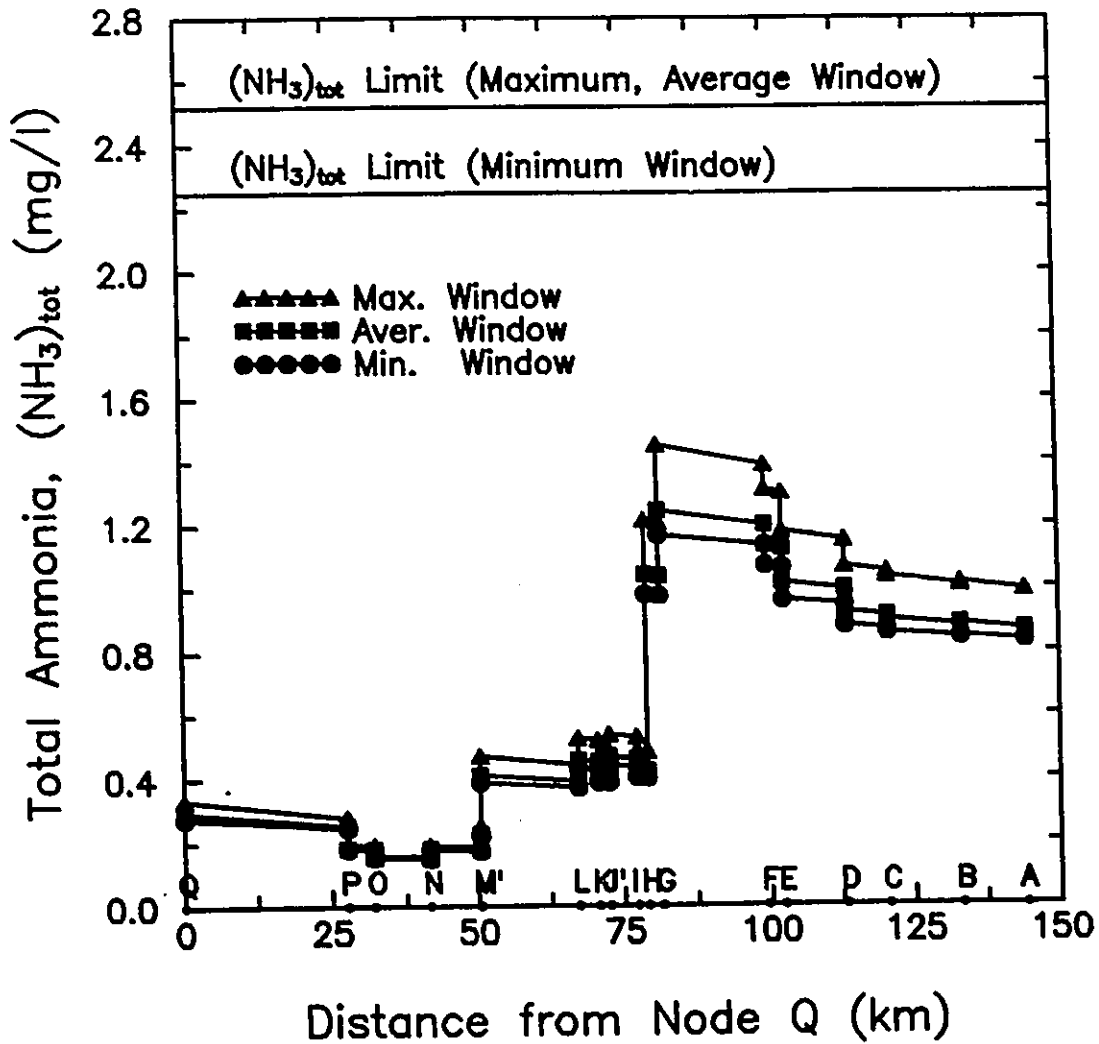


Figure 6.21: Predicted variation of total ammonia concentration along the South Nation River in study period 1 for $Q_{\text{min,daily}}^{30}$ river flows and the waste load of the plants in 2011 using the rates of set 2.

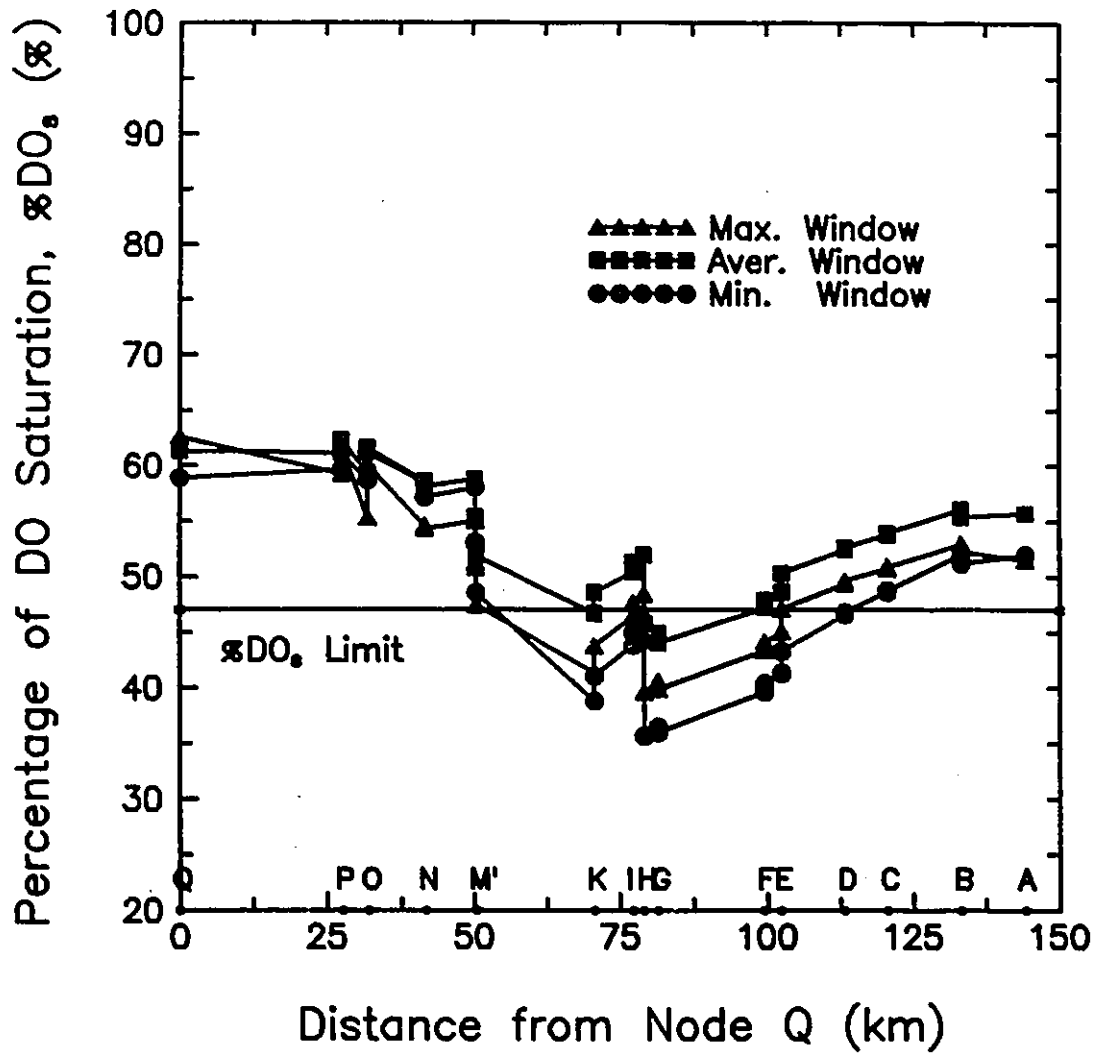


Figure 6.22: Predicted variation of the percentage of DO saturation along the South Nation River in study period 2 for $Q_{min,daily}^{30}$ river flows and the waste load of the plants in 1991 using the rates of set 1.

6.10 Strategies of Water Quality Control - Optimization of Water Quality

The optimization procedure of water quality described in part B of section 4.4 was applied to the South Nation River system for the spring study period, considering both $Q_{min,daily}^7$ and $Q_{min,daily}^{30}$ base river flows and the discharge conditions of years 1991 and 2011. In this procedure results were initially obtained using the model parameters of set 1. The water quality was considered as acceptable if the PWQO standards of all DO, ultimate BOD and total ammonia were satisfied simultaneously. The proposed optimal scheme for the plants was also tested by predicting the river water quality under the new conditions with the model parameters of set 2. In two cases the optimization procedure had to be continued. The following results of the optimization satisfy the water quality standards for both sets of parameters. The results will be presented separately for each combination of the above conditions.

a. $Q_{min,daily}^{30}$ river flows, 1991. The simulation results for maximum window conditions showed that a violation of the PWQO standards occurred at the discharge points of the "remote" plants of Maxville, Williamsburg and Winchester communities and of Ault industry. Similarly, violations occurred at the junction nodes AB and AC.

The remote communities and the possible requirement for the allocation of their discharge point and/or the staging of their discharge were initially considered. Whenever a relocation was suggested downstream of a node, it corresponded to a discharge point at a distance of 0.1 or 0.05 km downstream of the node. The distance was selected based on the distance of the node from its next node and on the number of plants (1 or 2) that would relocate their discharge after the same mentioned node. The drainage area of the relocated discharge point was estimated from the distance of the node from its next node and the difference in the drainage areas of the two nodes.

From this part of the analysis, it was found that the treatment plant of Williamsburg would have to relocate its discharge point downstream, right after the junction AF of McMartin Drain with Hess Creek, at node AF'. Alternatively the plant would have to change its degree of treatment and apply the extended aeration process. This option was considered because the distance between the current and the suggested discharge point (from the relocation option) was 9.8 km, and the related cost was expected to be significant.

The treatment plant of Maxville would have to relocate its effluent downstream of the junction T of West Branch Scotch River and Cumming Drain, at node T'.

The plant of Ault industry would have to relocate downstream of the junction AC of Annable Drain with Henderson Drain, at node AC'. As can be seen, the required dilution with respect to the ultimate BOD, at the discharge point of Ault, has a negative value. This can be explained from the definition of D_{req} , eq. (4.45), and the fact that the BOD_u concentrations at the wastewater, C_w (5.0 mg/l), and at the river node AB', immediately prior the discharge, C_r (3.343 mg/l), are smaller than the corresponding river standard, C_m (8.48 mg/l). This relation implies that the BOD_u concentration at node AB' after the discharge will not reach or exceed the river BOD limit, and Ault industry can relocate its discharge at that point. It is noted that a negative value of D_{req} could also result if both C_w and C_r exceeded the river standard C_m . However, in that case any relocation to the river point of reference would be prevented.

The plant of Winchester community, which had a greater annual waste volume and pollutant load than the Ault industry, would have to relocate further downstream, after the junction Y of the Castor River with East Castor River, at Y'. In that case the total river length between the current and the suggested discharge point, AD to Y' would be 25.6 km. The actual required pipe distance of the effluent would be probably different, however it would be expected to be significant. Thus it was tested whether the Winchester plant effluent could be relocated to the South Nation River and in particular to

node N', after node N, that was suggested in a previous study (Droste and Gibbons, 1992). As was estimated in this study, the piped distance between point AD and node N' would be 6.0 km. As was further found from the optimization, the discharge point after N' satisfied the quality limits for maximum window conditions. Hence the relocation of Winchester plant effluent to node N' was considered as an acceptable solution. It is noted that all the above plants could discharge their effluent during the maximum window.

The results of the optimization are presented in Tables 6.14 and 6.15. The predicted water quality in the South Nation River after the optimization of conditions for all windows is presented in Figures 6.23 to 6.25.

In these results the values of D_{av} , D_{req} and of the maximum allowed V_w , as well as the predictions of the variation of the dissolved oxygen and ultimate BOD, were obtained using the model parameters of set 1. The corresponding values related to the total ammonia were obtained using the model parameters of set 2. It should be noted that the values in the tables which are in parentheses represent possible solutions which were found during the optimization procedure but which were not considered as optimum ones.

Table 6.14: Relocation of the discharge points of treatment plants for $Q_{min.daily}^{30}$ river flows in study period 1 and for the waste load conditions of year 1991.

Plant	Discharge Point		D_{av}	D_{req}		
	Current	Proposed		DOD	BOD_u	$(NH_3)_{tot}$
Ault	AE	AC'	1.859	0.526	-0.272	1.490
Maxville	U	T'	13.508	0.524	6.532	4.776
(Williamsburg)	(AG)	(AF')	(17.918)	(0.525)	(6.537)	(4.776)
(Winchester)	(AD)	(Y')	(20.493)	(0.493)	(6.530)	(5.392)
Winchester	AD	N'	28.003	0.374	5.188	4.847

Table 6.15: Change of the degree of treatment of the treatment plants for $Q_{min.daily}^{30}$ river flows and for the waste load conditions of year 1991.

Plant	Node	Type	% DO_s (%)	BOD_u (mg/l)	$(NH_3)_{tot}$ (mg/l)
Williamsburg	AG	EA	65.518	6.283	1,009

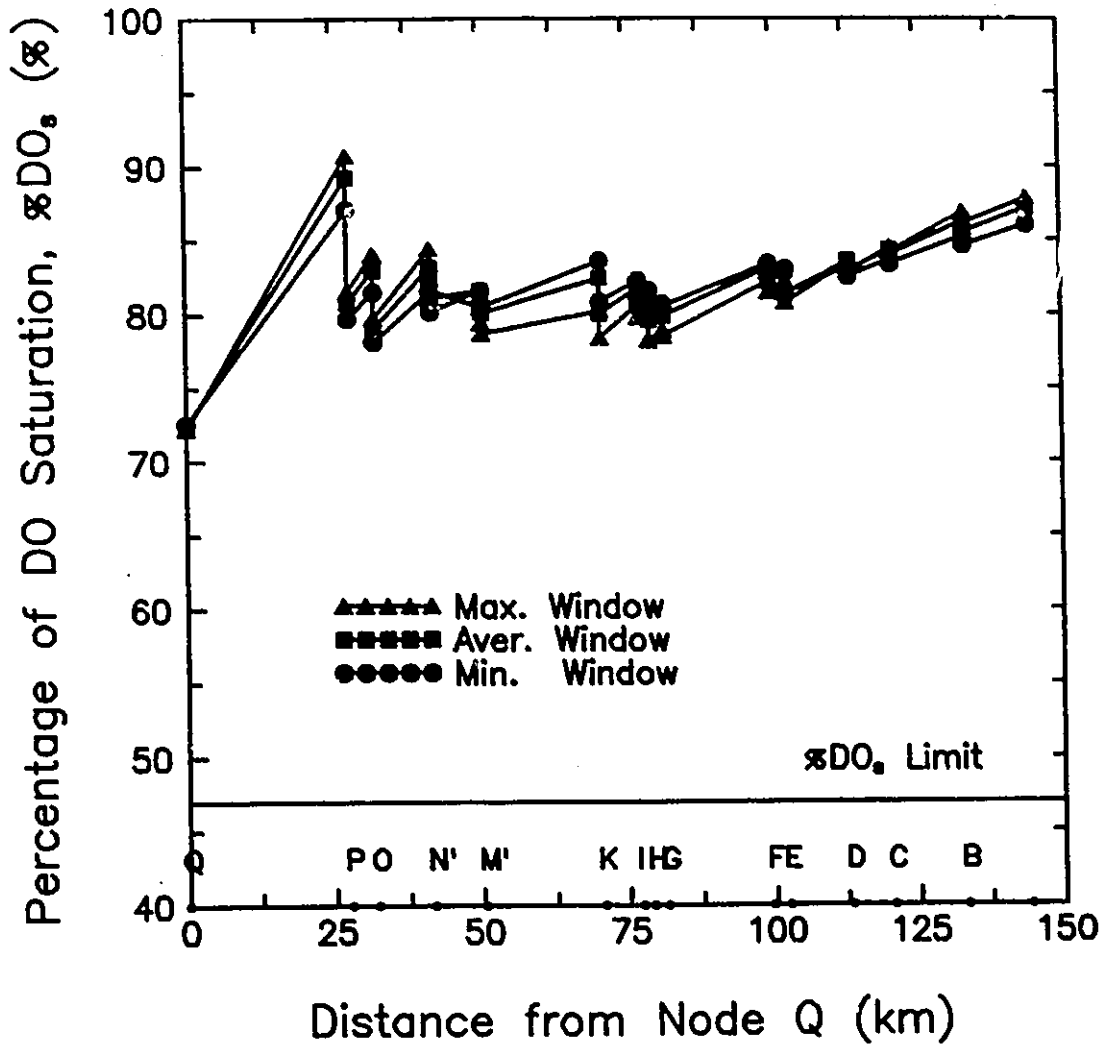


Figure 6.23: Predicted variation of the percentage of DO saturation along the South Nariion River in study period 1 after the application of the optimal discharge scheme under $Q_{min,daily}^{30}$ river flows and conditions of 1991 using the rates of set 1.

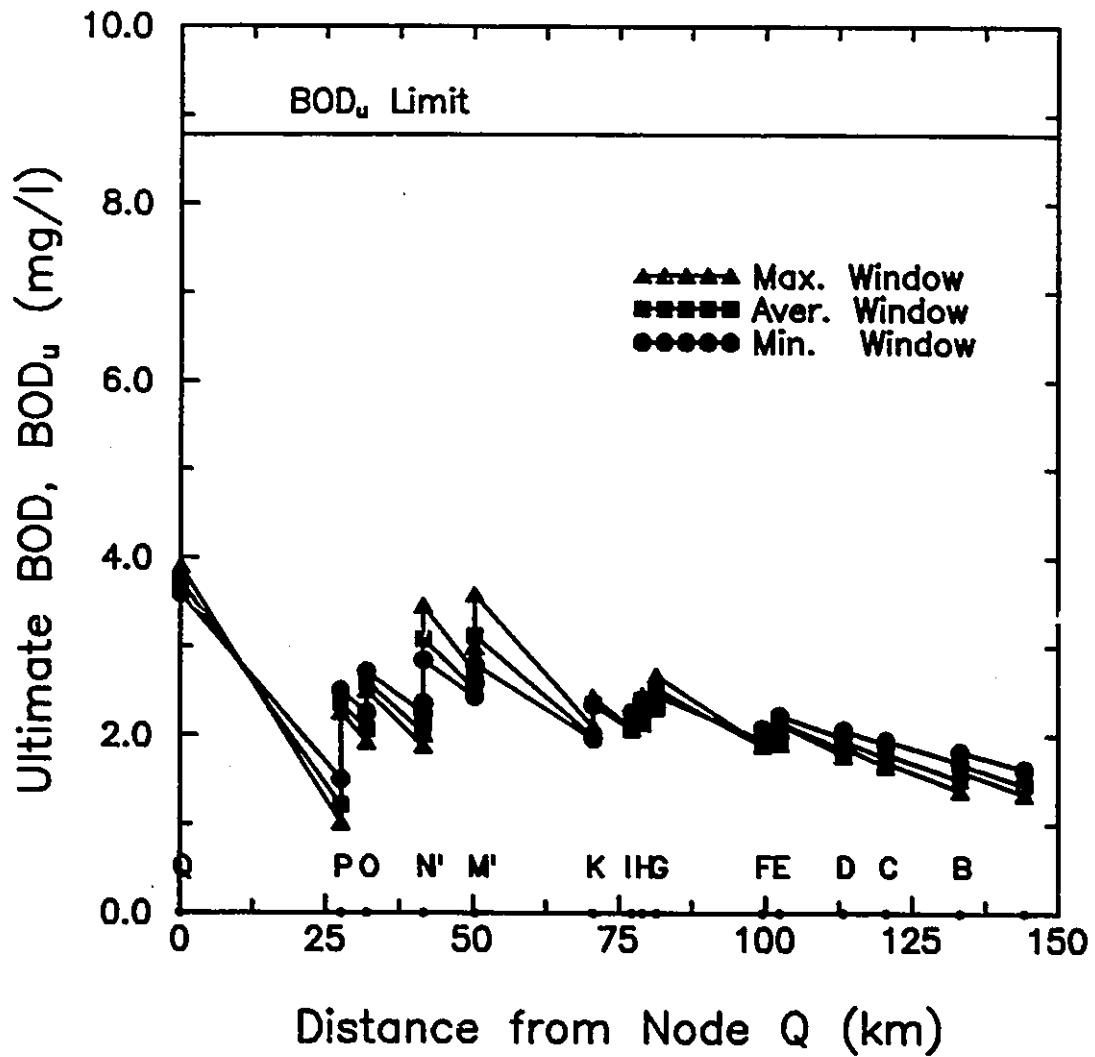


Figure 6.24: Predicted variation of the ultimate BOD concentration along the South Nation River in study period 1 after the application of the optimal discharge scheme under $Q_{min,daily}^{30}$ river flows and conditions of 1991 using the rates of set 1.

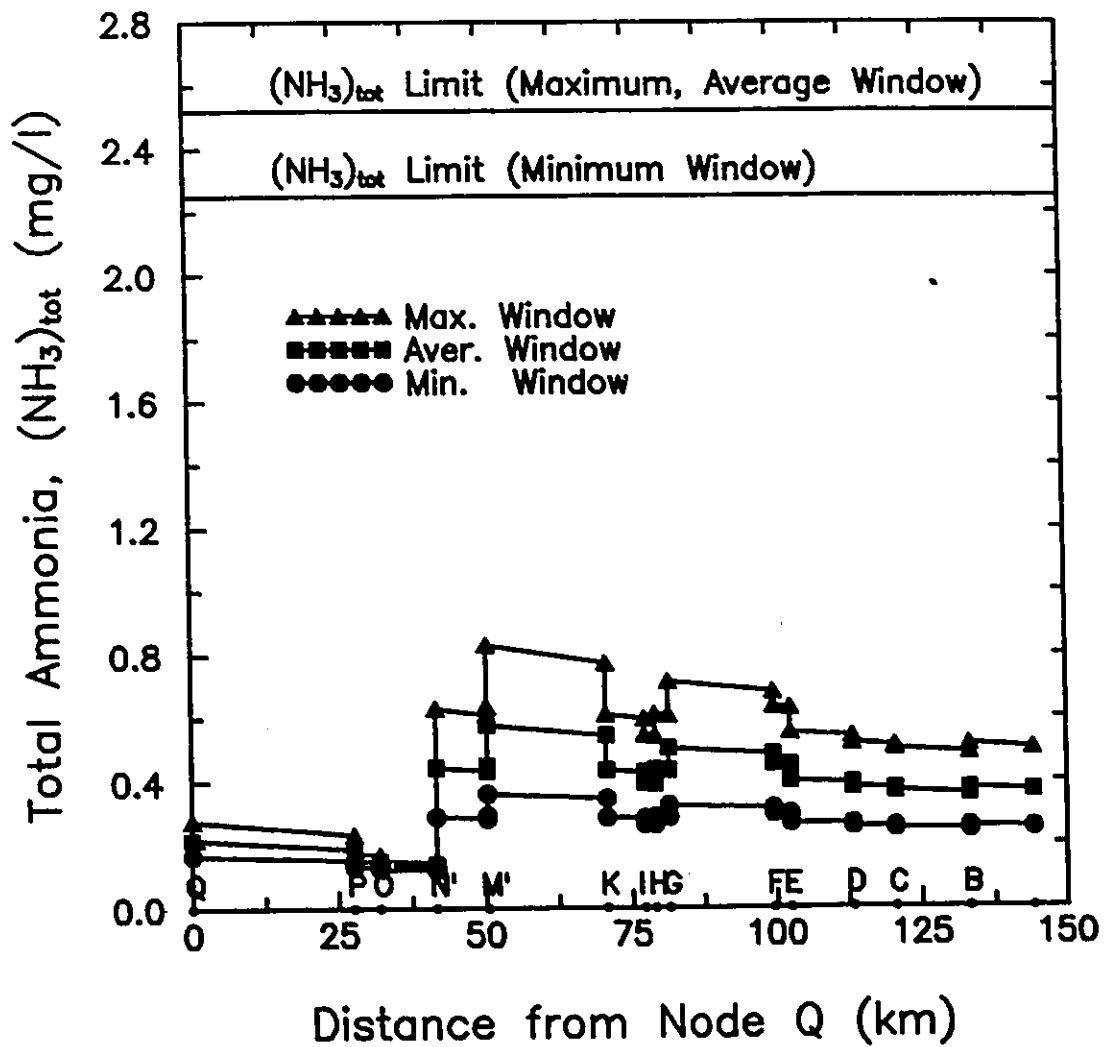


Figure 6.25: Predicted variation of total ammonia concentration along the South Nation River in study period 1 after the application of the optimal discharge scheme under $Q_{min,daily}^{30}$ river flows and conditions of 1991 using the rates of set 2.

b. $Q_{min.daily}^{30}$ river flows, 2011. The simulation results for the maximum window in the future conditions showed that the effluents of the same plants as in 1991 (case c) as well as of the "remote" Moose Creek plant and of the "interior" Embrun plant resulted in the violation of the water quality standards in the river.

The optimization procedure showed that the Moose Creek plant would have to discharge its effluent during the minimum window. The Maxville plant would have to relocate to point T' and the Williamsburg plant would have to relocate to point AF' or better to change its treatment to extended aeration as in case a. The Ault industry plant should relocate at AB'. The last three plants of Maxville, Williamsburg and Ault could discharge during the maximum window. The Winchester plant would have to relocate to node Y' and could also discharge in the maximum window. In that case the Embrun plant should discharge during the minimum window. Applying the same step as in case a, it was found that the Winchester plant should relocate its effluent at N' and could discharge in the maximum window. In that case the Embrun plant should discharge during the average window. The results of the optimization are presented in Tables 6.16 to 6.18 and in Figures 6.26 to 6.28.

Table 6.16: Relocation of the discharge points of treatment plants for $Q_{min.daily}^{30}$ river flows in study period 1 and for the waste load conditions of year 2011.

Plant	Discharge Point		D_{av}	D_{req}	DOD	BOD_u	$(NH_3)_{tot}$
	Current	Proposed					
Ault	AE	AB'	3.063	0.509	-0.266	1.489	
Maxville	U	T'	13.186	0.524	6.532	4.776	
(Williamsburg)	(AG)	(AF')	(16.26)	(0.525)	(6.537)	4.776	
(Winchester)	(AD)	(Y')	(19.291)	(0.650)	(9.590)	8.277	
Winchester	AD	N'	24.618	0.376	5.194	4.873	

Table 6.17: Staging of the discharge of the treatment plants effluent during the optimum windows of study period 1 for $Q_{min,daily}^{30}$ river flows and the waste load conditions of year 2011.

Plant	Node	Annual V_w (m^3)	Maximum Allowed V_w (m^3)			Q_w (m^3/s) (window)
			DOD	BOD_u	$(NH_3)_{tot}$	
(Embrun)	(Z)	(1.55×10^6)	(3.29×10^7)	(2.19×10^6)	(2.33×10^6)	(0.997) (min.)
Embrun	Z	1.55×10^6	2.86×10^7	2.08×10^6	2.49×10^6	0.579 (aver.)
Moose Creek	X	1.01×10^5	1.29×10^6	1.03×10^5	1.23×10^5	0.065 (min.)

Table 6.18: Change of the degree of treatment of the treatment plants for $Q_{min,daily}^{30}$ river flows and for the waste load conditions of year 2011.

Plant	Node	Type	% DO_5 (%)	BOD_u (mg/l)	$(NH_3)_{tot}$ (mg/l)
Williamsburg	AG	EA	64.928	7.101	1.082

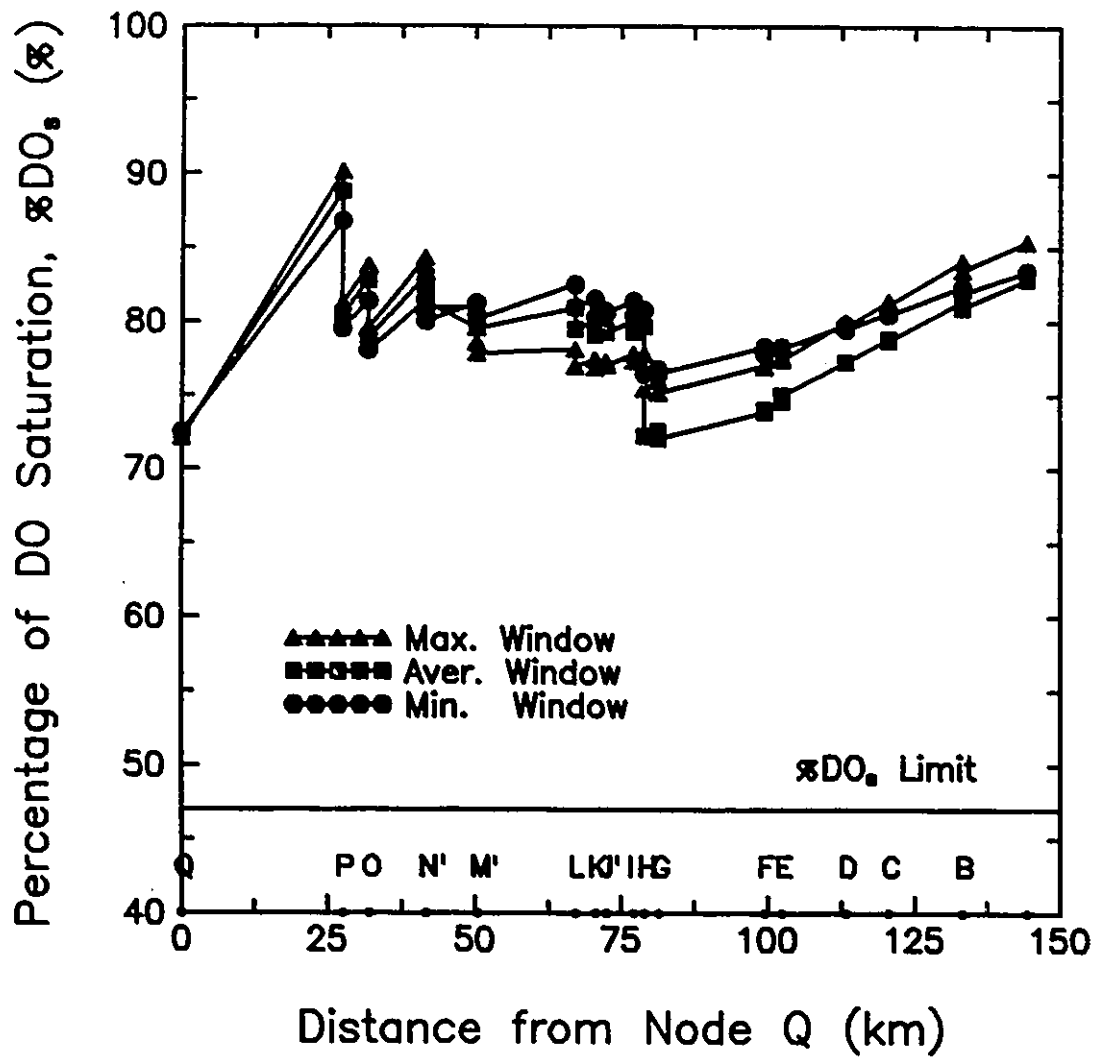


Figure 6.26: Predicted variation of the percentage of DO saturation along the South Nation River in study period I after the application of the optimal discharge scheme under $Q_{min,daily}^{30}$ river flows and conditions of 2011 using the rates of set 1.

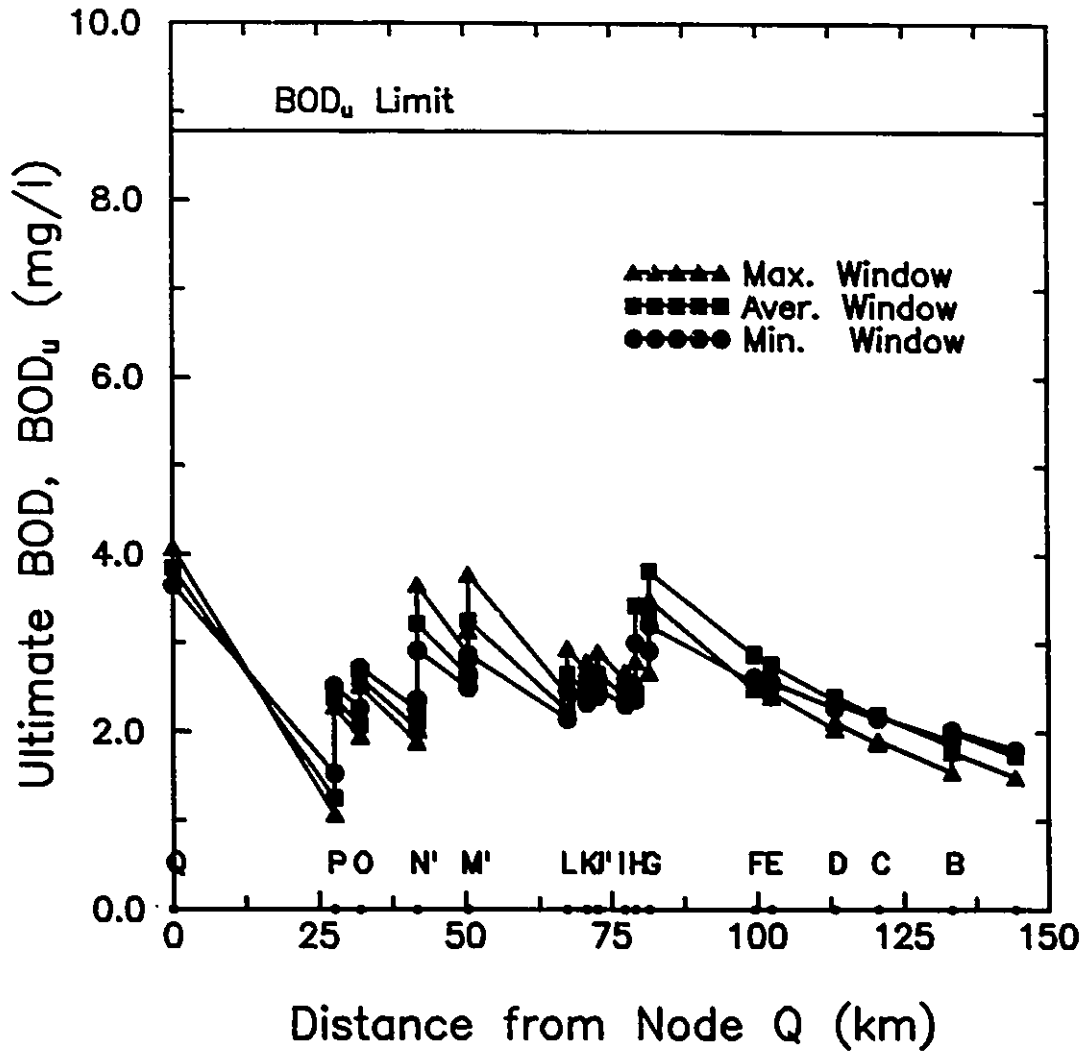


Figure 6.27: Predicted variation of the ultimate BOD concentration along the South Nation River in study period I after the application of the optimal discharge scheme under $Q_{min,daily}^{30}$ river flows and conditions of 2011 using the rates of set 1.

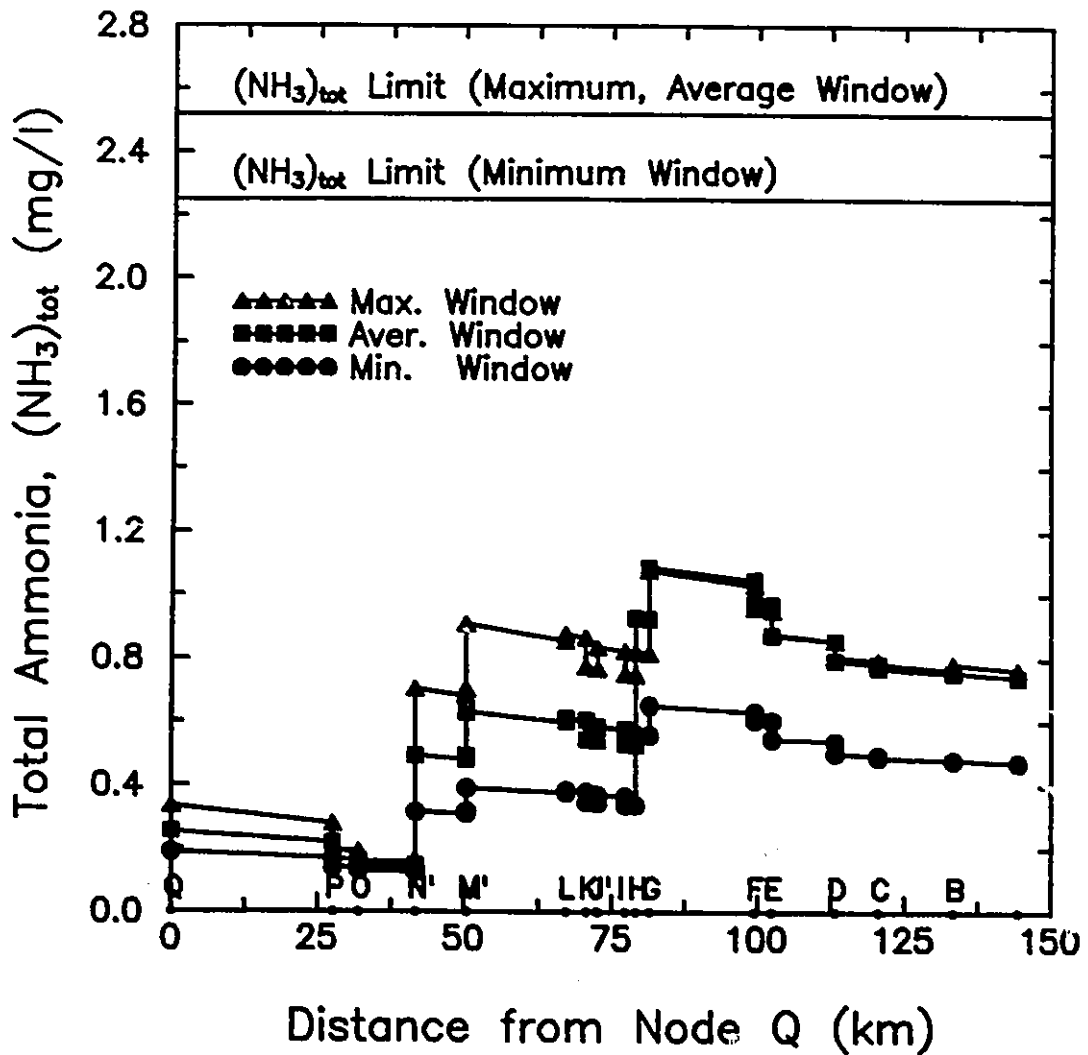


Figure 6.28: Predicted variation of total ammonia concentration along the South Nation River in study period 1 after the application of the optimal discharge scheme under $Q_{\text{min,daily}}^{30}$ river flows and conditions of 2011 using the rates of set 2.

c. $Q_{min,daily}^7$ river flows, 1991. The simulation results for maximum window conditions showed that a violation of the PWQO standards occurred from the "remote" plants of Maxville, Williamsburg and Winchester communities and of Ault industry, as well as from the "interior" plant of Embrun community.

The optimization procedure showed that the plants of Maxville and Winchester communities would have to relocate their effluent to nodes T' and N' as in case a and could discharge their effluent in the minimum and average windows, respectively. The Embrun plant should also discharge during the average window. The plant of Williamsburg would have to upgrade its treatment to extended aeration, similarly to case a and was allowed to discharge in the average window. The plant of Ault industry should relocate further its discharge downstream of the junction AB of the East Castor River with the Cumming Drain, at node AB', and discharge in the average window.

It is noted that the plants of Williamsburg and Winchester could also relocate to nodes AF' and Y' and discharge both in the average window. In that case the Embrun plant should discharge in the minimum window. However, that solution was expected to be less costly than the one suggested above.

The results obtained from the optimization are presented in Tables 6.19 to 6.21. The resulting water quality for the South Nation River after the optimization and under the conditions of all windows is shown in Figures 6.29 to 6.31.

Table 6.19: Staging of the discharge of the treatment plants effluent during the optimum windows of study period 1 for $Q_{min.daily}^7$ river flows and the waste load conditions of year 1991.

Plant	Node	Annual V_w (m^3)	Maximum Allowed V_w (m^3)			Q_w (m^3/s) (window)
			DOD	BOD_u	$(NH_3)_{tot}$	
Ault	AB'	1.94×10^5	1.94×10^6	-3.29×10^6	2.60×10^5	0.066 (aver.)
(Embrun)	(Z)	4.75×10^5	1.61×10^7	9.98×10^5	1.26×10^6	0.323 (min.)
Embrun	Z	4.75×10^5	1.86×10^7	1.30×10^6	6.88×10^5	0.162 (aver.)
Maxville	T'	5.99×10^4	9.72×10^5	7.81×10^4	(1.07×10^5)	0.041 (min.)
(Williamsburg)	(AF')	(6.42×10^4)	(1.56×10^6)	(1.10×10^5)	(7.88×10^4)	(0.022) (aver.)
Williamsburg	AG	6.42×10^4	3.87×10^5	7.29×10^4	6.98×10^4	0.022 (aver.)
(Winchester)	(Y')	(6.30×10^5)	(1.83×10^7)	(1.27×10^6)	(6.84×10^5)	(0.214) (aver.)
Winchester	N'	6.30×10^5	3.27×10^7	2.18×10^6	1.17×10^6	0.214 (aver.)

Table 6.20: Relocation of the discharge points of treatment plants for $Q_{min,daily}^7$ river flows in study period 1 and for the waste load conditions of year 1991.

Plant	Discharge Point		D_{av}	DOD	D_{req}	
	Current	Proposed			BOD_u	$(NH_3)_{tot}$
Ault	AE	AB'	4.500	0.449	-0.265	3.373
Maxville	U	T'	8.526	0.525	6.538	4.776
(Williamsburg)	(AG)	(AF)'	(11.217)	(0.461)	(6.530)	(9.149)
(Winchester)	(AD)	(Y)'	(12.949)	(0.445)	(6.432)	(11.913)
Winchester	AD	N'	17.570	0.338	5.079	9.422

Table 6.21: Change of the degree of treatment of the treatment plants for $Q_{min,daily}^7$ river flows in study period 1 and for the waste load conditions of year 1991.

Plant	Node	Type	$\%DO_s$ (%)	BOD_u (mg/l)	$(NH_3)_{tot}$ (mg/l)
Williamsburg	AG	EA	63.905	8.314	1.402

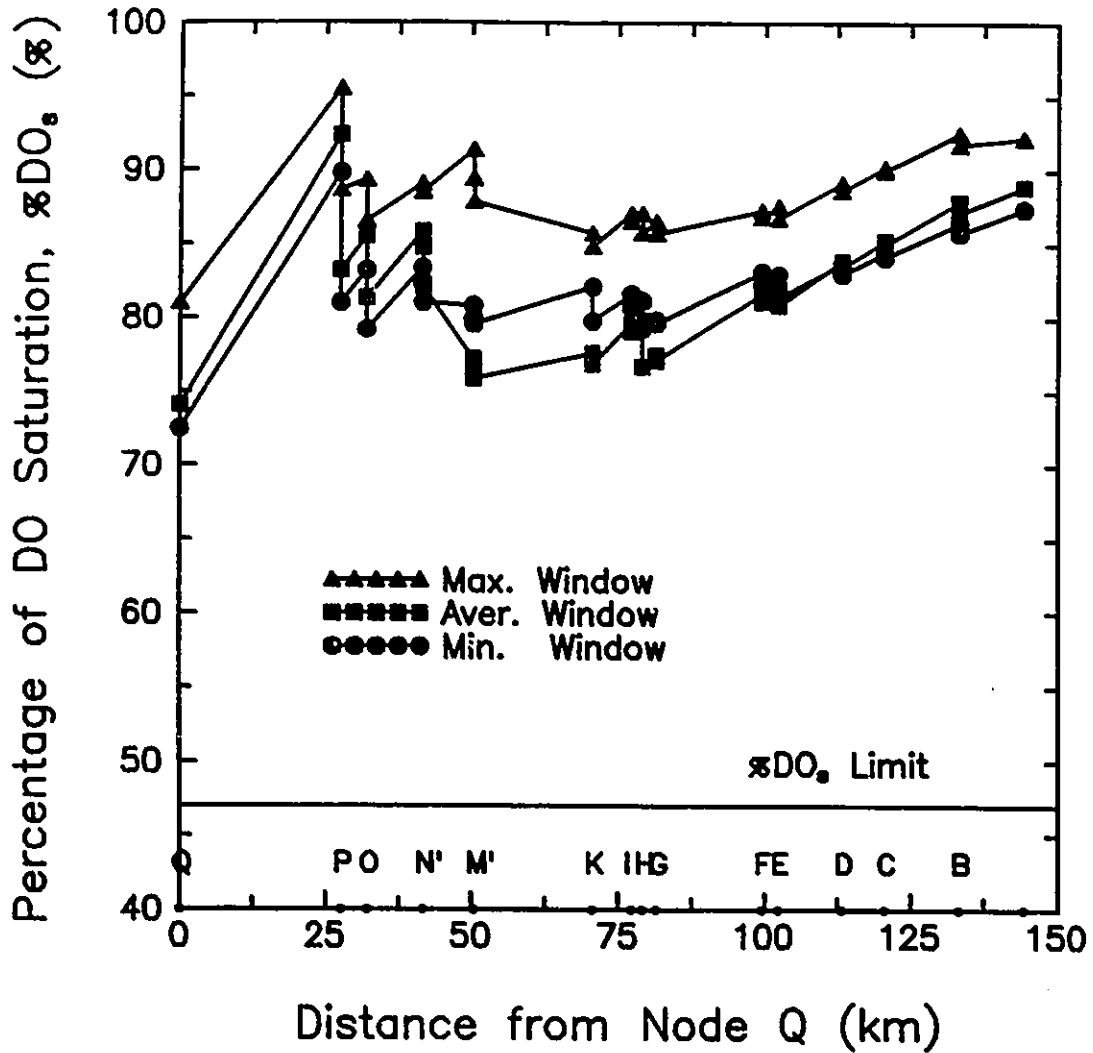


Figure 6.29: Predicted variation of the percentage of DO saturation along the South Nation River in study period I after the application of the optimal discharge scheme under Q_{min}^{daily} river flows and conditions of 1991 using the rates of set 1.

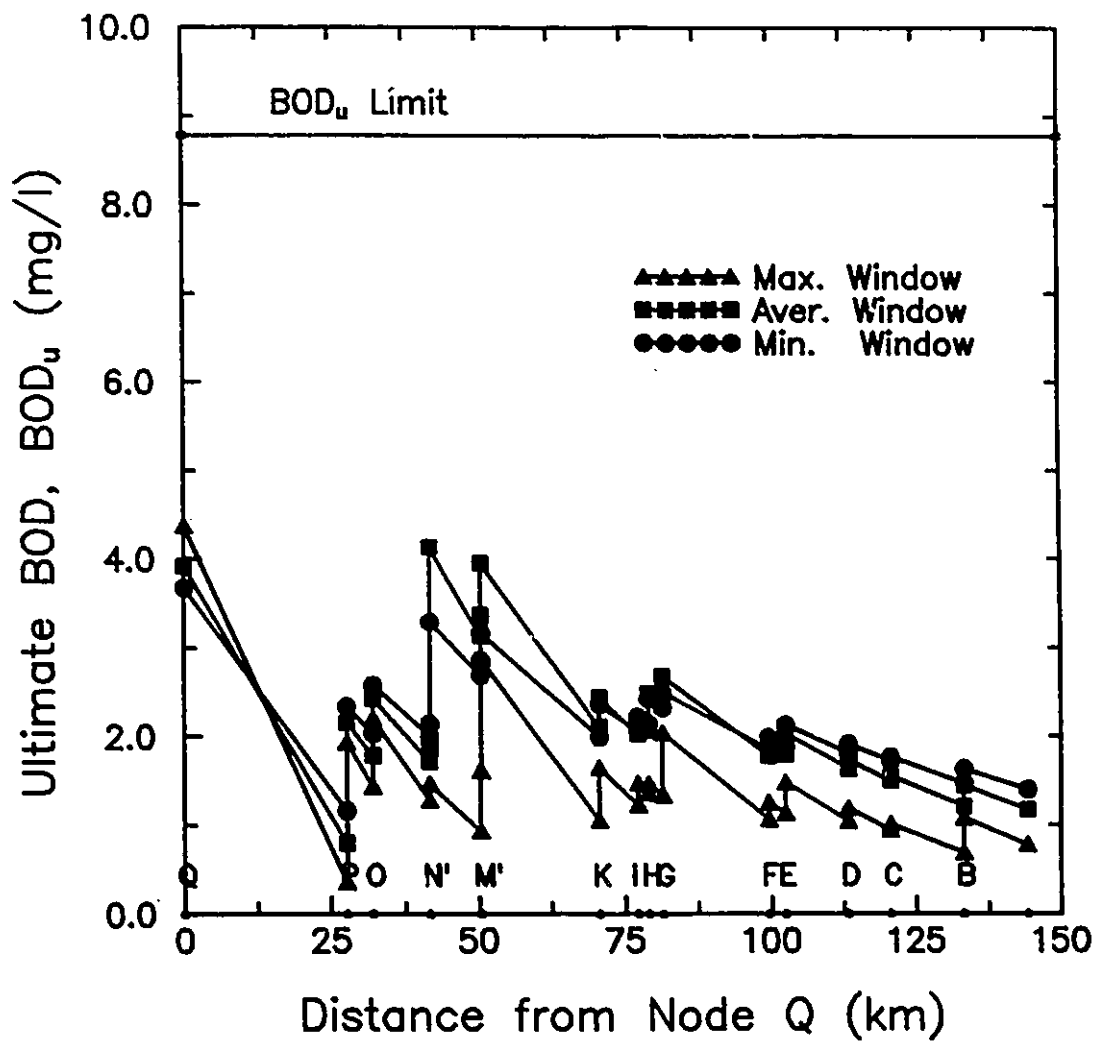


Figure 6.30: Predicted variation of the ultimate BOD concentration along the South Nation River in study period 1 after the application of the optimal discharge scheme under $Q_{min,daily}^7$ river flows and conditions of 1991 using the rates of set 1.

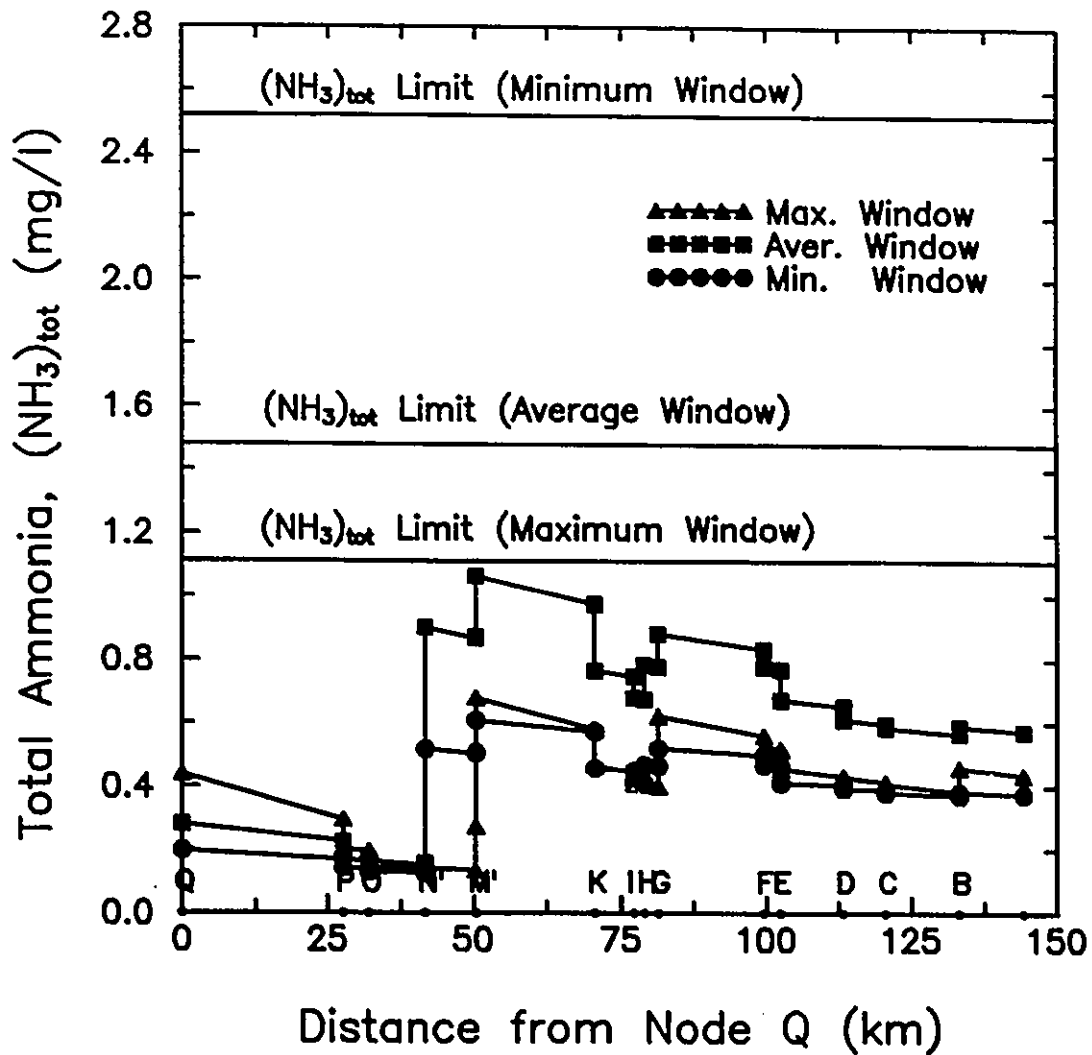


Figure 6.31: Predicted variation of total ammonia concentration along the South Nation River in study period 1 after the application of the optimal discharge scheme under $Q_{min,daily}^7$ river flows and conditions of 1991 using the rates of set 2.

d. $Q_{min,daily}^7$ river flows, 2011. The simulation results for the maximum window in future conditions showed a violation of the PWQO standards for the "remote" plants of Fournier, Moose Creek, Maxville, Russell, Williamsburg and Winchester communities and of Ault industry, as well as from the "interior" plants of Embrun and St. Isidore communities.

The optimization procedure was initially applied for the remote communities and gave the following results. The plants of Fournier and St. Isidore communities would have to discharge their effluent during the average window. The plants of Maxville and Winchester communities would have to relocate to nodes T' and N', respectively, similarly to the optimal scheme for present conditions, and should also discharge during the minimum and average windows. The plants of Moose Creek and Williamsburg communities would have to relocate to nodes F' and T' and could both discharge during the average window. However, due to the significant distance between the initially considered and the suggested from the optimization nodes, it was preferred for both plants to change their treatment to the extended aeration process. In that case, the Moose Creek and Williamsburg plants would be allowed to discharge during the average and minimum windows respectively.

The Ault and Russell plants should relocate to nodes AB' and Y', correspondingly, and also discharge in the minimum window. Finally, the Embrun plant should replace its treatment by using the extended aeration process and also discharge in the average window.

The results of the optimization are presented in Tables 6.22 to 6.24. The resulting water quality for the South Nation River under the conditions of all windows is shown in Figures 6.32 to 6.34.

Table 6.22: Staging of the discharge of the treatment plants effluent during the optimum windows of study period I for $Q_{min,daily}^7$ river flows and the waste load conditions of year 2011.

Plant	Node	Annual V_w (m ³)	Maximum Allowed V_w (m ³)			Q_w (m ³ /s) (window)
			DOD	BOD_u	$(NH_3)_{tot}$	
Ault	AB'	4.55x10 ⁵	1.72x10 ⁶	-3.29x10 ⁶	5.90x10 ⁵	0.310 (min.)
Embrun	Z	1.55x10 ⁶	1.87x10 ⁷	3.45x10 ⁶	3.06x10 ⁶	0.528 (aver.)
Fournier	R	5.07x10 ⁴	1.45x10 ⁶	1.02x10 ⁵	7.36x10 ⁴	0.017 (aver.)
Maxville	T'	6.13x10 ⁴	9.72x10 ⁵	7.81x10 ⁴	1.07x10 ⁵	0.042 (min.)
Moose Creek	X	1.01x10 ⁵	6.77x10 ⁵	1.28x10 ⁵	1.22x10 ⁵	0.035 (aver.)
(Moose Creek)	(F')	(1.01x10 ⁵)	(6.55x10 ⁷)	(5.67x10 ⁶)	(5.28x10 ⁵)	(0.035) (aver.)
Russell	Y'	8.65x10 ⁵	1.56x10 ⁷	1.34x10 ⁶	1.54x10 ⁶	0.589 (min.)
St. Isidore	S'	1.77x10 ⁵	5.48x10 ⁶	3.73x10 ⁵	2.40x10 ⁵	0.060 (aver.)
(Williamsburg)	(AF')	(7.08x10 ⁴)	(1.56x10 ⁶)	(1.10x10 ⁵)	(7.88x10 ⁴)	(0.024) (aver.)
Williamsburg	AG	7.08x10 ⁴	3.42x10 ⁵	7.34x10 ⁴	1.75x10 ⁵	0.048 (min.)
Winchester	N'	7.17x10 ⁵	3.26x10 ⁷	2.17x10 ⁶	1.17x10 ⁶	0.244 (aver.)

Table 6.23: Relocation of the discharge points of treatment plants for $Q_{min,daily}^7$ river flows in study period 1 and for the waste load conditions of year 2011.

Plant	Discharge Point		D_{av}	D_{req}		
	Current	Proposed		DOD	BOD_u	$(NH_3)_{tot}$
Ault	AE	AB'	1.933	0.511	-0.267	1.489
Maxville	U	T'	8.323	0.525	6.538	4.776
(Moose Creek)	(X)	(F')	(325.549)	(0.505)	(5.825)	(62.475)
Russell	AA	Y'	9.658	0.537	6.241	5.411
(Williamsburg)	(AG)	(AF')	(10.176)	(0.461)	(6.530)	(9.149)
Winchester	AD	N'	15.351	0.338	5.076	9.352

Table 6.24: Change of the degree of treatment of the treatment plants for $Q_{min,daily}^7$ river flows and for the waste load conditions of year 2011.

Plant	Node	Type	$\%DO_5$	BOD_u	$(NH_3)_{tot}$
			(%)	(mg/l)	(mg/l)
Moose Creek	X	EA	64.686	7.955	1.307
Embrun	Z	EA	70.297	6.120	0.918
Williamsburg	AG	EA	61.86	8.641	1.488

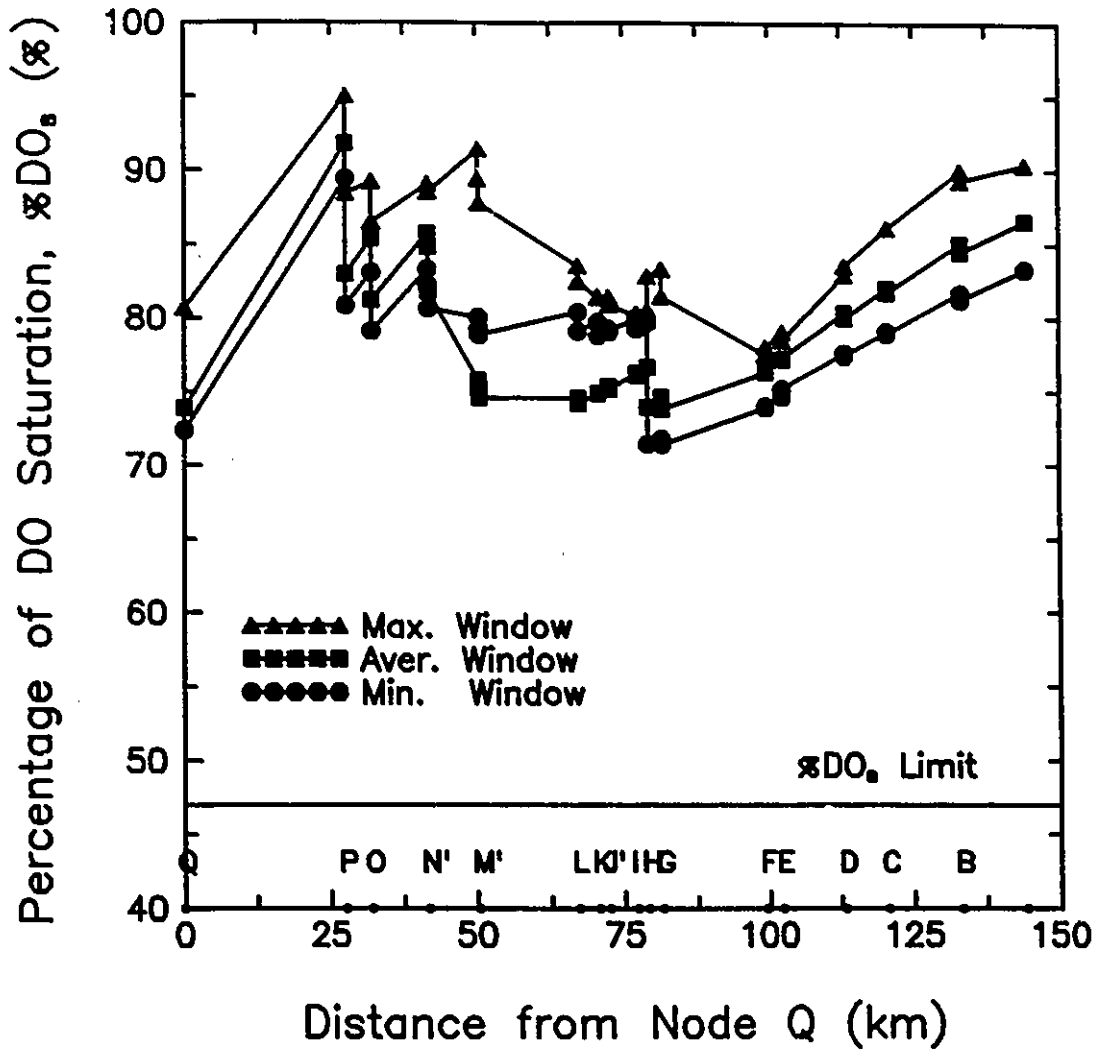


Figure 6.32: Predicted variation of the percentage of DO saturation along the South Nation River in study period 1 after the application of the optimal discharge scheme under $Q_{min,daily}^7$ river flows and conditions of 2011 using the rates of set 1.

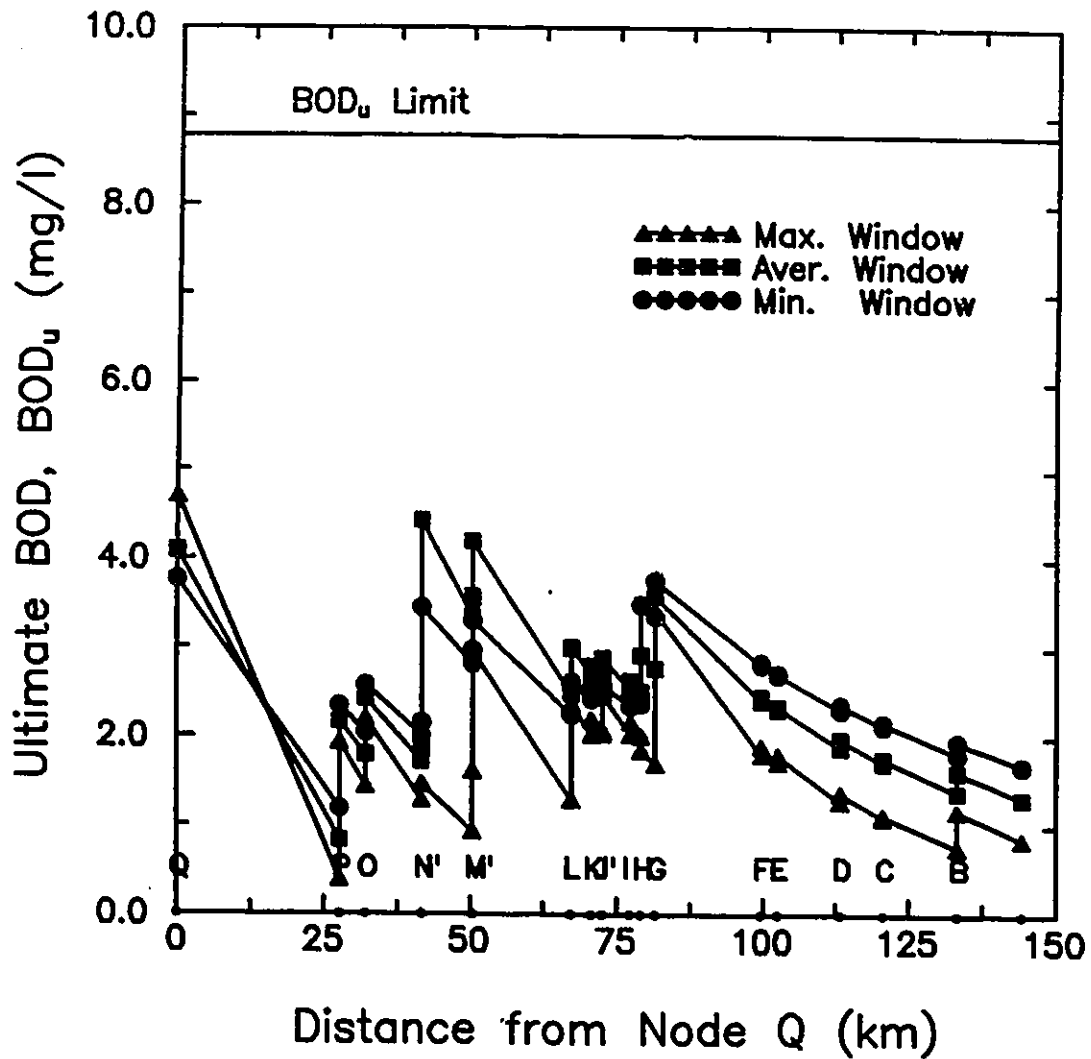


Figure 6.33: Predicted variation of the ultimate BOD concentration along the South Nation River in study period I after the application of the optimal discharge scheme under $Q_{min,daily}^7$ river flows and conditions of 2011 using the rates of set 1.

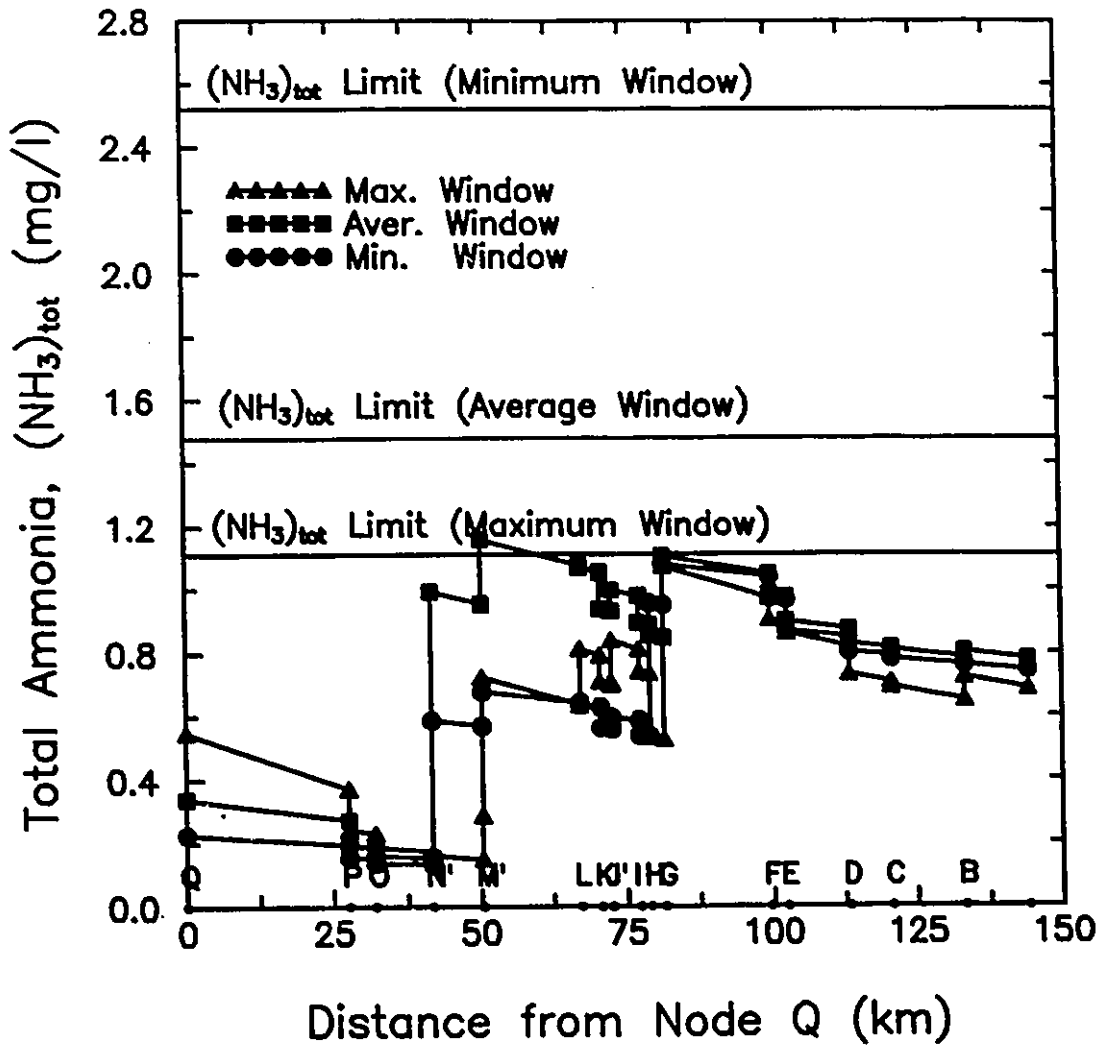


Figure 6.34: Predicted variation of total ammonia concentration along the South Nation River in study period I after the application of the optimal discharge scheme under $Q_{min,daily}^7$ river flows and conditions of 2011 using the rates of set 2.

Chapter 7

Conclusions - Future Work

In the present study an object-oriented analysis has been performed for the management of water quality in a river system. Emphasis has been placed on the control of its point-source pollution. An object-oriented design has been performed following the above analysis, and has been implemented in terms of the object-oriented computer language *Smalltalk*. As a result, a user-friendly and easily extended software tool has been developed. This tool has been validated for the case of the South Nation River Watershed.

7.1 Object-Oriented Programming

In the analysis performed, the physical entities of the river system with its point-source pollutants as well as the conceptual entities involved in its:

- flow analysis
- water quality simulation
- and control strategies

have been represented through objects. Distributing appropriately the properties and the

responsibilities of these objects and specifying the interactions and collaborations among them, the tasks involved in the management study could be efficiently accomplished.

The object-oriented approach has been also used in the design of a data-management component and of a graphical-user interface. These components allow:

- input of the required data for a given study from data files
- output in generated report files
- visualization of the river system
- interactive manipulation of the river-system properties
- flow control of the whole analysis.

The definition and design of all objects has been made keeping in mind their possible future development and the requirements for refinement of their responsibilities. In that way the analysis resulted in a system of well-specified hierarchies that could be easily extended to describe additional aspects of water quality management. During the implementation of the analysis the appropriate definition of the objects and their organization in well-structured hierarchies resulted in the reusability of the code and facilitated the gradual development of the software tool.

Another basic feature of the tool has been its user-friendliness and flexibility that allows the user to (i) check gradually the results of the analysis, (ii) specify new conditions at any stage of it, and (iii) repeat the steps performed so far, thus being able to test several alternative scenarios and generally control the flow of the analysis.

7.2 Water Quality Management in a River System

The features of the object-oriented analysis from the quality management perspective are summarized in the following:

Low flows can be estimated for each gauge station in the river system on a daily basis for a duration of 7 and 30 days and a recurrence interval of 20 years. These flows can be used for a conservative estimate of the maximum assimilative capacity of the river at the corresponding locations. The interval during which the above mentioned capacity is available can be considered as the *optimum window for the discharge of point-sources*. Based on these windows (for all the stations), overall optimum windows for the whole system can be estimated along with the corresponding minimum daily flows at each station.

The so-calculated flows can be used as the *design flows* for the simulations of water quality. A linear regression analysis between them and the drainage areas of the stations can be performed and, if acceptable, it can define a relationship for the estimation of the design flows for every point in the system. Otherwise design flows can be estimated applying a flow ratio method. The travel times can be estimated using a Manning-type equation.

The total loads of constituents-indicators of the water quality, 5-day BOD, the DO and the ammonia nitrogen, have been estimated at various quality stations of the river system from their data for any selected years. The effluents of the treatment plants of the communities and the industries have been considered as point-sources of pollutant load. The contribution of this load, as well as the load of the nonpoint sources, to the total one has also been estimated. The background concentrations of the above constituents are estimated as a percentile of the total one for each station. Their average values over all stations have been used as the background concentrations of the constituent in the river.

The maximum allowable concentrations of the treatment plants, that satisfy the in-pipe standards for the toxic undissociated ammonia and hydrogen sulphide, can be estimated from the expected temperature and pH of the effluents, by applying relationships that describe the dissociation phenomena. The required dilution at the discharge point of each plant effluent can be also estimated in order to satisfy the river standards for the toxic

constituents.

The discharge of each plant can be studied in the absence of the discharge of any other plant, and complete mixed conditions can be assumed between the effluent and the receiving water. The available dilution of the river at the same points can be also estimated. If such dilution is proven insufficient to provide the dilution requirements, a violation of the river standards in the toxic constituents is identified. Such situation suggests that a control strategy should be applied at the corresponding plants, and identification of the problematic plants is achieved.

For a more representative estimation of the water quality, the simulation of water quality in the river system can be performed. Steady-state conditions are assumed to occur during the optimum discharge windows. The river flows are considered equal to the corresponding design flows and the discharge of the annual volume of the plants can be considered to occur at an equal rate during the window intervals. The concentrations of the effluents were set equal to the typical expected concentrations for the specific type of treatment. The modelled constituents are the DO, the 5-day BOD, and the total ammonia nitrogen, which are oxidized, decayed or exchanged with the atmosphere and the sediment. Their transport in the system has been represented through a node-reach model. The model takes into account the plants effluent discharge and its travel along with the base flow (corresponding to the design river flows), which is assumed to have the background concentrations of the modelled constituents. The simulation results reveal the water quality patterns in the river and show the locations where the river quality standards are not satisfied.

An optimization procedure can be then applied for the improvement of quality in the whole system up to a level that will satisfy the river quality standards. This procedure considers reallocation of the point-source load for the remote communities, staging of the effluent discharge in the optimum discharge windows, and the increase of the degree of treatment in treatment plants. The alternative treatment processes that have been

considered include: (i) use of lagoons, (ii) the New Hamburg process, and (iii) extended aeration along with the activated sludge process.

7.3 Study of the South Nation River

The above analysis was applied to the study of the South Nation River with the software tool developed for this purpose. The river accepts the load of the treatment plants from many municipalities and from a few industries. All treatment plants apply treatment with lagoons except of those industries which prior to the lagoon system have custom-designed treatment plants. The water quality of the river was studied in the periods of spring and fall. However, simulations during fall showed that there was a general violation of the DO criterion, so the only period that was considered for discharge was the spring period.

The estimated optimum discharge windows in this period range from the end of March / middle of April (minimum window) until middle of March / end of April or middle of May (maximum window). The minimum daily flows were found to be acceptable when estimated with the flow area regression equation. The travel times from the furthest upstream node of the system to the junction of the South Nation with the Ottawa River were found to range from 11 to 20 and from 8 to 14 days at $Q_{min,daily}^7$ and $Q_{min,daily}^{30}$, respectively. The background concentrations of ultimate BOD, DO and of total ammonia were estimated as equal to 3.4, 8.7 and 0.11 mg/l, respectively.

The comparison of the maximum allowable concentrations of the toxic substances in the plants effluent with the corresponding expected concentration from the various treatment processes showed that the quality in all in-pipe plant effluents resulted in violation of the total undissociated hydrogen sulphide standard. From the comparison of the available dilution factor with the required one it was shown that similar violations were

occurring with respect to the river standards. This suggests that a post aeration cell should be used to improve both the effluent and river qualities with respect to sulphide. The corresponding analysis for the free ammonia constituent showed that under certain pH conditions, the in-pipe effluent could violate the in-pipe standards while similar violations in the river quality were expected to occur under any conditions for the plants of Winchester and Maxville and for certain conditions for a series of other plants.

The simulation of the river quality in presence of all the plants discharging has been used to identify plants violating the river-quality standards of the modelled constituents. Most of the identified plants have been found to discharge in the Castor River. The optimization procedure has been applied for them. The results suggested that the Winchester and Maxville plants as well as the industry of Ault should relocate their discharge points. The Williamsburg plant should upgrade its treatment facilities. Under $Q_{min.daily}^7$ future conditions the communities of Embrun and Moose Creek should also upgrade their treatment. Under certain of the simulation conditions the staging of the effluents discharge in the optimum windows was sufficient for the agreement of the river quality with water quality standards.

7.4 Future Work

A few suggestions for future work are listed below:

1. Code implementations could be ported to more powerful operating systems. This is a relatively easy task due to the availability of *Smalltalk* for various operating systems. The first candidates should be UNIX and OS/2, since the inherent limitations of the widely used MS-DOS/MS-Windows combination are known. The code running under UNIX or OS/2 could take advantage of the speed and capabilities of these operating systems. Thus, complicated large watersheds could

be efficiently described and studied in detail. Software development tools already available for the above mentioned operating systems could also help the development of the present code. Furthermore, powerful Geographic Information Systems (GIS) and databases are available (mainly for UNIX) that could be interfaced to the code as will be explained below.

2. Interfacing of the software with a database and a GIS. The use of a relational or object-oriented database would offer a better organization of the large amount of data that are required in a water quality management study. The use of a GIS could allow better visualization of the river system or watershed under study. It could also be of great help in the presentation of the results generated.
3. Integration with models for the prediction and control of the nonpoint source pollution in a river. This could further extend the watershed management capabilities of the present tool.
4. Integration with alternative models of water quality forecasting and river management. The models could be of an increased degree of complexity and detail with respect to the hydrodynamics of the river system, the modelled constituents and the quality control strategies.
5. Given the importance of the model parameter values, a module of sensitivity analysis and autocalibration could be included.
6. Inclusion of an advising (expert) module. This could assist among others in the evaluation of input data, in the selection of the appropriate relationships or models to be applied, as well as in the selection of an optimal management scheme for the particular river system or watershed.
7. Combination of the developed model for the river water quality with a predictive model for the groundwater quality.
8. Addition of *stochastic modelling* to account for the variation of the model inputs over time and to provide a better understanding and evaluation of their outputs.

The models and modules referenced in the above recommendations could be selected among the ones already available as public-domain or commercial tools. Such tools could be interfaced with the existing code and thus form a coherent software system. Alternatively, modules could be developed within the existing code taking advantage of its object-oriented design, as was done for the river quality model implemented for this study.

The various directions for future work and development are imposed by the inherent complexity of the quality management of a river system. The present work outlines the potential of object orientation in the development of tools that will aid in the study and policy-making of water resources systems.

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Appendix A

Notation for Object-Oriented Analysis and Design

A variety of object-oriented analysis and design methods have been proposed for the description of a system (Graham, 1994). Each one defines and evolves with its development its own notation as a means to express its underlying principles and facilitate the communication between the analyst and programmer involved in the study of the system (Graham, 1994; Booch, 1994, Wirfs-Brock et al., 1990).

The notation applied in the present work is based on the one suggested by Coad and Yourdon (1993). Their notation and method as well as CRC are the two most widely applied approaches. This notation is characterized by (i) its simplicity, (ii) a reasonable completeness, and (iii) a continuity in representation from the analysis to the design phases. It has influenced other notations of both ternary and hybrid methods (data description of OMT method, SOMA method). A detailed description of this notation is given in the following.

One basic symbol is the *class-and-object* symbol, which consists of a rectangle with rounded corners being surrounded by a similar but lighter outer rectangle and represents a *concrete class* (a class with one or more instances). The inner box has three areas showing the name of the class and a list of attributes as well as a list of operations of

the class instances. This symbol without the outer rectangle represents an *abstract class* and its contents define at least part of the state and behaviour of its subclass instances.

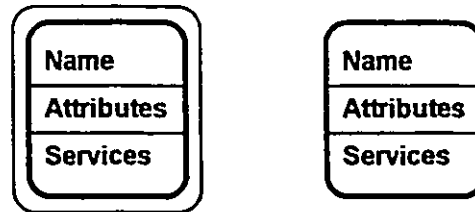


Figure A.1 : Notation for concrete classes with their instances (left) and for abstract classes (right).

The classification of objects in inheritance hierarchies is represented by the *generalization-specialization* symbol, which is a line with a semicircle between the more general and one or more of the more specialized classes, with the concave part pointing to the general class. Note that this line should reach the inner rectangle when joining class-and-object symbols.

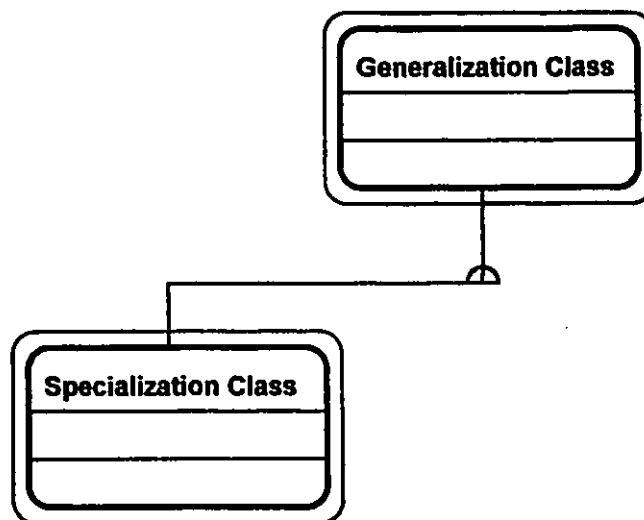


Figure A.2 : Notation of classification structure or class hierarchy.

The composition structure is represented by a line between the whole (assembly) object and the part object with a triangle pointing to the whole and numbers near the ends of the line showing the number (or range of number) of parts object the whole has, and the number (or range of number) of assemblies an object is part of. Alternatively, *the extended relational analysis notation* may be followed. According to the latter, at each end of the linking line in the direction of the object there is a *modality* and a *multiplicity* symbol. *Modality* refers to whether the relationship with the object linked is necessary (the symbol is O and read as must be) or optional (the symbol is | and read may be). *Multiplicity* refers to whether the relationship is with many (symbol is crow) or one (symbol is one) linked objects. The same notation applies for *container-contents* and *collection-members* relationships.

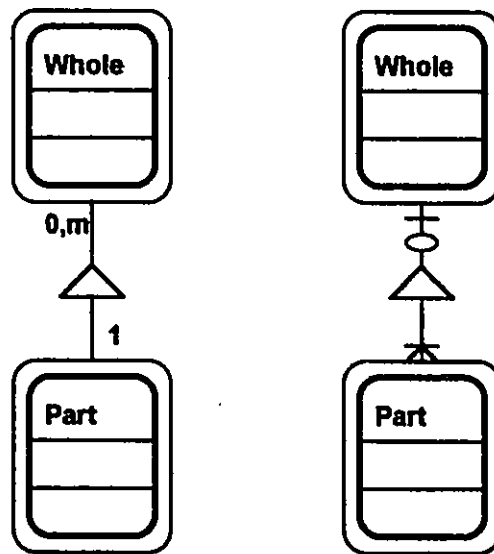


Figure A.3 : Notation for composition structure.

A more general association between objects of the same or of two different classes, which means that an instance of one class knows about one or more instances of the same or of a different class, is shown with a single line connecting them. At the end of the line near the object that needs to know about other objects, there is the number or range of those other objects.

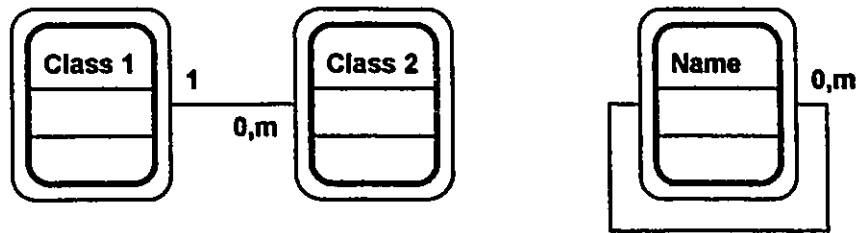


Figure A.4 : Connection between objects of different or of the same class.

A message that is passed from the *sender object* to the *receiver object*, that may be an instance of the same or of a different class or a class itself, and the response of the receiver back to the sender is shown as a shaded arrow. An optional label that may consist of a scenario letter and a message number or a name describing the message may be on the arrow. Note that this is an actual implementation message and not a declaration of a client-server contract. Notice also that in both connection and message passing symbols, the line may end in the outer or inner of the class-and-object symbol, if it refers to an instance or a class (usually abstract class).

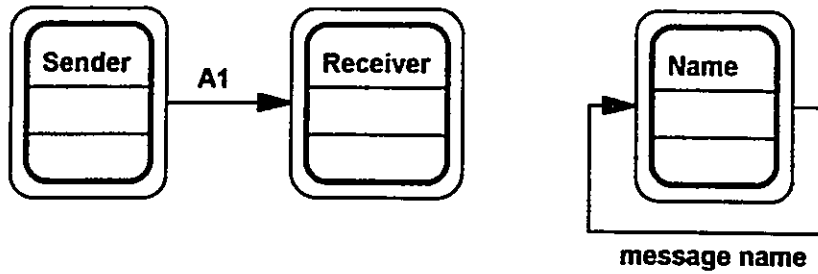


Figure A.5 : Notation for message passing between instances of different or of the same class.

A part of the model with the related classes that is hidden from view in a schematic diagram of the model is represented by an ellipsis (triple dot).

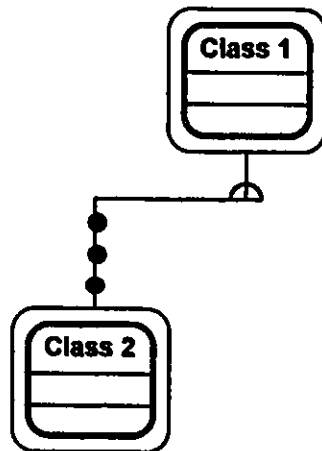


Figure A.6 : Notation for a hidden-from-view part of model.

A *subject* i.e., a subsystem of several semantically related objects that offers a view of higher abstraction level, is shown in its collapsed form as a square box with the name of the subject and a reference number. Alternatively, it may show a list of its classes or the

actual part of the model it represents with all the objects, relationships and structures. Message passing between subjects is also allowed.

1. Subject Name

Figure A.7 : Notation for subjects of the domain system.

A *design component* is shown in its collapsed form as a shaded box with the name and a reference tag for the component. This tag may be PDC, HIC, TMC or DMC, standing correspondingly for a Problem Domain Component, a Human Interaction Component, a Task Management Component or a Data Management Component. Simiarily to the subject symbol, it can reveal its classes or a detailed description of its part in the model.

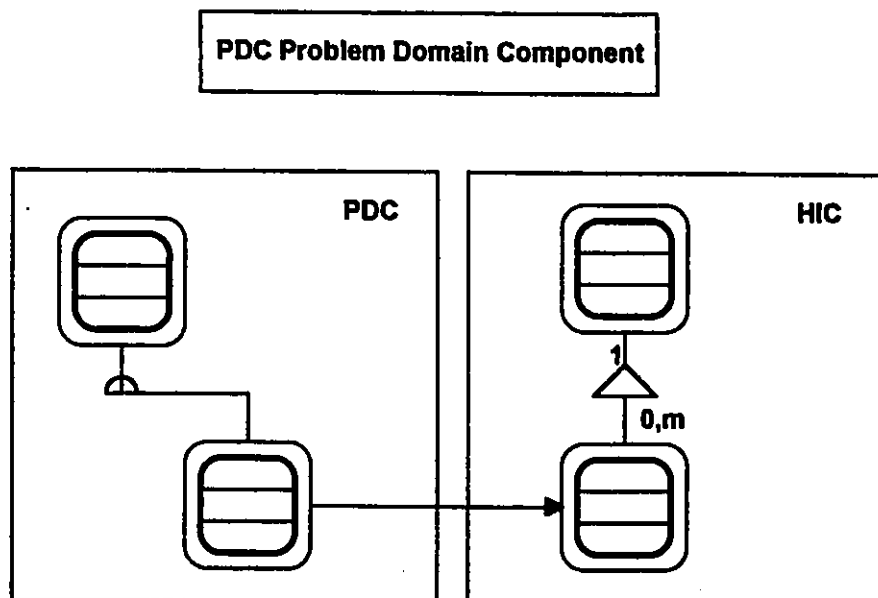


Figure A.8 : Notation for design components of the system in their collapsed (left) and fully expanded form (right).

Appendix B

Results of the South Nation River Case Study

Table B.1: Flow gauge stations in the river system with adequate data for the low flow analysis, the years of their record, and their overall tributary area (Droste and Gibbons, 1992).

Station Code	River Name (Location)	Years of record	Area (km²)
02LB005	South Nation River (Plantagenet)	1915-1986	3820
02LB006	Castor River (Russell)	1948-1986	449
02LB007	South Nation River (Spencerville)	1948-1986	243
02LB008	Bear Brook (Bourget)	1976-1986	436
02LB013	South Nation River (Casselman)	1972-1986	2400
02LB017	North Branch South Nation River (Heckston)	1977-1986	106
02LB020	South Castor River (Kenmore)	1978-1986	201
02LB022	Payne River (Berwick)	1976-1986	150

Table B.2: Water quality sampling stations in the river system with adequate data for the period of 1980-1991 (Droste and Gibbons, 1992).

MOE Station Code	River Name (Location)
718-2070-100-02	South Nation River (Casselman)
18-2070-110-02	South Nation River (Chesterville)
18-2070-020-02	South Nation River (Plantagenet)
18-2070-140-02	Castor River (Russell, Confluence 5)
18-2070-145-02	Castor River (Russell, Confluence 3)
18-2070-040-02	Scotch River (St. Isidore)
18-2070-060-02	Scotch River (St. Isidore, Upper)

Table B.3: General characteristics of the treatment facilities in the South Nation River Watershed (Droste and Gibbons, 1992).

Treatment Plant	Served Population		Annual Waste Volume (10^3 m^3)		Type	Volume (10^3 m^3)	Area (ha)
	1991	2011	1991	2011			
Bourget	-	1577	-	230.2	-	-	-
Casselman	2400	6368	365.0	944.3	Two Cell Lagoon	243.0	16.2
Chesterville	1500	1656	296.0	318.8	Two Cell Lagoon	185.0	11.9
Crysler	-	1500	-	219.0	-	-	-
Embrun	3500	10874	474.5	1551.1	Six Cell Lagoon	800.0	53.4
Fournier	-	349	-	50.9	-	-	-
Maxville	800	810	59.9	61.3	Two Cell Lagoon	164.0	9.8
Moose Creek	-	696	-	101.6	-	-	-
Plantagenet	900	1098	225.2	254.1	One Cell Lagoon	100.0	6.9
Russell	1800	6404	193.1	865.3	Two Cell Lagoon	237.0	15.8
Spencerville	350	475	51.1	69.4	Two Cell Lagoon	116.0	6.9
St. Albert	-	948	-	138.4	-	-	-
St. Isidore	700	1040	127	176.7	Three Cell Lagoon	220.0	14.6
Williamsburg	439	485	64.1	70.8	Two Cell Lagoon	75.0	4.9
Winchester	220	2642	629.6	717.2	Five Cell Lagoon	470.0	19.0
Ault	-	-	193.8	455.0	Aer. Biol. Treatment Sec. Treatment,	275.6	10.1
Nestles	-	-	225.9	225.9	Aerated Lagoon	-	-
St. Albert's Cheese	-	-	-	22.6	-	-	-

Table B.4: The optimum discharge windows, the minimum river flows, and the implied dilution volumes of all river gauge stations for $Q_{i,low}^7$ flows in the spring study period.

Station Code	Discharge Window	Δt_{opt} (days)	$Q_{min,low}^7$ (m ³ /s)	V (10 ⁶ m ³)
02LB005	March 28 - April 13	17	30.00	44.10
02LB006	March 28 - April 24	28	1.78	4.30
02LB007	March 24 - May 10	48	0.91	3.88
02LB008	March 23 - May 9	48	1.19	4.95
02LB013	March 19 - April 18	31	11.02	29.50
02LB017	March 24 - April 22	30	0.50	1.31
02LB020	March 24 - April 25	33	0.82	2.35
02LB022	March 25 - April 27	34	0.46	1.34

Table B.5: The optimum discharge windows, the minimum river flows, and the implied dilution volumes of all river gauge stations for $Q_{t,low}^{30}$ flows in the spring study period.

Station Code	Discharge Window	Δt_{opt} (days)	$Q_{min,low}^{30}$ (m ³ /s)	V (10 ⁶ m ³)
02LB005	March 26 - April 15	21	59.31	108.00
02LB006	March 28 - April 14	18	6.79	10.60
02LB007	March 22 - April 18	28	3.34	8.09
02LB008	March 22 - April 22	32	5.23	14.50
02LB013	March 16 - April 17	33	23.88	68.10
02LB017	March 16 - April 22	38	0.62	2.04
02LB020	March 16 - April 17	33	1.85	5.26
02LB022	March 16 - April 24	40	0.85	2.95

Table B.6: The optimum discharge windows, the minimum river flows, and the implied dilution volumes of all river gauge stations for $Q_{t,low}^7$ flows in the fall study period.

Station Code	Discharge Window	Δt_{opt} (days)	$Q_{min,low}^7$ (m ³ /s)	V (10 ⁶ m ³)
02LB005	November 10 - December 15	36	0.54	1.67
02LB006	November 20 - December 13	24	0.67	1.40
02LB007	November 21 - December 21	31	0.08	0.21
02LB008	November 11 - December 28	48	0.32	1.31
02LB013	November 16 - December 13	28	1.33	3.22
02LB017	December 11 - December 28	18	0.05	0.08
02LB020	September 4 - December 28	116	0.07	0.68
02LB022	November 9 - December 19	41	0.09	0.32

Table B.7: The optimum discharge windows, the minimum river flows, and the implied dilution volumes of all river gauge stations for $Q_{i,low}^{30}$ flows in the fall study period.

Station Code	Discharge Window	Δt_{opt} (days)	$Q_{min,low}^{30}$ (m ³ /s)	V (10 ⁶ m ³)
02LB005	November 17 - December 12	26	2.12	4.75
02LB006	November 16 - December 17	32	0.64	1.76
02LB007	November 27 - December 17	21	0.22	0.40
02LB008	November 4 - December 17	44	0.46	1.76
02LB013	November 14 - December 17	34	1.45	4.26
02LB017	November 16 - December 17	32	0.04	0.12
02LB020	September 16 - December 17	93	0.08	0.63
02LB022	November 16 - December 15	30	0.19	0.49

Table B.8: Minimum daily $Q_{t,low}^7$ flows, $Q_{min,daily}^7$, of all river gauge stations for all windows in study period 1.

Station Code	A_D (km ²)	$Q_{min,daily}^7$ (m ³ /s)		
		Minimum Window	Average Window	Maximum Window
02LB005	3820.0	31.26	15.08	7.16
02LB006	449.0	1.88	1.19	0.40
02LB007	243.0	1.54	0.92	0.46
02LB008	436.0	2.65	1.66	0.67
02LB013	2400.0	16.06	8.51	5.08
02LB017	106.0	0.66	0.35	0.21
02LB020	201.0	1.52	0.82	0.49
02LB022	150.0	0.89	0.46	0.39

Table B.9: Minimum daily $Q_{i,low}^{30}$ flows, $Q_{min.daily}^{30}$, of all river gauge stations for all windows in study period 1.

Station Code	A_D (km ²)	$Q_{min.daily}^{30}$ (m ³ /s)		
		Minimum Window	Average Window	Maximum Window
02LB005	3820.0	66.41	29.73	15.58
02LB006	449.0	6.84	2.15	1.45
02LB007	243.0	3.76	2.32	1.71
02LB008	436.0	6.79	3.78	2.00
02LB013	2400.0	26.89	22.50	19.33
02LB017	106.0	0.68	0.67	0.60
02LB020	201.0	2.02	1.71	1.36
02LB022	150.0	1.30	1.06	0.86

Table B.10: Minimum daily $Q_{t,low}^7$ flows, $Q_{min.daily}^7$, of all river gauge stations for all windows in study period 2.

Station Code	A_D (km ²)	$Q_{min.daily}^7$ (m ³ /s)		
		Minimum Window	Average Window	Maximum Window
02LB005	3820.0	1.03	0.40	0.13
02LB006	449.0	0.68	0.31	0.14
02LB007	243.0	0.13	0.07	0.02
02LB008	436.0	0.46	0.40	0.26
02LB013	2400.0	1.34	0.95	0.59
02LB017	106.0	0.06	0.03	0.01
02LB020	201.0	0.13	0.07	0.08
02LB022	150.0	0.16	0.08	0.03

Table B.11: Minimum daily $Q_{t,low}^{30}$ flows, $Q_{min,daily}^{30}$, of all river gauge stations for all windows in study period 2.

Station Code	A_D (km ²)	$Q_{min,daily}^{30}$ (m ³ /s)		
		Minimum Window	Average Window	Maximum Window
02LB005	3820.0	2.14	1.77	0.99
02LB006	449.0	0.80	0.67	0.18
02LB007	243.0	0.23	0.16	0.03
02LB008	436.0	0.53	0.56	0.48
02LB013	2400.0	1.77	1.68	0.53
02LB017	106.0	0.05	0.05	0.00
02LB020	201.0	0.08	0.08	0.08
02LB022	150.0	0.23	0.18	0.11

Table B.12: Minimum daily flows of all junction nodes (jn) and discharge points (dp) in the system during all optimum windows in study period 1. Both $Q_{min,daily}^7$ and $Q_{min,daily}^{30}$ flows were calculated with regression coefficients.

Node	Type	$Q_{min,daily}^7$ (m ³ /s)			$Q_{min,daily}^{30}$ (m ³ /s)		
		Min.	Aver.	Max.	Min.	Aver.	Max.
A	jn	30.03	14.89	7.52	60.84	31.99	20.22
B	dp	29.43	14.60	7.37	59.62	31.35	19.82
C	jn	28.12	13.95	7.04	56.97	29.95	18.94
D	jn	27.37	13.58	6.86	55.45	29.16	18.43
E	jn	23.98	11.89	6.01	48.57	25.54	16.14
F	jn	20.24	10.04	5.07	41.00	21.56	13.63
G	dp	18.54	9.19	4.64	37.55	19.74	12.48
H	jn	18.49	9.17	4.63	37.46	19.69	12.45
I	jn	12.43	6.16	3.11	25.17	13.24	8.37
J'	dp	11.09	5.50	2.78	22.47	11.82	7.47
J	dp	11.09	5.50	2.78	22.46	11.81	7.47
K	jn	11.03	5.47	2.76	22.35	11.75	7.43
L	dp	9.57	4.75	2.40	19.38	10.19	6.44
M'	dp	8.17	4.05	2.05	16.56	8.71	5.50
M	dp	8.17	4.05	2.05	16.54	8.70	5.50
N	jn	7.52	3.73	1.88	15.23	8.01	5.06
O	jn	6.30	3.13	1.58	12.77	6.71	4.24
P	jn	3.88	1.92	0.97	7.85	4.13	2.61
Q	dp	1.87	0.93	0.47	3.79	1.99	1.26
U	dp	0.02	0.01	0.00	0.03	0.02	0.01
T	jn	0.34	0.17	0.08	0.69	0.36	0.23
S'	dp	1.50	0.75	0.38	3.04	1.60	1.01
S	jn	1.49	0.74	0.37	3.03	1.59	1.01
AD	dp	0.06	0.03	0.02	0.12	0.07	0.04
AE	dp	0.05	0.03	0.01	0.11	0.06	0.04
AC	jn	0.15	0.08	0.04	0.31	0.16	0.10
AB	jn	0.59	0.29	0.15	1.20	0.63	0.40
AA	dp	3.54	1.75	0.89	7.16	3.77	2.38
Y	jn	5.38	2.67	1.35	10.89	5.73	3.62
Z	dp	5.39	2.67	1.35	10.92	5.74	3.63
AG	dp	0.12	0.06	0.03	0.25	0.13	0.08
AF	jn	0.49	0.24	0.12	1.00	0.53	0.33
R	dp	0.46	0.23	0.12	0.94	0.49	0.31
V	dp	3.36	1.67	0.84	6.80	3.58	2.26
X	dp	0.22	0.11	0.05	0.44	0.23	0.15

Table B.13: Minimum daily flows of all junction nodes (jn) and discharge points (dp) in the system during all optimum windows in study period 2. $Q_{min,daily}^7$ flows were estimated as proportional to flow at station 02LB005, and $Q_{min,daily}^{30}$ flows were calculated with regression coefficients.

Node	Type	$Q_{min,daily}^7$ (m ³ /s)			$Q_{min,daily}^{30}$ (m ³ /s)		
		Min.	Aver.	Max.	Min.	Aver.	Max.
A	jn	1.06	0.41	0.13	2.46	2.13	1.01
B	dp	1.03	0.40	0.13	2.41	2.09	0.99
C	jn	0.99	0.38	0.12	2.30	2.00	0.94
D	jn	0.96	0.37	0.12	2.24	1.94	0.92
E	jn	0.84	0.33	0.11	1.96	1.70	0.80
F	jn	0.71	0.28	0.09	1.66	1.44	0.68
G	dp	0.65	0.25	0.08	1.52	1.32	0.62
H	jn	0.65	0.25	0.08	1.51	1.31	0.62
I	jn	0.44	0.17	0.05	1.02	0.88	0.42
J'	dp	0.39	0.15	0.05	0.91	0.79	0.37
J	dp	0.39	0.15	0.05	0.91	0.79	0.37
K	jn	0.39	0.15	0.05	0.90	0.78	0.37
L	dp	0.34	0.13	0.04	0.78	0.68	0.32
M'	dp	0.29	0.11	0.04	0.67	0.58	0.27
M	dp	0.29	0.11	0.04	0.67	0.58	0.27
N	jn	0.26	0.10	0.03	0.61	0.53	0.25
O	jn	0.22	0.09	0.03	0.52	0.45	0.21
P	jn	0.14	0.05	0.02	0.32	0.28	0.13
Q	dp	0.07	0.03	0.01	0.15	0.13	0.06
U	dp	0.00	0.00	0.00	0.00	0.00	0.00
T	jn	0.01	0.00	0.00	0.03	0.02	0.01
S'	dp	0.05	0.02	0.01	0.12	0.11	0.05
S	jn	0.05	0.02	0.01	0.12	0.11	0.05
AD	dp	0.00	0.00	0.00	0.01	0.00	0.00
AE	dp	0.00	0.00	0.00	0.00	0.00	0.00
AC	jn	0.01	0.00	0.00	0.01	0.01	0.01
AB	jn	0.02	0.01	0.00	0.05	0.04	0.02
AA	dp	0.12	0.05	0.02	0.29	0.25	0.12
Y	jn	0.19	0.07	0.02	0.44	0.38	0.18
Z	dp	0.19	0.07	0.02	0.44	0.38	0.18
AG	dp	0.00	0.00	0.00	0.01	0.01	0.00
AF	jn	0.02	0.01	0.00	0.04	0.04	0.02
R	dp	0.02	0.01	0.00	0.04	0.03	0.02
V	dp	0.12	0.05	0.01	0.27	0.24	0.11
X	dp	0.01	0.00	0.00	0.02	0.02	0.01

Table B.14: Statistics of water quality in all sampling stations of the river system for the years of 1980-1991 and for the months of March to May.

Station	Constituent	Min.	Aver.	Max.	25p	50p	75p
020	BOD ₅	0.80	4.03	14.30	1.58	2.50	5.30
020	NH ₃ -N	0.00	0.14	0.45	0.03	0.10	0.23
020	DO	2.40	11.64	15.00	10.50	12.00	13.50
020	Temp	1.00	7.65	19.50	2.00	7.00	13.00
020	pH	7.29	7.89	8.51	7.66	7.84	8.10
040	BOD ₅	0.40	2.05	7.10	1.10	1.58	2.09
040	NH ₃ -N	0.01	0.12	0.42	0.02	0.07	0.17
040	DO	6.00	9.27	13.00	7.75	9.50	10.25
040	Temp	1.00	10.00	20.00	3.00	10.50	15.75
040	pH	7.55	8.13	8.47	7.97	8.20	8.34
060	BOD ₅	0.60	1.64	2.50	0.93	1.75	2.40
060	NH ₃ -N	0.01	0.06	0.13	0.02	0.07	0.09
060	DO	7.00	10.23	14.00	9.00	10.00	11.25
060	Temp	1.00	9.38	17.00	3.25	9.50	15.00
060	pH	7.48	8.10	8.51	7.99	8.14	8.27
100	BOD ₅	0.60	1.11	1.70	0.63	1.05	1.65
100	NH ₃ -N	0.01	0.11	0.28	0.07	0.10	0.12
100	DO	6.00	10.24	13.00	9.00	10.00	11.00
100	Temp	1.00	9.48	20.00	2.50	9.00	15.50
100	pH	7.56	8.13	8.66	7.95	8.15	8.39
110	BOD ₅	0.50	1.64	3.30	0.90	1.40	2.30
110	NH ₃ -N	0.00	0.05	0.14	0.02	0.04	0.06
110	DO	1.20	7.79	12.00	4.00	8.90	11.40
110	Temp	0.30	9.81	20.00	3.50	8.00	17.00
110	pH	7.44	8.11	8.37	7.99	8.18	8.33
140	BOD ₅	0.20	1.02	1.50	0.45	1.20	1.43
140	NH ₃ -N	0.04	0.06	0.09	0.04	0.06	0.09
140	DO	4.00	10.29	15.00	9.00	10.00	11.50
140	Temp	1.00	9.54	21.00	3.00	10.00	15.00
140	pH	7.62	8.11	8.44	7.92	8.13	8.30
145	BOD ₅	0.22	1.19	2.80	0.72	1.07	1.50
145	NH ₃ -N	0.00	0.05	0.29	0.02	0.03	0.06
145	DO	5.00	10.61	15.00	9.00	11.00	13.00
145	Temp	1.00	10.34	21.00	2.75	12.00	17.00
145	pH	7.66	8.19	8.52	8.03	8.15	8.42

Table B.15: Statistics of water quality in all sampling stations of the river system for the years of 1980-1991 and for the months of October to December.

Station	Constituent	Min.	Aver.	Max.	25p	50p	75p
020	BOD ₅	0.80	2.79	11.30	1.00	2.40	2.60
020	NH ₃ -N	0.00	0.08	0.44	0.02	0.04	0.12
020	DO	2.40	12.14	19.00	11.00	12.00	13.75
020	Temp	0.80	6.28	16.00	3.00	5.00	9.00
020	pH	7.67	8.08	8.40	7.95	8.11	8.25
040	BOD ₅	0.57	2.06	5.60	1.02	1.62	3.20
040	NH ₃ -N	0.00	0.09	0.38	0.03	0.04	0.13
040	DO	3.00	1.05	14.00	9.75	12.00	13.00
040	Temp	1.00	5.15	15.00	2.00	5.00	6.00
040	pH	7.70	8.15	8.61	8.01	8.11	8.23
060	BOD ₅	0.70	1.85	2.70	0.93	2.15	2.47
060	NH ₃ -N	0.01	0.03	0.04	0.02	0.03	0.04
060	DO	4.00	11.37	18.00	10.00	12.00	13.00
060	Temp	1.00	5.05	13.00	2.00	5.00	7.00
060	pH	7.60	8.12	8.61	8.00	8.14	8.28
100	BOD ₅	0.40	1.43	2.80	0.95	1.30	2.10
100	NH ₃ -N	0.01	0.06	0.14	0.03	0.07	0.09
100	DO	8.00	11.68	14.00	11.00	12.00	12.00
100	Temp	1.00	6.83	20.00	2.00	6.00	10.00
100	pH	7.60	8.19	8.55	8.06	8.21	8.34
110	BOD ₅	0.90	1.64	3.00	1.05	1.30	2.35
110	NH ₃ -N	0.00	0.04	0.13	0.02	0.03	0.06
110	DO	0.80	11.01	31.00	1.15	11.0	17.00
110	Temp	0.10	5.72	15.00	0.80	2.50	11.00
110	pH	7.40	8.13	8.46	8.01	8.19	8.35
140	BOD ₅	0.30	1.49	5.10	0.67	1.05	1.85
140	NH ₃ -N	0.00	0.03	0.05	0.02	0.02	0.03
140	DO	4.00	11.27	15.00	10.00	12.00	13.00
140	Temp	1.00	6.07	17.00	2.00	6.00	8.00
140	pH	7.50	8.18	8.59	8.05	8.26	8.36
145	BOD ₅	0.40	1.31	3.60	0.65	1.05	1.63
145	NH ₃ -N	0.00	0.02	0.04	0.01	0.02	0.03
145	DO	4.00	11.81	16.00	10.75	13.00	14.0
145	Temp	1.00	5.94	16.00	2.00	5.50	8.00
145	pH	7.60	8.28	8.66	8.13	8.31	8.45

Table B.16: Available dilution at the point-sources locations for $Q_{min,daily}^7$ and $Q_{min,daily}^{30}$ flows of all optimum windows in study period 1 for the discharge scheme of year 1991.

Plant	Available Dilution, D (-)					
	$Q_{min,daily}^7$			$Q_{min,daily}^{30}$		
	Min.	Aver.	Max.	Min.	Aver.	Max.
Casselman	74.60	73.99	58.26	160.00	144.88	118.18
Chesterville	40.56	40.23	31.68	87.00	78.78	64.26
Embrun	16.69	16.56	13.04	35.81	32.42	26.45
Maxville	0.38	0.38	0.30	0.81	0.73	0.60
Plantagenet	191.96	190.38	149.91	411.71	372.82	304.11
Russell	26.90	26.68	21.01	57.70	52.25	42.62
Spencerville	53.82	53.37	42.03	115.42	104.52	85.26
St.Isidore	17.37	17.23	13.57	37.26	33.74	27.52
Winchester	0.14	0.14	0.11	0.31	0.28	0.23
Williamsburg	2.82	2.80	2.20	6.05	5.47	4.47
Ault	0.41	0.41	0.32	0.88	0.79	0.65
Nestles	53.09	52.66	41.46	113.88	103.12	84.11

Table B.17: Available dilution at the point-sources locations for $Q_{min,daily}^7$ and $Q_{min,daily}^{30}$ flows of all optimum windows in study period 2 and for the discharge scheme of year 1991.

Plant	Available Dilution, D (-)					
	$Q_{min,daily}^7$			$Q_{min,daily}^{30}$		
	Min.	Aver.	Max.	Min.	Aver.	Max.
Casselman	0.46	1.98	0.97	5.74	9.66	6.48
Chesterville	0.25	1.08	0.53	3.12	5.25	3.52
Embrun	0.10	0.44	0.22	1.28	2.16	1.45
Maxville	0.00	0.01	0.00	0.03	0.05	0.03
Plantagenet	1.19	5.09	2.50	14.77	24.85	16.67
Russell	0.17	0.71	0.35	2.07	3.48	2.34
Spencerville	0.33	1.43	0.70	4.14	6.97	4.67
St. Isidore	0.11	0.46	0.23	1.34	2.25	1.51
Winchester	0.00	0.00	0.00	0.01	0.02	0.01
Williamsburg	0.02	0.07	0.04	0.22	0.36	0.24
Ault	0.00	0.01	0.01	0.03	0.05	0.04
Nestles	0.33	1.41	0.69	4.09	6.87	4.61

Table B.18: Available dilution at the point-sources locations for $Q_{min,daily}^7$ and $Q_{min,daily}^{30}$ flows of all optimum windows in study period 1 and for the projected discharge scheme of year 2011.

Plant	Available Dilution, D (-)					
	$Q_{min,daily}^7$			$Q_{min,daily}^{30}$		
	Min.	Aver.	Max.	Min.	Aver.	Max.
Bourget	21.42	21.25	16.73	45.95	41.61	33.94
Casselman	28.84	28.60	22.52	61.85	56.00	45.68
Chesterville	37.65	37.37	29.43	80.82	73.19	59.70
Crysler	64.18	63.65	50.12	137.65	124.65	101.68
Embrun	5.11	5.07	3.99	10.96	9.921	8.09
Fournier	13.38	13.27	10.45	28.70	25.99	21.20
Maxville	0.37	0.37	0.29	0.79	0.72	0.58
Moose Creek	3.12	3.10	2.44	6.70	6.07	4.95
Plantagenet	170.17	168.77	132.90	364.98	330.51	269.59
Russell	6.00	5.96	4.69	12.88	11.66	9.51
Spencerville	39.65	39.33	30.97	85.05	77.02	62.82
St. Albert	117.80	116.83	92.00	252.60	228.80	186.63
St. Isidore	12.49	12.39	9.76	26.79	24.26	19.7
Williamsburg	2.56	2.54	2.00	5.48	4.97	4.05
Winchester	0.13	0.13	0.10	0.27	0.25	0.20
Ault	0.17	0.17	0.14	0.37	0.34	0.28
Nestles	53.09	52.66	41.46	113.88	103.12	84.11
St. Albert's Cheese	719.61	713.70	562.00	1543.42	1397.64	1140.05

Table B.19: Available dilution at the point-sources locations for $Q_{min,daily}^7$ and $Q_{min,daily}^{30}$ flows of all optimum windows in study period 2 and for the projected discharge scheme of year 2011.

Plant	Available Dilution, D (-)					
	$Q_{min,daily}^7$			$Q_{min,daily}^{30}$		
	Min.	Aver.	Max.	Min.	Aver.	Max.
Bourget	0.13	0.57	0.28	1.65	2.77	1.86
Casselman	0.18	0.76	0.38	2.22	3.73	2.50
Chesterville	0.23	1.00	0.49	2.90	4.88	3.27
Crysler	0.40	1.70	0.84	4.94	8.31	5.57
Embrun	0.03	0.14	0.07	0.39	0.66	0.44
Fournier	0.08	0.35	0.17	1.03	1.73	1.16
Maxville	0.00	0.01	0.00	0.03	0.05	0.03
Moose Creek	0.02	0.08	0.04	0.24	0.40	0.27
Plantagenet	1.06	4.51	2.21	13.10	22.03	14.78
Russell	0.04	0.16	0.08	0.46	0.78	0.52
Spencerville	0.25	1.05	0.52	3.05	5.13	3.44
St. Albert	0.73	3.12	1.53	9.07	15.25	10.23
St. Isidore	0.08	0.33	0.16	0.96	1.62	1.09
Williamsburg	0.02	0.07	0.03	0.20	0.33	0.22
Winchester	0.00	0.00	0.00	0.01	0.02	0.01
Ault	0.00	0.00	0.00	0.01	0.02	0.02
Nestles	0.33	1.41	0.69	4.09	6.87	4.61
St. Albert's Cheese	4.46	19.08	9.36	55.38	93.15	62.51

Table B.20: Required river dilution for typical lagoon effluent concentrations of $(NH_3)_{tot}$ and $(H_2S)_{tot}$ in order to meet the river PWQO standards of $(NH_3)_u$ (0.02 mg/l) and of $(H_2S)_u$ (0.002 mg/l).

Month	T (°C)	pH	Required Dilution, D				
			$[(NH_3)_{tot}]_w$ (mg/l)			$[(H_2S)_{tot}]_w$ (mg/l)	
			14.0	6.0	4.0	3.0	1.0
March	1.00	6.90	0.00	0.00	0.00	1101.99	366.66
	3.00	6.90	0.00	0.00	0.00	1081.46	359.82
	1.00	7.50	1.04	0.00	0.00	615.53	204.51
April	3.00	7.50	1.42	0.03	0.00	590.57	196.19
	6.00	7.10	0.21	0.00	0.00	893.49	297.16
	10.00	7.10	0.64	0.00	0.00	845.09	281.03
	6.00	7.90	6.98	2.38	1.23	283.57	93.86
May	10.00	7.90	9.98	3.66	2.07	254.26	84.09
	13.00	7.60	5.66	1.82	0.86	405.19	134.40
	17.00	7.60	7.98	2.81	1.51	368.92	122.31
	13.00	8.10	22.35	8.90	5.53	156.64	51.55
October	17.00	8.10	31.76	12.89	8.17	139.71	45.90
	10.00	7.60	4.31	1.25	0.48	434.54	144.18
	13.00	7.60	5.66	1.82	0.86	405.19	134.40
	10.00	7.90	9.98	3.66	2.07	254.26	84.09
November	13.00	7.90	12.91	4.90	2.89	234.37	77.46
	4.00	7.60	2.32	0.41	0.00	498.91	165.64
	7.00	7.60	3.21	0.79	0.18	465.78	154.59
	4.00	8.00	7.64	2.66	1.42	247.96	81.99
December	7.00	8.00	10.06	3.69	2.10	227.65	75.22
	1.00	7.40	0.62	0.00	0.00	700.51	232.84
	3.00	7.40	0.92	0.00	0.00	674.74	224.25
	1.00	7.90	4.23	1.22	0.46	325.12	107.71
	3.00	7.90	5.23	1.64	0.74	307.81	101.94

Table B.21: Maximum potential discharge volume of point sources on study period 2 in 1991 that meets the PWQO river quality standards for $(NH_3)_u$ (0.02 mg/l) based on $Q_{min,daily}^7$ flows.

Plant	V_{tot} (m^3)	V_{min} (m^3)	$\frac{V_{min}}{V_{tot}}$ (-)	V_{aver} (m^3)	$\frac{V_{aver}}{V_{tot}}$ (-)	V_{max} (m^3)	$\frac{V_{max}}{V_{tot}}$ (-)
Casselman	365000	44999	0.12	192454	0.53	94433	0.26
Chesterville	296015	19844	0.07	8486	0.29	41643	0.14
Embrun	474500	13092	0.03	55992	0.12	27474	0.06
Maxville	59860	37	0.00	160	0.00	78	0.00
Plantagenet	225205	71444	0.32	305559	1.36	149930	0.67
Russell	193085	8585	0.04	36715	0.19	18015	0.09
Spencerville	51100	4545	0.09	19437	0.38	9537	0.19
St.Isidore	127020	3647	0.03	15598	0.12	7654	0.06
Williamsburg	64240	299	0.00	1280	0.02	628	0.01
Winchester	629625	150	0.00	640	0.00	314	0.00
Ault	193815	78	0.00	333	0.00	163	0.00
Nestles	225935	11793	0.05	50438	0.22	24749	0.11

Table B.22: Maximum potential discharge volume of point sources on study period 2 in 1991 that meets the PWQO river quality standards for $(NH_3)_u$ (0.02 mg/l) based on $Q_{min,daily}^{30}$ flows.

Plant	V_{tot} (m^3)	V_{min} (m^3)	$\frac{V_{min}}{V_{tot}}$ (-)	V_{aver} (m^3)	$\frac{V_{aver}}{V_{tot}}$ (-)	V_{max} (m^3)	$\frac{V_{max}}{V_{tot}}$ (-)
Casselman	365000	558491	1.53	939362	2.57	630337	1.73
Chesterville	296015	246284	0.83	414241	1.40	277267	0.94
Embrun	474500	162487	0.34	273297	0.58	183390	0.39
Maxville	59860	464	0.01	781	0.01	524	0.01
Plantagenet	225205	886715	3.94	1491423	6.62	1000784	4.44
Russell	193085	106545	0.55	179205	0.93	120251	0.62
Spencerville	51100	56406	1.10	94873	1.86	63662	1.25
St.Isidore	127020	45264	0.36	76133	0.60	51087	0.40
Williamsburg	64240	3714	0.06	6247	0.10	4192	0.07
Winchester	629625	1857	0.00	3123	0.00	2096	0.00
Ault	193815	967	0.00	1626	0.01	1091	0.01
Nestles	225935	146368	0.65	246185	1.09	165197	0.73

Table B.23: Maximum potential discharge volume of point sources on study period 2 in 2011 that meets the PWQO river quality standards for $(NH_3)_u$ (0.02 mg/l) based on $Q_{min,daily}^7$ flows.

Plant	V_{tot} (m ³)	V_{min} (m ³)	$\frac{V_{min}}{V_{tot}}$ (-)	V_{aver} (m ³)	$\frac{V_{aver}}{V_{tot}}$ (-)	V_{max} (m ³)	$\frac{V_{max}}{V_{tot}}$ (-)
Bourget	230315	8154	0.04	34875	0.15	17112	0.07
Casselman	944255	44999	0.05	192454	0.20	94433	0.10
Chesterville	318645	19844	0.06	84869	0.27	41643	0.13
Crysler	219000	23229	0.11	99347	0.45	48747	0.22
Embrun	1550885	13092	0.01	55992	0.04	27474	0.02
Fournier	50735	1122	0.02	4799	0.09	2355	0.05
Maxville	61320	37	0.00	160	0.00	78	0.00
Moose Creek	101470	524	0.01	2240	0.02	1099	0.01
Plantagenet	254040	71444	0.27	305559	1.20	149930	0.59
Russell	865050	8585	0.01	36715	0.04	18015	0.02
Spencerville	69350	4545	0.07	19437	0.28	9537	0.14
St. Albert	138335	26932	0.19	115184	0.83	56158	0.41
St. Isidore	176660	3647	0.02	15598	0.09	7654	0.04
Williamsburg	70810	299	0.00	1280	0.02	628	0.01
Winchester	717225	150	0.00	640	0.00	314	0.00
Ault	454790	78	0.00	333	0.00	163	0.00
Nestles	225935	11793	0.05	50438	0.22	24749	0.11
St. Albert Cheese	22630	16010	0.71	68472	3.03	33597	1.48

Table B.24: Maximum potential discharge volume of point sources on study period 2 in 2011 that meets the PWQO river quality standards for $(NH_3)_u$ (0.02 mg/l) based on $Q_{min,daily}^{30}$ flows.

Plant	V_{tot} (m^3)	V_{min} (m^3)	$\frac{V_{min}}{V_{tot}}$ (-)	V_{aver} (m^3)	$\frac{V_{aver}}{V_{tot}}$ (-)	V_{max} (m^3)	$\frac{V_{max}}{V_{tot}}$ (-)
Bourget	230315	101206	0.44	170225	0.74	114226	0.50
Casselman	944255	558491	0.59	939362	0.99	630337	0.67
Chesterville	318645	246284	0.77	414241	1.30	277967	0.87
Crysler	219000	288299	1.32	484908	2.21	325386	1.49
Embrun	1550885	162487	0.10	273297	0.18	183390	0.12
Fournier	50735	13927	0.27	23425	0.46	15719	0.31
Maxville	61320	464	0.01	781	0.01	524	0.01
Moose Creek	101470	6499	0.06	10932	0.11	7336	0.07
Plantagenet	254040	886715	3.49	1491423	5.87	1000784	3.94
Russell	865050	106545	0.12	179205	0.21	120251	0.14
Spencerville	69350	56406	0.81	94873	1.37	63662	0.92
St. Albert	138335	334259	2.42	562212	4.06	377259	2.73
St. Isidore	176660	45264	0.26	76133	0.43	51087	0.29
Williamsburg	70810	3714	0.05	6247	0.09	4192	0.06
Winchester	717225	1857	0.00	3123	0.00	2096	0.00
Ault	454790	967	0.00	1626	0.00	1091	0.00
Nestles	225935	146368	0.65	246185	1.09	165197	0.73
St. Albert Cheese	22630	198701	8.78	334208	14.77	224263	9.91

Table B.25: Predicted variation of DO , BOD_u (set 1) and of $(NH_3)_{tot}$ (set 2) during the average discharge window in study period 1 for $Q_{min.daily}^7$ river flows and discharge of plants in 1991.

Nodes		%DO _s (%DO _s > 47) (mg/l) (Y / N)		L (L < L _{lim}) (mg/l) (Y / N)		N (N < N _{lim}) (mg/l) (Y / N)	
Start	End	Start	End	Start	End	Start	End
Q	P	73.78 (Y)	91.51 (Y)	4.180 (Y)	0.853 (Y)	0.370 (Y)	0.298 (Y)
P	O	82.81 (Y)	85.20 (Y)	2.176 (Y)	1.799 (Y)	0.204 (Y)	0.199 (Y)
O	N	81.11 (Y)	85.67 (Y)	2.426 (Y)	1.723 (Y)	0.167 (Y)	0.159 (Y)
N	M	84.37 (Y)	87.83 (Y)	1.891 (Y)	1.418 (Y)	0.211 (Y)	0.202 (Y)
M	MI	85.82 (Y)	85.83 (Y)	1.952 (Y)	1.946 (Y)	0.305 (Y)	0.304 (Y)
MI	K	84.61 (Y)	84.98 (Y)	2.937 (Y)	1.554 (Y)	0.627 (Y)	0.575 (Y)
K	I	82.38 (Y)	84.43 (Y)	2.023 (Y)	1.681 (Y)	0.460 (Y)	0.449 (Y)
I	H	83.36 (Y)	83.93 (Y)	1.871 (Y)	1.783 (Y)	0.412 (Y)	0.410 (Y)
H	G	77.58 (Y)	78.31 (Y)	2.418 (Y)	2.291 (Y)	0.850 (Y)	0.844 (Y)
G	F	77.75 (Y)	81.21 (Y)	2.808 (Y)	1.873 (Y)	1.005 (Y)	0.950 (Y)
F	E	80.69 (Y)	81.45 (Y)	1.994 (Y)	1.872 (Y)	0.885 (Y)	0.878 (Y)
E	D	80.46 (Y)	83.38 (Y)	2.100 (Y)	1.686 (Y)	0.767 (Y)	0.744 (Y)
D	C	82.81 (Y)	84.66 (Y)	1.787 (Y)	1.552 (Y)	0.714 (Y)	0.700 (Y)
C	B	84.41 (Y)	87.36 (Y)	1.599 (Y)	1.254 (Y)	0.685 (Y)	0.663 (Y)
B	A	86.57 (Y)	88.19 (Y)	1.555 (Y)	1.265 (Y)	0.706 (Y)	0.686 (Y)
AG	AF	63.91 (Y)	58.14 (Y)	14.097 (N)	3.118 (Y)	3.773 (N)	3.065 (N)
AF	N	69.42 (Y)	82.22 (Y)	3.337 (Y)	1.792 (Y)	1.034 (Y)	0.949 (Y)
AD	AC	39.26 (N)	-69.46 (N)	38.850 (N)	31.566 (N)	12.267 (N)	11.921 (N)
AE	AC	45.85 (N)	25.10 (N)	6.200 (Y)	4.306 (Y)	4.374 (N)	4.159 (N)
AC	AB	-37.17 (N)	-54.10 (N)	22.974 (N)	20.922 (N)	9.271 (N)	9.152 (N)
AB	Y	-5.35 (N)	43.23 (N)	14.293 (N)	3.949 (Y)	5.726 (N)	4.796 (N)
AA	Y	73.14 (Y)	71.99 (Y)	4.898 (Y)	4.231 (Y)	0.617 (Y)	0.604 (Y)
Y	Z	67.03 (Y)	67.57 (Y)	4.014 (Y)	3.857 (Y)	1.303 (Y)	1.296 (Y)
Z	H	65.89 (Y)	65.16 (Y)	5.889 (Y)	3.585 (Y)	1.938 (N)	1.810 (N)
U	T	45.22 (N)	-70.75 (N)	32.864 (N)	14.422 (N)	10.213 (N)	9.117 (N)
T	S	52.93 (Y)	86.97 (Y)	5.069 (Y)	1.164 (Y)	1.453 (Y)	1.186 (Y)
S	S"	77.60 (Y)	77.70 (Y)	2.874 (Y)	2.856 (Y)	0.380 (Y)	0.380 (Y)
S"	D	75.36 (Y)	83.48 (Y)	5.053 (Y)	1.473 (N)	1.107 (Y)	0.934 (Y)

Table B26: Predicted variation of DO , BOD_u (set 1) and of $(NH_3)_{tot}$ (set 2) during the average discharge window in study period 1 for $Q_{min,daily}^7$ river flows and discharge of plants in 2011.

Nodes		%DO _s (%DO _s > 47) (mg/l) (Y / N)		L (L < L _{lim}) (mg/l) (Y / N)		N (N < N _{lim}) (mg/l) (Y / N)	
Start	End	Start	End	Start	End	Start	End
Q	P	73.52 (Y)	90.68 (Y)	4.439 (Y)	0.910 (Y)	0.459 (Y)	0.369 (Y)
P	O	82.44 (Y)	84.88 (Y)	2.199 (Y)	1.819 (Y)	0.239 (Y)	0.233 (Y)
O	N	80.92 (Y)	85.54 (Y)	2.437 (Y)	1.732 (Y)	0.188 (Y)	0.179 (Y)
N	M	84.21 (Y)	87.71 (Y)	1.901 (Y)	1.425 (Y)	0.234 (Y)	0.225 (Y)
M	M1	85.71 (Y)	85.73 (Y)	1.958 (Y)	1.952 (Y)	0.325 (Y)	0.324 (Y)
M1	L	84.42 (Y)	83.68 (Y)	3.015 (Y)	1.777 (Y)	0.670 (Y)	0.623 (Y)
L	K	81.69 (Y)	81.54 (Y)	2.625 (Y)	2.375 (Y)	0.751 (Y)	0.741 (Y)
K	J	80.65 (Y)	80.84 (Y)	2.508 (Y)	2.385 (Y)	0.662 (Y)	0.657 (Y)
J	J1	80.75 (Y)	80.76 (Y)	2.416 (Y)	2.409 (Y)	0.662 (Y)	0.662 (Y)
J1	I	80.39 (Y)	80.45 (Y)	2.743 (Y)	2.408 (Y)	0.769 (Y)	0.755 (Y)
I	H	79.85 (Y)	80.11 (Y)	2.512 (Y)	2.395 (Y)	0.690 (Y)	0.686 (Y)
H	G	62.47 (Y)	62.94 (Y)	4.147 (Y)	3.936 (Y)	1.750 (N)	1.738 (N)
G	F	62.14 (Y)	63.76 (Y)	5.093 (Y)	3.451 (Y)	2.090 (N)	1.980 (N)
X	F	64.69 (Y)	84.25 (Y)	13.312 (N)	1.177 (Y)	3.504 (N)	2.508 (N)
F	E	64.67 (Y)	65.71 (Y)	3.423 (Y)	3.222 (Y)	1.872 (N)	1.856 (N)
V	E	72.71 (Y)	74.49 (Y)	5.255 (Y)	2.783 (Y)	0.739 (Y)	0.677 (Y)
E	D	66.92 (Y)	71.01 (Y)	3.170 (Y)	2.564 (Y)	1.685 (N)	1.636 (N)
D	C	71.69 (Y)	74.42 (Y)	2.572 (Y)	2.243 (Y)	1.520 (N)	1.492 (N)
R	C	71.70 (Y)	68.58 (Y)	6.271 (Y)	3.336 (Y)	1.088 (Y)	0.997 (Y)
C	B	74.33 (Y)	78.85 (Y)	2.270 (Y)	1.794 (Y)	1.472 (Y)	1.425 (Y)
B	A	78.45 (Y)	81.28 (Y)	2.074 (Y)	1.697 (Y)	1.439 (Y)	1.400 (Y)
AG	AF	63.13 (Y)	54.92 (Y)	14.879 (N)	3.344 (Y)	4.042 (N)	3.290 (N)
AF	N	68.30 (Y)	81.57 (Y)	3.406 (Y)	1.834 (Y)	1.122 (Y)	1.031 (Y)
AD	AC	38.72 (N)	-68.53 (N)	39.398 (N)	32.311 (N)	12.455 (N)	12.119 (N)
AE	AC	40.17 (N)	15.46 (N)	6.748 (Y)	5.107 (Y)	5.218 (N)	5.022 (N)
AC	AB	-30.70 (N)	-45.23 (N)	20.761 (N)	19.099 (N)	8.926 (N)	8.824 (N)
AB	Y	-7.61 (N)	35.74 (N)	14.178 (N)	4.299 (Y)	6.088 (N)	5.164 (N)
AA	Y	68.73 (Y)	57.24 (Y)	9.253 (N)	8.048 (Y)	2.111 (N)	2.071 (N)
Y	Z	55.99 (Y)	55.33 (Y)	6.425 (Y)	6.185 (Y)	2.349 (N)	2.337 (N)
Z	H	52.50 (Y)	31.89 (N)	11.290 (N)	7.144 (Y)	3.913 (N)	3.673 (N)
U	T	45.03 (N)	-72.24 (N)	33.057 (N)	14.590 (N)	10.279 (N)	9.183 (N)
T	S	52.38 (Y)	86.77 (Y)	5.119 (Y)	1.177 (Y)	1.483 (N)	1.211 (Y)
S	S"	77.56 (Y)	77.66 (Y)	2.876 (Y)	2.858 (Y)	0.387 (Y)	0.387 (Y)
S"	D	74.48 (Y)	80.02 (Y)	5.848 (Y)	1.723 (Y)	1.376 (Y)	1.163 (Y)

Table B.27: Predicted variation of DO , BOD_u (set 1) and of $(NH_3)_{tot}$ (set 2) during the average discharge window in study period 1 for $Q_{min,daily}^{30}$ river flows and discharge of plants in 1991.

Nodes		%DO _s (%DO _s > 47) (mg/l) (Y / N)		L (L < L _{lim}) (mg/l) (Y / N)		N (N < N _{lim}) (mg/l) (Y / N)	
Start	End	Start	End	Start	End	Start	End
Q	P	72.34 (Y)	88.87 (Y)	3.819 (Y)	1.243 (Y)	0.247 (Y)	0.212 (Y)
P	O	80.56 (Y)	82.76 (Y)	2.371 (Y)	2.074 (Y)	0.162 (Y)	0.159 (Y)
O	N	78.90 (Y)	83.04 (Y)	2.597 (Y)	2.041 (Y)	0.142 (Y)	0.138 (Y)
N	M	81.84 (Y)	84.97 (Y)	2.182 (Y)	1.781 (Y)	0.167 (Y)	0.163 (Y)
M	M1	83.52 (Y)	83.54 (Y)	2.103 (Y)	2.098 (Y)	0.216 (Y)	0.215 (Y)
M1	K	82.91 (Y)	85.85 (Y)	2.616 (Y)	1.664 (Y)	0.386 (Y)	0.363 (Y)
K	I	82.52 (Y)	84.17 (Y)	2.114 (Y)	1.854 (Y)	0.300 (Y)	0.295 (Y)
I	H	82.91 (Y)	83.37 (Y)	2.028 (Y)	1.960 (Y)	0.275 (Y)	0.274 (Y)
H	G	79.24 (Y)	79.78 (Y)	2.431 (Y)	2.339 (Y)	0.521 (Y)	0.519 (Y)
G	F	79.46 (Y)	82.53 (Y)	2.616 (Y)	1.957 (Y)	0.606 (Y)	0.583 (Y)
F	E	81.74 (Y)	82.31 (Y)	2.077 (Y)	1.985 (Y)	0.546 (Y)	0.542 (Y)
E	D	80.86 (Y)	83.07 (Y)	2.203 (Y)	1.884 (Y)	0.478 (Y)	0.468 (Y)
D	C	82.41 (Y)	83.82 (Y)	1.978 (Y)	1.789 (Y)	0.451 (Y)	0.445 (Y)
C	B	83.53 (Y)	85.84 (Y)	1.832 (Y)	1.540 (Y)	0.436 (Y)	0.426 (Y)
B	A	85.14 (Y)	86.74 (Y)	1.732 (Y)	1.495 (Y)	0.448 (Y)	0.439 (Y)
AG	AF	66.64 (Y)	63.70 (Y)	9.686 (N)	3.155 (Y)	2.260 (Y)	1.946 (Y)
AF	N	70.16 (Y)	79.90 (Y)	3.356 (Y)	2.152 (Y)	0.633 (Y)	0.596 (Y)
AD	AC	41.98 (N)	-41.43 (N)	35.066 (N)	29.212 (N)	10.969 (N)	10.705 (N)
AE	AC	50.80 (Y)	38.83 (N)	5.601 (Y)	4.132 (Y)	3.451 (N)	3.314 (N)
AC	AB	-9.40 (N)	-22.30 (N)	20.065 (N)	18.524 (N)	7.749 (N)	7.667 (N)
AB	Y	25.01 (N)	48.65 (Y)	11.010 (N)	4.013 (Y)	3.906 (N)	3.415 (N)
AA	Y	71.97 (Y)	72.97 (Y)	4.196 (Y)	3.783 (Y)	0.376 (Y)	0.371 (Y)
Y	Z	69.17 (Y)	69.71 (Y)	3.743 (Y)	3.637 (Y)	0.783 (Y)	0.780 (Y)
Z	H	68.70 (Y)	71.14 (Y)	4.768 (Y)	3.325 (Y)	1.150 (Y)	1.096 (Y)
U	T	50.05 (Y)	-39.02 (N)	26.767 (N)	13.414 (N)	8.121 (N)	7.406 (N)
T	S	61.41 (Y)	84.47 (Y)	4.444 (Y)	1.543 (Y)	0.852 (Y)	0.740 (Y)
S	S"	75.50 (Y)	75.60 (Y)	2.986 (Y)	2.973 (Y)	0.264 (Y)	0.263 (Y)
S"	D	74.39 (Y)	83.60 (Y)	4.138 (Y)	1.715 (Y)	0.653 (Y)	0.580 (Y)

Table B28: Predicted variation of DO , BOD_u (set 1) and of $(NH_3)_{tot}$ (set 2) during the average discharge window in study period 1 for $Q_{min.daily}^{30}$ river flows and discharge of plants in 2011.

Nodes		%DO _s (%DO _s > 47) (mg/l) (Y / N)		L (L < L _{lim}) (mg/l) (Y / N)		N (N < N _{lim}) (mg/l) (Y / N)	
Start	End	Start	End	Start	End	Start	End
Q	P	72.21 (Y)	88.27 (Y)	3.954 (Y)	1.289 (Y)	0.293 (Y)	0.252 (Y)
P	O	80.28 (Y)	82.51 (Y)	2.392 (Y)	2.092 (Y)	0.182 (Y)	0.179 (Y)
O	N	78.75 (Y)	82.91 (Y)	2.608 (Y)	2.050 (Y)	0.154 (Y)	0.149 (Y)
N	M	81.69 (Y)	84.85 (Y)	2.192 (Y)	1.789 (Y)	0.180 (Y)	0.175 (Y)
M	MI	83.41 (Y)	83.44 (Y)	2.110 (Y)	2.105 (Y)	0.227 (Y)	0.227 (Y)
MI	L	82.76 (Y)	84.91 (Y)	2.661 (Y)	1.827 (Y)	0.410 (Y)	0.390 (Y)
L	K	82.79 (Y)	83.16 (Y)	2.381 (Y)	2.217 (Y)	0.457 (Y)	0.452 (Y)
K	J	81.81 (Y)	82.11 (Y)	2.374 (Y)	2.291 (Y)	0.409 (Y)	0.407 (Y)
J	J1	82.01 (Y)	82.05 (Y)	2.310 (Y)	2.305 (Y)	0.409 (Y)	0.409 (Y)
J1	I	81.84 (Y)	82.42 (Y)	2.482 (Y)	2.262 (Y)	0.467 (Y)	0.461 (Y)
I	H	81.41 (Y)	81.72 (Y)	2.384 (Y)	2.304 (Y)	0.425 (Y)	0.423 (Y)
H	G	71.59 (Y)	71.89 (Y)	3.508 (Y)	3.378 (Y)	1.041 (Y)	1.036 (Y)
G	F	71.27 (Y)	72.57 (Y)	4.032 (Y)	3.036 (Y)	1.243 (Y)	1.197 (Y)
X	F	67.15 (Y)	81.39 (Y)	9.163 (N)	1.522 (Y)	2.080 (Y)	1.638 (Y)
F	E	72.67 (Y)	73.21 (Y)	3.046 (Y)	2.915 (Y)	1.130 (Y)	1.123 (Y)
V	E	71.79 (Y)	76.83 (Y)	4.385 (Y)	2.793 (Y)	0.441 (Y)	0.415 (Y)
E	D	73.68 (Y)	75.92 (Y)	2.906 (Y)	2.492 (Y)	1.015 (Y)	0.995 (Y)
D	C	76.00 (Y)	77.59 (Y)	2.521 (Y)	2.284 (Y)	0.923 (Y)	0.911 (Y)
R	C	71.25 (Y)	73.91 (Y)	4.935 (Y)	3.146 (Y)	0.629 (Y)	0.593 (Y)
C	B	77.48 (Y)	80.22 (Y)	2.308 (Y)	1.947 (Y)	0.899 (Y)	0.878 (Y)
B	A	79.78 (Y)	81.79 (Y)	2.126 (Y)	1.840 (Y)	0.884 (Y)	0.867 (Y)
AG	AF	66.12 (Y)	61.20 (Y)	10.218 (N)	3.351 (Y)	2.442 (Y)	2.105 (Y)
AF	N	69.42 (Y)	79.36 (Y)	3.412 (Y)	2.189 (Y)	0.684 (Y)	0.645 (Y)
AD	AC	-41.14 (N)	-42.28 (N)	35.933 (N)	30.156 (N)	4.587 (N)	4.441 (N)
AE	AC	43.34 (N)	26.19 (N)	6.338 (Y)	4.970 (Y)	11.266 (N)	11.006 (N)
AC	AB	-8.578 (N)	-20.14 (N)	18.822 (N)	17.505 (N)	7.784 (N)	7.709 (N)
AB	Y	20.47 (N)	42.02 (N)	11.352 (N)	4.354 (Y)	4.388 (N)	3.861 (N)
AA	Y	69.61 (Y)	66.29 (Y)	6.631 (Y)	5.994 (Y)	1.212 (Y)	1.195 (Y)
Y	Z	63.60 (Y)	63.57 (Y)	5.200 (Y)	5.056 (Y)	1.413 (Y)	1.408 (Y)
Z	H	61.13 (Y)	51.97 (Y)	8.227 (Y)	5.832 (Y)	2.434 (Y)	2.325 (Y)
U	T	49.82 (Y)	-40.46 (N)	27.004 (N)	13.58 (N)	8.202 (N)	7.485 (N)
T	S	61.13 (Y)	84.30 (Y)	4.475 (Y)	1.555 (Y)	0.869 (Y)	0.755 (Y)
S	S"	75.47 (Y)	75.56 (Y)	2.989 (Y)	2.975 (Y)	0.267 (Y)	0.267 (Y)
S"	D	73.90 (Y)	81.38 (Y)	4.576 (Y)	1.904 (Y)	0.803 (Y)	0.714 (Y)