Models for Water Quality Management

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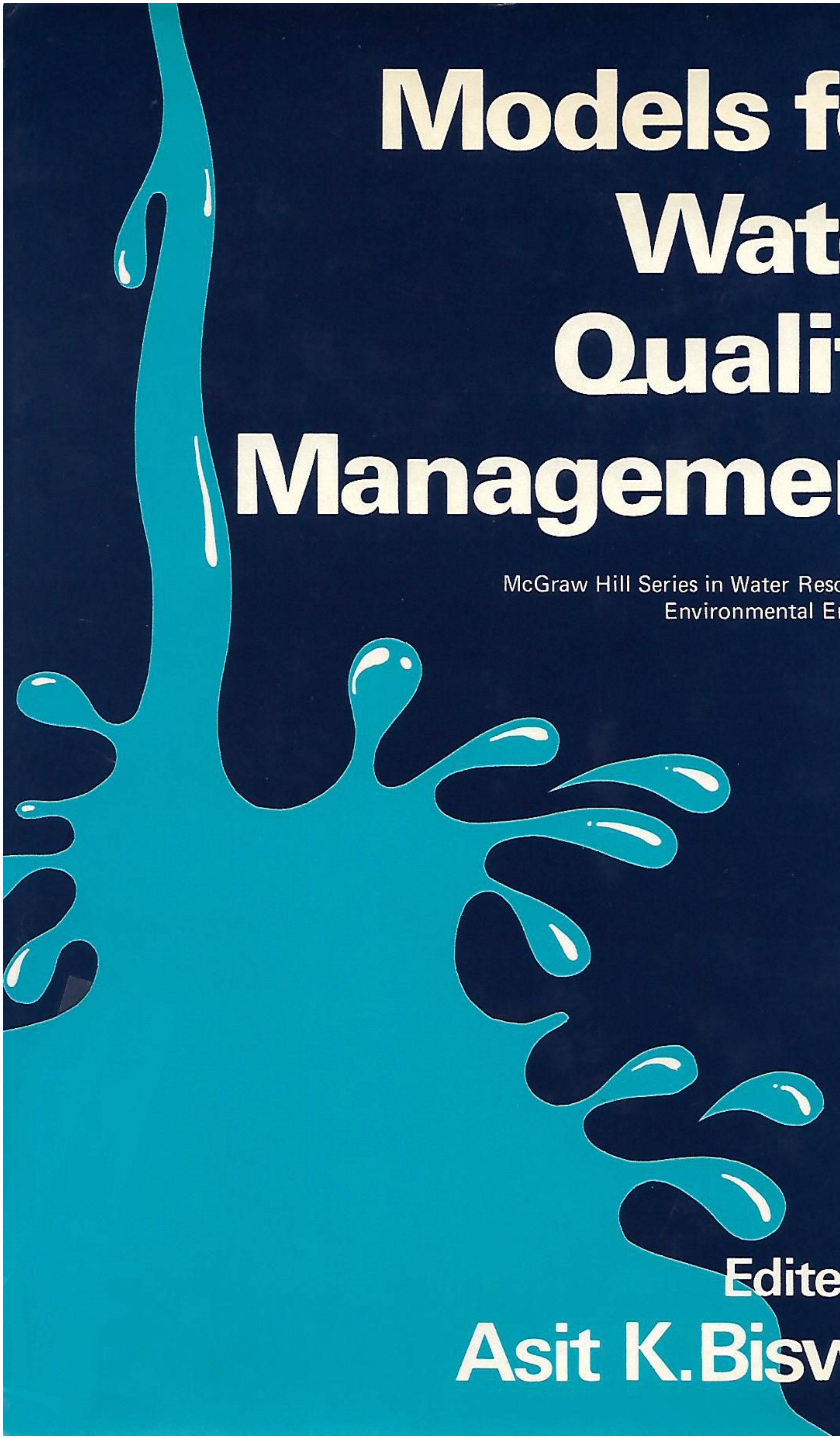

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FOREWORD

Water is essential for human survival; the terrestrial ecosystem cannot function without it. In addition to drinking and personal hygiene, water is needed for agricultural production, industrial and manufacturing processes, hydroelectric power generation, waste assimilation, recreation, navigation, enhancement of fish and wildlife, and a variety of other purposes. When a resource is used for so many diverse purposes, it is important that it be developed and used rationally and efficiently.

The per capita water use in developing countries is still very low. According to the World Health Organization of the United Nations, some 1,200 million people, or 30 percent of the present world population, lack safe drinking water, and 1,400 million people have no sanitary waste disposal facilities. In order to rectify the present deplorable situation, the United Nations has designated the decade 1980–1990 as the International Drinking Water Supply and Sanitation Decade. The United Nations Water Conference held at Mar del Plata, Argentina, recommended that this Decade should be devoted to implementing the national plans for drinking water supply and sanitation in accordance with the plan of action contained in the Conference Action Plan.

As the standard of living of people in developing countries improves, their per capita water consumption will increase as well. In addition, more water will be necessary for irrigation to increase agricultural production and for industrial development. Agriculture is the largest user of water, and accounts for some 80 percent of global consumption. It takes approximately 1,000 tonnes of water to grow one tonne of grain and 2,000 tonnes to grow 1 tonne of rice. Similarly, animal husbandry and fisheries require significant quantities of water. According to the Food and Agricultural Organization of the United Nations, the total area irrigated in the world amounts to 223 million hectares, of which 92 million hectares is in developing countries. Some 15 percent of the world's cropland is irrigated, but it yields from 30 to 40 percent of all agricultural production. By 1990, it is estimated that the total area irrigated in the world would increase to

273 million hectares, of which 119 million hectares would be in developing countries. This will require more water.

So far as industrial water demand is concerned, the Lima declaration and Plan of Action envisages that the total share of manufacturing output in developing countries will increase to 25 percent by the year 2000. Industrial water requirements will therefore increase substantially, if such targets are to be met successfully. Effects of such an increase in industrial activity on water will depend on availability of supply and, more importantly, on the standards set and enforced by the various administrative bodies for the quality of receiving waters. The total volume of water available in the world is constant and only less than half of one percent of the total constitutes the basic supply available to man. With increasing water requirements, scientific water management can no longer be considered as desirable, it is now an absolute must. Thus, we must take advantage of latest scientific developments to improve the planning and management processes.

One of the important scientific developments in recent years in the area of water resource management has been the application of systems analysis and computer technology. In order to assess the latest developments of the state of the art and their applications to solve real-world problems, I asked Dr. Asit K. Biswas to attend, on behalf of the United Nations Environment Programme, a scientific meeting on the subject convened in Europe. On his return, Dr. Biswas reported somewhat pessimistically that several authors were "not only reinventing the wheel, but also the wood of which the wheel is made". For whatever reasons, even though adequate knowledge was available, it was not being utilized properly. This phenomenon, of course, is not unique: it is quite common in many fields. For example, some twenty-three centuries ago, Plato graphically described how deforestation increases soil erosion and floods, and yet today simple countermeasures are not taken.

With this in mind, I asked Dr. Biswas to prepare a book which would provide relevant information on important water quality models developed in different countries thus far. Dr. Biswas, as the current President of International Society of Ecological Modelling and Vice-President of International Water Resources Association, was for me an ideal person to undertake such a task. He has assembled some of the leading international experts from seven countries to contribute to this book. Thus, I am convinced that the book will be welcomed by water scientists all over the world, and that it will be extensively used as basic material in establishing solutions for one of the most serious world problems. I hope that in this way we are implementing what Confucius said in the fifth century BC, "The essence of knowledge is, having it, to apply it."

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EDITOR'S INTRODUCTION

The total amount of water available in the world, if used more efficiently and rationally, can meet vastly higher demands to satisfy human needs. Present estimates indicate that the total volume of water on earth is 1.4×10^9 km^3, 97.3 percent of which is saline and thus cannot be used for direct human consumption. The remainder, only 2.7 percent, is fresh water, 77.2 percent of which is stored in polar ice-caps and glaciers, 22.4 percent as groundwater and soil moisture (about two-thirds lies deeper than 750 m below the surface), 0.35 percent in lakes and swamps, 0.04 percent in the atmosphere and 0.01 percent is in streams. Expressed in a different manner, nearly 90 percent of fresh water is stored in ice-caps, glaciers and as deep groundwater, and consequently not easily accessible for human use. Mostly it is surface water in rivers, streams and lakes, amounting to less than half of one percent of available fresh water, that basically constitutes the available water supply for man, even though groundwater has been heavily developed in certain parts of the world.

While the total amount of water available in the world is constant, its distribution over different regions is uneven and this contributes to tremendous variation in terms of local water availability. In addition to the geographical variability of water, its availability in all regions varies with time, depending on hydro-meteorological conditions. Much emphasis has been placed in recent decades on assessing and analyzing water availability to satisfy human needs and also simultaneously to reduce the harmful effects of droughts and floods. This is understandable since with rising global population and increases in our standard of living, the total water requirements have continued to grow. Thus, it is not surprising to find the importance placed on water availability studies.

It is important, however, to realize that availability of water does not only mean the quantity of water available for different purposes—domestic, agricultural, industrial, hydroelectric power generation, navigation, recreation, low flow augmentation, and wild-life enhancement—but also its quality. The fundamental questions that must be answered for any water development project are how much water is available and for what purpose it will be used. For example, it is quite possible that adequate quantities of water may be available for a specific purpose, but the quality of water may be quite inappropriate for that use because it is unacceptable. Furthermore, a certain quality of water that can be used for irrigation may not be used for industrial purposes and vice versa. Quality and quantity of water are closely interrelated, and both must be considered simultaneously for all water management practices. Quantity of water, without any reference to its quality, can easily be a meaningless term for overall planning and management purposes.

During the last two decades it became increasingly evident that pollution was impairing different uses of water in many parts of the world, and in certain cases even destroying possible utilization of important sources of supply. With accelerated and uncontrolled developments, more and more waste products were being discharged to water courses. In many cases not enough is known with any degree of reliability about the impacts of these pollutants on human health. Extensive monitoring and research programs have to be initiated before safe levels of specific pollutants can be determined, either individually or synergistically with other pollutants.

On a global basis there is no doubt that much better data exist on hydrometeorological variables associated with water quantity as compared to information available on different water quality elements. While reasonably accurate estimates of world water balance are known, information on its quality leaves much to be desired. Thus, with very few exceptions, even approximate continental or global assessments of different water quality parameters are not available. Nor is much known about the magnitude and type of municipal, industrial, and agricultural wastes entering water bodies, which could constitute a growing hazard to human health and pose serious adverse environmental problems. Since it is comparatively easier to control point sources of pollution, much of the emphasis in recent years has been placed on this aspect. But this clearly is not enough. For example, even for a major industrialized country like the United States, which has made a determined attempt to control water pollution, its National Commission on Water Quality reported in 1976 that 92 percent of suspended solids, 37 percent of biochemical oxygen demand and 98 percent of coliform bacteria will still remain uncontrolled in natural surface water, *even* when all discharges from point sources have been eliminated, which by itself is not an easy task. This is largely due to agricultural activities. Currently there are no general measurements of volumes of synthetic organic compounds and heavy metals reaching water courses throughout the world, some of which eventually entering the oceans.

Water plays an important part as a medium through which toxic chemicals and heavy metals are dispersed to ecosystems, other than the ones intended, both through drainage waters and by evaporation and subsequent precipitation. The importance of the latter should not be underestimated. For example, it was estimated that England received nearly 36 metric tonnes of chlorinated hydrocarbons as fallout per year at one time. The problem of acid rain, which is creating some tension between several European countries and also between Canada and the United States, is also primarily due to the evaporation–

precipitation process. Such dispersal mechanisms, including the dynamics of water transport, mean that toxic materials can be detected in locations far away from the points of application. Thus, significant quantities of pesticides, including DDT and its derivatives, have been found in animals in Antarctica, like penguins and their eggs, skua, and fish, even though such pesticides were neither necessary nor used in that area.

The process of selective concentration, by which concentration of toxic chemicals increase as they pass relatively unchanged through successive levels of food chains and food webs, further complicates and worsens the situation. This process ensures that the toxic effects of chemicals are most discernible in top carnivores. A typical example is the Minimata disease, first noticed in Minimata Bay, Japan, where continuing discharge of mercury eventually increased its overall concentration in fish to such dangerous levels that the local fishermen who depended on fish as a major source of food suffered heavily from the ill-effects of mercury poisoning. Similar problems have been noticed with pollutants like cadmium and arsenic.

Water quality management will also have other direct impacts on human health. Currently much of the population of developing countries does not have access to clean water. According to a questionnaire survey carried out in 1976 by the World Health Organization of the United Nations, in which 67 developing countries participated, only 20 percent of the rural population and 75 percent of the urban communities have access to safe water. Since rural population far exceeds the urban population in most developing countries, the combined figure for rural and urban sectors is only 35 percent. The United Nations Conference on Human Settlements, held at Vancouver, Canada, in June 1976, passed a major resolution urging provision of clean water to everyone by 1990. This target was subsequently endorsed by the United Nations Water Conference, held at Mar del Plata, Argentina, in March 1977. The importance of clean water was further demonstrated by the United Nations when the decade 1980–1990 was proclaimed as the International Water Supply and Sanitation Decade. However, judging by the advances made since the United Nations Conferences on Human Settlement and Water, it is highly unlikely that the 1990 target will be met.

Improvement in the quality of water available for human consumption will contribute to the reduction of health hazards due to diseases like cholera, typhoid, infectious hepatitis, and bacillary dysentry. It would further reduce human contacts with vectors of water-borne diseases like schistosomiasis, trypanosomiasis, and guinea worm. Current water and health situation on a global basis has been estimated as follows:

Gastro-enteritis: 400 million cases every year
Schistosomiasis: 200 million cases every year
Filariasis: 200 million cases every year
Malaria: 160 million cases every year
Onchocerciasis: 20–40 million cases every year.

These statistics clearly indicate the need for better water quality management.

While this need is easy to demonstrate beyond any reasonable doubt, the real question then becomes how to carry out such management processes effectively. One technique that has become progressively important during the last decade is the use of models to improve such processes. However, even though much of the technical advances are available and reasonably well-established, a far too high percentage of such models developed are mainly academic exercises. While no one would deny the need for developing such models, both to advance the existing state of the art and to provide "hands-on" training to engineers and scientists, there is an urgent necessity to utilize the models developed to improve the water quality management process in the real world, a fact eloquently emphasized by Dr. Tolba in his Foreword to this book.

Equally important is the urgent necessity to ensure that the current developments are easily available to practitioners and academics all over the world, so that they do not spend scarce resources in reinventing the wheel. Thus, when Dr. Tolba asked me to prepare a book which will both demonstrate the present state of the art as well as outline the major water quality models developed in different countries, I accepted the challenge enthusiastically. Initially, I considered the possibility of writing the entire book by myself. However, on reflection, it soon became evident that with my existing international commitments, it would take considerable time to complete such a major work. Furthermore, I felt that if the scientists who actually built the models were to discuss them, they could carry out the task better and quicker. Accordingly, important water quality models developed in Belgium, Canada, Denmark, England, Federal Republic of Germany, France, and the United States were selected, and eminent international authorities connected with their development were invited to prepare the chapters within an overall framework. The result is this book, which in a sense can be viewed as a companion volume to my earlier work *Systems Approach to Water Management*, also published by McGraw-Hill, and tremendously well-received by water planners and managers all over the world.

Finally, I would like to express my appreciation to Mr. Dominique Larré, currently the Director of the Industry Office of the United Nations Environment Programme, for his advice and encouragement during the preparation of this book.

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1
WATER QUALITY MODELS FOR RIVER SYSTEMS

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1-1 INTRODUCTION

The achievement of regional water quality goals, especially in the more developed areas of the world, often involves substantial capital investments and changes in public attitudes concerning resource management. The economic impacts may include not only the cost of facilities designed to reduce the discharge of contaminants into natural waters or to improve the quality of waste-receiving waters, but also any limitations on continued unrestricted economic development in a particular region or river basin. Those responsible for the formulation and approval of water quality plans or management policies must have a means of estimating and evaluating the temporal and spatial economic and environmental or ecologic impacts of these plans and policies. This need has stimulated the development and application of a wide range of mathematical modeling techniques for predicting various physical and economic impacts of alternative pollution control plans and policies.

To outline even the barest skeleton of each major type of mathematical model currently proposed, and being used, for predicting water quality in fresh water and estuarine river systems would be a lengthy undertaking. For an appreciation of just how many water quality models are available and are discussed in the current literature, readers might refer to any of the annotated bibliographies on this subject, such as those published by the Water Pollution Control Federation[1] and by the U.S. Department of the Interior.[2]

Why are there so many different types of water quality models? The answer is simply that there is a wide variety of river systems, each having its own hydrologic characteristics and its own particular pollution problems. In addition, there is often a multitude of public and private agencies involved in water quality and quantity management, and each has its own particular planning objectives and institutional, financial, and legal constraints and goals.

For any specific situation, the appropriate model and the required data depend on the purpose of the study. Broad regional long-range water quality planning does not require the detail that is appropriate, for example, when evaluating a proposed industrial waste outfall or discharge site. There is no best single water quality model for all river systems and for all planning situations. An important decision which must be made early in the planning process by those involved in water quality planning, is the selection of the modeling method or methods appropriate for their needs; these should be capable of development, calibration, validation, and execution within the limits of available time and money. This chapter will review a number of the more typical water quality predictive models developed for and applied to river systems. These models will range from the relatively simple to the relatively complex, yet each has proven to be effective in certain planning situations. The expansion of these models to include various water quality management alternatives, and their costs, will also be reviewed. Throughout, the limitations as well as the advantages of developing and applying these models will be emphasized.

1-2 TYPES OF WATER QUALITY MODELS

Before discussing some specific mathematical models for estimating water quality in river systems, certain types of models and some of their characteristics will be distinguished, thus introducing a few of the terms used by those who develop water quality models. From a manager's viewpoint, models can be classified by their applicability to various hydrologic systems, the particular aspects of that hydrologic system that are simulated by the model, and by the method of model solution or analysis. Here, the discussion will be limited to water quality models developed for streams or rivers, including those affected by tidal action.

Most of the water quality models in actual use today are extensions of two fairly simple equations proposed by Streeter and Phelps in 1925[3] for predicting the biochemical oxygen demand (BOD) and dissolved oxygen (DO) concentrations or deficits resulting from the discharge of biodegradable organic waste into river systems.[4,5]. Often used with these BOD–DO models are other fairly simple first-order exponential decay, dilution and sedimentation models for other non-conservative and conservative substances. Relatively recently more complex multi-parameter water quality models have been proposed and applied to predict more accurately the physical, chemical, and biological interactions of many constituents and organisms found in natural water bodies.[6,7] These multi-constituent water quality simulation models generally require more data and computer time, but they can also provide much more detailed and comprehensive information on the quantity and quality of water resulting from various water and land management policies.

Water quality models can be used to evaluate steady-state conditions, in which the values of the water quality and quantity variables do not change with time, or to evaluate dynamic time-varying conditions. The latter type of model permits an evaluation of transient phenomena such as nonpoint storm runoff and accidental spills of pollutants. Steady-state models are usually simpler and require less computational effort than dynamic or transient models, and are more relevant to long-term planning than to short-term management and control.

Assumptions pertaining to the mixing of pollutants in river systems dictate the spatial dimensionality of the model. Although all real physical systems are three-dimensional, sufficient accuracy may be obtained in many river systems by modeling only one or two dimensions. One-dimensional models assume complete mixing in the vertical and lateral directions. Two-dimensional models may assume either lateral mixing, as in stratified estuaries, or vertical mixing as in relatively shallow and wide rivers.

Undoubtedly, the most data-demanding model type is the stochastic or probabilistic model as compared to its deterministic counterpart. Most deterministic models yield estimates of mean values of various constituents, whereas probabilistic models explicitly take into account the randomness or uncertainty of various physical, biological, or chemical processes. Validation of stochastic models is especially difficult due to the quantity of prototype data necessary to compare probability distributions of variables rather than just their expected or mean values. This review of water quality models will be confined to one-dimensional deterministic models that are the type most widely used for both long- and short-term planning, management, and control of river and estuary systems.

1-3 COMPUTATIONAL METHODS

Model solution techniques play a significant role in model development. Solution methods range from hand computation and the use of nomographs to computer-aided optimization and simulation procedures. If the water quality management problem is simple enough to be solved manually or with the aid of a nomograph, it is by far the most inexpensive and preferable method to use. Simple models are much preferred to complex ones if they indeed provide the information needed. For more complex problems, model solutions may require thousands of computations that would be too expensive and too prone to errors if attempted without the aid of a computer.

The basic argument for using computers is that they can yield results for relatively complex simulation and optimization models quickly, accurately, and cheaply. The largest cost component of computer modeling is model development prior to calibration, verification, and simulation.

Most water quality models designed for computer solution are simulation models. Simulation models indicate the values of the water quality variables given the river flow, the quantity and quality of the waste loadings and the extent of measures designed to reduce waste discharges or to increase the waste assimilation capacity of the receiving river system. For each set of such assumptions, a simulation run is required. For many situations the number of reasonable water quality management alternatives is sufficiently large to preclude a simulation of each alternative. In cases where the time and/or cost would prohibit trial and error simulation, optimization models have been developed and applied as a means of substantially reducing the number of management alternatives that should be simulated. Optimization models are usually more limited than simulation models, especially with respect to the number of water quality constituents, but they explicitly include variables defining the range of management alternatives and objectives being considered. If used properly, optimization models can assist in identifying those management alternatives that best satisfy management objectives Examining these particular management alternatives with a verified simulation model provides a more accurate basis for comparing their economic and environmental impacts.

Simulation models are usually solved either by formal integration of the basic differential equations or with the aid of numerical analysis techniques, such as finite difference or finite element methods. Each of these solution procedures require the solution of simultaneous sets of linear or nonlinear equations. Optimization models are often specifically structured for solution by a particular constrained optimization (mathematical programming) procedure (algorithm). Two of the most commonly used optimization methods are linear and dynamic programming, the former being preprogrammed for computer solution and readily available at most scientific computer facilities throughout the developed world. Occasionally nonlinear or mixed-integer programming algorithms are used to solve water quality optimization models.

1-4 MODEL DEVELOPMENT, CALIBRATION AND VERIFICATION

A water quality model is simply a set of mathematical expressions defining the physical, biological and chemical processes that are assumed to take place in a water body. A water quality management (or optimization) model will include also one or more mathematical expressions defining alternative management options and the management objectives which are either to be maximized or minimized. Given a set of streamflow conditions, wastewater discharges, and the like, the model will provide a set of outputs that include values for each of the water quality variables for each time period and location in the river system.

The common basis of most water quality models is the principle of continuity or mass balance. Given the particular water quality constituents of interest and the important physical, biological and chemical processes that affect these quality constituents, a mass balance is developed that takes into account three phenomena: the inputs of constituents to the river system from outside the system, the transport of constituents through the river system, and the reactions within the river system that either increase or decrease constituent concentrations or masses.

The inputs of pollutant constituents to rivers usually come from natural processes and wastewater discharges of municipal, industrial or agricultural activities, in the form of point sources or nonpoint runoff. The physical and biochemical characteristics of the point waste sources are much more understood, and measurable, than those of the nonpoint sources. The complex, often random, time-variable nature of both point and nonpoint sources is often disregarded because of insufficient knowledge and modeling complexity, with a consequent reduction in model output reliability.

The transport of constituents, by dispersion and/or advection, is dependent on the hydrologic and hydrodynamic characteristics of the river. Advective transport dominates river flow that results primarily from surface water runoff and groundwater inflow. In contrast, dispersion is the predominant transport phenomenon in estuaries or rivers subjected to tidal action. As one would expect, variations in surface water runoff and groundwater inflow have more of an effect on freshwater river quality than on estuarine river quality.

Transport phenomena of freshwater and estuarine river systems are better understood than those of lakes and oceans. Hence, water quality models of river systems tend to be more reliable than those of lakes and oceans. In addition, steady-state assumptions, often made to reduce model complexity and data requirements, are usually more realistic for rivers than for lakes and oceans. While steady-state assumptions may be more realistic for rivers, such assumptions preclude an adequate evaluation of the effects of time-variable inputs of constituents to river systems.

Biological, chemical and physical reactions among constituents are also better understood in river systems than in lakes or oceans. This is especially true for wastes affecting the biochemical oxygen demand and dissolved oxygen in the river, and for wastes affecting the concentration of bacteria, various forms of carbon and nitrogen, and chemical compounds of simple structure commonly contained in industrial effluents. Knowledge of reactions involving heavy metals and many complex synthetic compounds and toxic materials is relatively limited. The interactive nonlinear and time-varying reactions of nutrients that cause eutrophication are sufficiently understood to permit some reasonable modeling of these constituents, but these fairly sophisticated and complex models require considerable data and time to develop, calibrate and verify. Such eutrophication models are often of greater relevance to lake quality management than to river quality management.

The development of models is both a science and an art. Although most models are developed according to some basic modeling procedures, each model reflects the creativity of its developer. A model reflects what the developer sees as the important components, and as the relationships between components, of the prototype. The modeler must decide just how much detail to include in the model. This in turn is a function of the purpose of the model, i.e., its intended use, and of how quickly the model is needed for planning and decision making.

Once the general outline and purpose of the model is defined, model development proceeds from conceptualization (sometimes just in words), to mathematical definition or representation, to modification if necessary for computational reasons (e.g., the incorporation of piecewise linear functions, finite differences or finite elements), to calibration, to verification and sensitivity testing, to documentation, and finally to use in defining and evaluating possible policy alternatives. If the results of any model are to be useful in a particular river water quality planning or control situation, the model must be calibrated and verified.

Model calibration is performed using one or more observed data sets of both inputs and outputs. The model parameters and indeed the model itself are adjusted or modified so as to produce an output that is as close to the actual observed water quality as is possible. This is usually a subjective trial and error procedure, again reflecting the art more than the science of modeling.

Model verification requires an independent set of input and output data to test the calibrated model. The verification data must be independent of that used to calibrate the model. A model is verified if the model's predictions, for a range of conditions, compares favorably with observed field data. Here again the criteria for deciding whether or not model output and observed field data are essentially the same, for the same input conditions, are largely subjective. What constitutes a satisfactory comparison depends on the nature of the problem, the type of model developed and its purpose, and the extent and reliability of available input and output data. The fact that most models cannot accurately predict what actually happens does not detract from their value in helping planners understand the prototype and estimate the relative change in water quality associated with given changes in the inputs, resulting from alternative wastewater management policies. (For example, see the response surfaces developed for the Delaware Estuary by Spofford and Kelly.[8,9])

1-5 SOME ILLUSTRATIVE STEADY STATE MODELS

In this section, some models will be developed to illustrate a few of the basic approaches to water quality modeling of river systems. This will be a discussion of some of the main mathematical expressions or components of water quality models which may apply in various situations.

First, consider a one-dimensional river, extending from a small stream to the tidal river and finally to the saline estuary. The quantity or mass of various constituents $C_i, i = 1, 2, \ldots,$ is a function of the rate of inputs and outputs (sources and sinks) of the constituents, of the dispersion and advection of the constituents, and of the various physical, chemical, biological and possibly radiological reactions that affect the constituents.

The partial differential equation defining the mass balance of any constituent C in a one-dimensional river system is

$$\frac{\partial C}{\partial t} = \frac{\partial}{\partial X}\left(E\frac{\partial C}{\partial X} - UC\right) \pm \sum_k S_k \tag{1-1}$$

in which C is the concentration (ML^{-3}) of a particular constituent, t is time (T), X is the distance (L) downstream from the source of the constituent, E is the dispersion coefficient (L^2T^{-1}), U is net downstream velocity (LT^{-1}) and S_k is a source or sink ($ML^{-3}T^{-1}$) of the constituent C. Equation (1-1) states that at a particular site in the river system, the change in concentration with respect to time, $\partial C/\partial t$, equals the change in the X direction due to dispersion $E(\partial C/\partial X)$, and advection UC, plus any sources or minus any sinks S_k. These source or sink terms include the various reactions that may take place either increasing or decreasing the concentration of constituent C at a particular site in the river system.

The expression within the parentheses in Eq. (1-1) is termed the flux. Flux due to dispersion in the X direction, $E(\partial C/\partial X)$, is assumed to be proportional to the concentration gradient in the direction of decreasing concentration. Constituents are transferred by dispersion from zones of higher concentration to zones of lower concentration. The coefficient of dispersion E, depends, in part, on the amplitude and frequency of the tide. The term UC is the advective flux caused by the movement of water containing the constituent.

Many of the reactions affecting the decrease or increase of constituent concentrations are often represented by first-order kinetics that assume the reaction rates are proportional to the constituent concentrations. While higher order kinetics may be more correct, constituent concentration predictions based on first-order assumptions are usually acceptable in natural river systems.

1-5.1 Static Models

Assuming a constant streamflow, constant inputs of constituents, and a constant dispersion coefficient, Eq. (1-1) for steady-state conditions in one dimensional streams, rivers and estuaries reduces to

$$\frac{\partial C}{\partial t} = 0 = E\frac{d^2C}{dX^2} - U\frac{dC}{dX} \pm \sum_k S_k \tag{1-2}$$

where $\sum_k S_k$ may be 0 if the constituent C is conservative or equal to a reaction rate $K(T^{-1})$ times the constituent concentration C, if the constituent is nonconservative. The net downstream velocity U is averaged over the tidal cycle. This steady-state equation may apply to low-flowing river systems, conditions often found in late summer or early fall. It is at this time when high temperatures and low velocities are coupled with low quantities of dilution water that the most severe water quality conditions may occur.

Equation (1-2) may be integrated to give an equation for predicting the constituent concentration at any location X downstream from a point $X = 0$ of constant constituent input rate $W_0(MT^{-1})$.[10] Assuming a steady-state flow rate (L^3T^{-1}) of Q_X at site X

$$C_X = \frac{W_0}{Q_X m} \exp\left\{\frac{UX}{2E}(1 \pm m)\right\} \tag{1-3}$$

where

$$m = (1 + 4KE/U^2)^{1/2} \tag{1-3a}$$

In Eqs. (1-2) and (1-3) the parameters E, U and K are assumed constant. Equation (1-3) assumes no sinks except for decay. The positive sign in the exponent $(1 + m)$ pertains to the region above the point of discharge or outfall ($X < 0$), and the negative sign $(1 - m)$ refers to the region downstream of the outfall. The velocity term U is always nonnegative. Note that for conditions of complete mixing at the point of discharge, the initial constituent concentration at $X = 0$ is

$$C_0 = W_0/Q_0 m \tag{1-4}$$

In freshwater rivers not under a tidal influence, E approaches 0 and hence m approaches 1. In these conditions, $C_0 = W_0/Q_0$, which highlights the freshwater flux Q_0 of these sections of the river.

As the river approaches the sea, the dispersion E increases and the net freshwater velocity U decreases. Recalling that the flow Q equals the cross-sectional area A times the velocity, $Q = AU$, and m can be expressed as $[(U^2 + 4KE)^{1/2}]/U$, then as U approaches 0, the term $Q_0 m$ in Eq. (1-4) approaches $A\sqrt{4KE}$, which can be regarded as the dispersive flux of a nonconservative constituent. Hence for estuaries, the initial concentration, $C_0 = W_0/A\sqrt{4KE}$.

In river systems where the dispersion coefficient E is negligible, the constituent concentrations downstream of $X = 0$ can be estimated by assuming $E = 0$ and then integrating Eq. (1-2).

$$C_X = \frac{W_0}{Q_X} \exp(-KU/X) \tag{1-5}$$

This represents an exponential decay over the time the constituent travels downstream from a single point of discharge at $X = 0$.

Comparing Eq. (1-3) with Eq. (1-5), note that if the dispersion coefficient is relatively large compared to the velocity U, then the exponent in Eq. (1-3) approaches a function of the square root of K, rather than of K as in Eq. (1-5). Consequently, predictions of constituent concentrations in estuaries tend to be less sensitive to errors in the estimation of the decay rate constant K than are predictions in nontidal rivers.

To illustrate these series of one-dimensional steady-state equations, consider the discharge of wastes having a biochemical oxygen demand (BOD) from multiple point sources along a river system. Let the index or subscript i represent the particular sites along a river system at which quantities of carbonaceous BOD, BOD_i^c, and nitrogenous BOD, BOD_i^n, are being discharged at a constant rate (MT^{-1}). Each waste at each source i will have its unique reaction rate constants, K_i^c and K_i^n.

The total BOD concentration at any site j in the river will equal the sum of all the quantities of carbonaceous and nitrogenous BOD at site j resulting from each source site, divided by the flux, either Q_j or $A\sqrt{4KE}$ as appropriate, at that site. Thus for sections of rivers in which the dispersion coefficient E is negligible, the total BOD at site j will equal

$$BOD_j = \frac{1}{Q_j}\left\{\sum_i\left[BOD_i^c \exp\left(-K_i^c \frac{X_{ij}}{U_{ij}}\right) + BOD_i^n \exp\left(-K_i^n \frac{X_{ij} - x_i}{U_{ij}}\right)\right]\right\} + \hat{BOD}_j \tag{1-6}$$

in which X_{ij}/U_{ij} denotes the average time of flow from points i to j, the sum, $\sum_i$, is over all sites i upstream of site j, and $\hat{BOD}_j$ is the concentration of BOD at j resulting from other than the upstream point sources i. The constant x_i is an assumed known distance subtracted from X_{ij} $(X_{ij} > X_i)$ to approximate the lag in nitrogenous BOD. This lag is, in part, a function of the degree of wastewater treatment at each source i.

For those sites j in sections of the river system that are in transition from purely freshwater to totally estuarine, the BOD concentration at site j resulting from BOD discharges at sites i is

$$\begin{aligned} BOD_j = \frac{1}{Q_j}\sum_i\left\{\frac{BOD_i^c}{m_i^c}\exp\left[\frac{U_{ij}X_{ij}}{2E}(1 \pm m_i^c)\right]\right.\\ \left. + \frac{BOD_i^n}{m_i^n}\exp\left[\frac{U_{ij}(X_{ij} - x_i)}{2E}(1 \pm m_i^n)\right]\right\} \end{aligned} \tag{1-7}$$

In the above equation, m_i^c and m_i^n are similar to the m defined by Eq. (1-3a); however, the K in Eq. (1-3a) is replaced by K_i^c and K_i^n respectively. The dispersion coefficient, E, is the average coefficient between sites i and j. In this case the sum, $\sum_i$, is over all sites i, and the term $(1 - m_i)$ applies if a site i is upstream of site j and, conversely, $(1 + m_i)$ applies if i is downstream of j.

Finally, for predominantly estuarine systems, the velocity U is negligible and the exponents in Eq. (1-7), rewritten as

$$\frac{UX}{2E}\left[\frac{U \pm (U^2 + 4KE)^{1/2}}{U}\right]$$

approach the form $\pm X\sqrt{K/E}$ (+ if $X < 0$, − if $X > 0$), and the terms $Q_j m_i$ become $2A\sqrt{K_i E}$. Hence, for estuaries, the total BOD concentration at site j

resulting from BOD discharges at sites i is

$$\mathrm{BOD}_j = \frac{1}{2A}\sum_i \left[\frac{\mathrm{BOD}_i^c}{\sqrt{K_i^c E}} \exp(\pm X_{ij}\sqrt{K_i^c/E}) + \frac{\mathrm{BOD}_i^n}{\sqrt{K_i^n E}} \exp(\pm (X_{ij} - x_i)\sqrt{K_i^n/E})\right] \tag{1-8}$$

1-5.2 Coupled Reactions

Some constituent concentrations are functions of two or more simultaneous reactions. Consider, for example, dissolved oxygen. The appropriate continuity equation for the dissolved deficit concentration, D, in steady-state conditions is

$$0 = E\frac{d^2D}{dX^2} - U\frac{dD}{dX} + K^c\mathrm{BOD}_X^c + K^n\mathrm{BOD}_X^n - K_a D \tag{1-9}$$

where K_a is the reaeration rate constant (T^{-1}), and each BOD concentration

$$\mathrm{BOD}_X = \mathrm{BOD}_0 \exp\left[\frac{UX}{2E}(1 \pm m)\right]$$

as defined by Eqs. (1-3) and (1-3a). Integration of Eq. (1-9) yields the deficit concentration at any site j resulting from the discharge of carbonaceous and nitrogenous BOD at sites i.

$$D_j = \frac{1}{Q_j}\sum_i \left(\frac{K_i^c\mathrm{BOD}_i^c}{(K_a - K_i^c)}\left\{\frac{1}{m_i^c}\exp\left[\frac{UX}{2E}(1 \pm m_i^c)\right] - \frac{1}{m_a}\exp\left[\frac{UX}{2E}(1 \pm m_a)\right]\right\}\right.$$
$$\left. + \frac{K_i^n\mathrm{BOD}_i^n}{(K_a - K_i^n)}\left\{\frac{1}{m_i^n}\exp\left[\frac{U(X - x)}{2E}(1 \pm m_i^n)\right] - \frac{1}{m_a}\exp\left[\frac{UX}{2E}(1 \pm m_a)\right]\right\}\right) \tag{1-10}$$

where

$$m_a = \sqrt{1 + 4K_a E/U^2}$$

and K_a is understood to be the appropriate reaeration constant between sites i and j. In nontidal freshwater systems, E approaches 0 and each m approaches 1, and the exponents are of the form $-KX/U$ as in Eq. (1-6).

In estuaries, U approaches 0, and the dissolved oxygen deficit, D, resulting from the discharge of BOD from point source sites i is

$$D_j = \frac{1}{2A}\sum_i \left(\frac{K_i^c\mathrm{BOD}_i^c}{(K_a - K_i^c)}\left\{\frac{\exp(\pm X_{ij}\sqrt{K_i^c/E})}{\sqrt{K_i^c E}} - \frac{\exp(\pm X_{ij}\sqrt{K_a/E})}{\sqrt{K_a E}}\right\}\right.$$
$$\left. + \frac{K_i^n\mathrm{BOD}_i^n}{(K_a - K_i^n)}\left\{\frac{\exp[\pm(X_{ij} - x_i)\sqrt{K_i^n/E}}{\sqrt{K_i^n E}} - \frac{\exp(\pm X_{ij}\sqrt{K_a/E})}{\sqrt{K_a E}}\right\}\right) \tag{1-11}$$

The exponents of e are always negative, regardless of the sign of X, since if $X < 0$, then $+X$ applies, and vice versa.

The dissolved oxygen (DO) concentration at any site j is simply the DO concentration that would exist for the steady-state flow conditions assuming no point discharges BOD_i, less the deficit D_j, provided the actual DO concentrations in the river system do not get too low, i.e., near zero. If anaerobic conditions exist, the reaction rates K change, and Eqs. (1-6) to (1-11) are no longer valid.

Modifications of these equations, identification of their parameters, and methods of incorporating differential or integrated equations into models of actual river systems to evaluate water quality management alternatives are discussed in more detail by Thomann,[5] Dorfman, et al.[11] and in Biswas.[4,7] Applications of such equations to river systems have been extensive.[12] Although they do not capture the detail of time-varying multi-species aquatic ecosystem models now being developed, they require less data. Hence they may be more useful to those responsible for water quality planning and management in situations where data availability or even time and money, precludes the application of more detailed models.[12,13,14]

In river systems where nitrogen is a major constituent, the assumption of a constant reaction, K_i^n, as in Eq. (1-11), may not be appropriate. The reaction may vary along the river system simply because of the changing concentrations of the various forms of nitrogen, each form of nitrogen having its own reaction rate constant. A modification of Eq. (1-11), together with data on the input of each form of nitrogen and its corresponding reaction rate, will enable the prediction of the separate components of the nitrification process. O'Connor, et al.[7] proposed some sequential reaction models to account for each nitrogen component.

Consider the following four nitrogen concentration components, (ML^{-3}):

N_1 = organic nitrogen,
N_2 = ammonia nitrogen, NH_3
N_3 = nitrite nitrogen, NO_2
N_4 = nitrate nitrogen, NO_3

Let K_i^n represent the first-order decay (including settling) of nitrogen form $i(T^{-1})$, $K_{i,i+1}^n$ represent the forward reaction coefficient (T^{-1}), and $W_i(X)$ the discharge of form i at site $X(MT^{-1})$. Recalling that E is the dispersion coefficient (applicable to estuaries and in units of L^2T^{-1}), and U is the net downstream velocity (LT^{-1}), the solution of the following four equations will permit a more detailed and realistic prediction of the concentration of each form of nitrogen in steady-state river systems.

$$0 = E\frac{d^2N_1}{dX^2} - U\frac{dN_1}{dX} - K_1^nN_1 + W_1(X)$$

$$0 = E\frac{d^2N_i}{dX^2} - U\frac{dN_i}{dX} - K_i^nN_i + K_{i-1,i}^nN_{i-1} + W_i(X) \quad i = 2, 3, 4 \quad (1\text{-}12)$$

The decrease in the dissolved oxygen concentration in such river systems as a result of the individual nitrogen constituents is caused by ammonia oxidation, $3.43\,K_{23}^nN_2$, and nitrite oxidation, $1.14\,K_{34}^nN_3$. The dissolved oxygen deficit concentration, D'', resulting from the oxidation of these two nitrogen forms can be predicted by the equation

$$0 = E\frac{d^2D''}{dX^2} - U\frac{dD''}{dX} - K_aD'' + 3.43\,K_{23}''N_2(X) + 1.14\,K_{34}''N_3(X) \quad (1\text{-}13)$$

where K_a is the reaeration rate constant (T^{-1}) and $N_2(X)$ and $N_3(X)$ are obtained from the solution of Eqs. (1-12).

These models can be further expanded to include denitrification and algal utilization. Denitrification may involve the reduction of NO_3 to NO_2 and the conversion of NO_2 to N_2 gas. These reactions occur under conditions of low dissolved oxygen. Assimilatory nitrate reduction can convert NO_3 to NH_3. The utilization of ammonia nitrogen and nitrate nitrogen by phytoplankton may also take place, and hence produce organic nitrogen, thereby completing a very simplified version of the nitrogen cycle. This process is generally defined by reaction rate constants K_{ij}'' for i and j equal to 1, 2, 3 and 4. Feed forward reactions involve reaction rates $K_{ij}''(i < j)$ where j is usually $i + 1$, and feedback reactions involve reaction rates $K_{ij}''(i > j)$, where j is usually $i - 1$. All these reactions $(K_{ij}''N_i)$ are then included in Eqs. (1-12).

$$0 = E\frac{d^2N_i}{dX^2} - U\frac{dN_i}{dX} - K_i''N_i + \sum_{j \neq i}(K_{ji}''N_j) + W_i(X) \qquad i = 1, 2, 3, 4 \quad (1\text{-}14)$$

In general, each $K_i'' \geqslant \sum_j K_{ij}'', j \neq i$.

Using finite difference approximations of Eqs. (1-12), four algebraic equations, one for each form of nitrogen i, can be written for each finite section of the river system, k, as appropriate.

$$\begin{aligned} 0 = \sum_j [-Q_{kj}(\alpha_{kj}N_{ik} + \beta_{kj}N_{ij}) + E_{kj}'(N_{ij} - N_{ik})] \\ - V_k\left[K_{ik}''N_{ik} + \sum_h K_{hik}''N_{hk}\right] + W_{ik} \qquad i = 1, 2, 3, 4, \qquad \forall k \end{aligned} \quad (1\text{-}15)$$

In Eq. (1-15), Q_{kj} is the net flow (L^3T^{-1}) from an adjacent section k to section j (+ if from k to j, − if from j to k), V_k is the volume (L^3) of segment k, α_{kj} is a finite difference weight $(\max(1/2, 1 - E'/Q))$, β_{kj} is $1 - \alpha_{kj}$, and E_{kj}' is the exchange (bulk dispersion) coefficient (L^3T^{-1}) defined by

$$E_{kj}' = 2(E_{kj}A_{kj})/(L_k + L_j) \quad (1\text{-}16)$$

The variables N_{ik} are the concentrations (ML^{-3}) of nitrogen form i in section k. Parameters K_{hik}'' are the reaction rate constants (T^{-1}) for the conversion of nitrogen form h to form i in section k, and W_{ik} is the direct discharge (MT^{-1}) of nitrogen form i into section k. In Eq. (1-16), E_{kj} is the dispersion coefficient (L^2T^{-1}) between sections k and j, A_{kj} is the interfacial cross-section area (L^2), and L_k and L_j are the lengths (L) of sections k and j.

The spatial distribution of the dissolved oxygen deficit concentration, D_k'', due to ammonia oxidation NH_3 to NO_2 (reaction rate, $3.43\,K_{23}''$), and nitrite oxidation, NO_2 to NO_3 (reaction rate, $1.14\,K_{34}''$), can be predicted from the simultaneous solution of similar finite difference equations.

$$0 = \sum_j [-Q_{kj}(\alpha_{kj}D_k'' + \beta_{kj}D_j'') + E_{kj}'(D_j'' - D_k'')]$$
$$- V_k K_{ak} D_k'' + V_k(3.43)K_{23}'' N_{1k} + V_k(1.14)K_{34}'' N_{3k} \qquad (1\text{-}17)$$

where $K_{ak}(T^{-1})$ is the reaeration rate constant for section k.

Equations (1-15) and (1-17) represent five equations for each section of the river. They can be combined and put into a convenient matrix form for solution using any of a number of available computer programs for solving large sets of linear equations. Since some of the feedback reaction coefficient values are functions of the dissolved oxygen deficit, parameter adjustments will be necessary to insure an adequate prediction of the concentrations of dissolved oxygen and each form of nitrogen. Just how these adjustments are made is indeed part of the art of water quality modeling.

The models discussed in this section are all first-order kinetic steady-state models. While they vary in complexity and in their requirements for data, they remain relatively simple models. In these simple models, flows and temperatures do not vary with time and the complex nonlinear kinetic interactions between the microorganisms and the constituents are approximated by linear or first-order reactions.

While for more detailed studies nonlinear nonsteady-state models may be required, many problems can be approached by first assuming steady-state conditions and first-order reaction kinetics. Such assumptions certainly simplify the mathematics and the solution techniques. Unless the problem context and available data warrant more complex models, these relatively simple models can be, and have been, used to help understand and solve a variety of river system water quality management models.

1-5.3 Water Quality Indices

Before discussing some time-varying models, the use of water quality indices should be mentioned. There are many water quality indices. One, proposed by Meta Systems, Inc.,[15] defines a biomass potential to indicate the extent to which nitrogen and other nutrients in waste substances affect the biological activity of natural river systems. For specified hydraulic conditions, the biomass potential, BP, is defined as a linear combination of the five-day carbonaceous biochemical oxygen demand, BOD_5^c, the total (organic) reducible nitrogen, TRN, and the biologically available phosphorus, P.

$$BP = 1.47\,BOD_5^c + 4.57\,TRN + 30P \qquad (1\text{-}18)$$

The first two terms of the right-hand side of Eq. (1-18) are the stoichiometric estimates of ultimate BOD that will result from the continued oxidation of both carbonaceous and nitrogenous material. The coefficient of 30 is an estimate of the contribution of phosphorus to the biomass if phosphorus is a limiting element.

Over a range of types of organic waste and aquatic microorganisms there exists an approximate equivalence between biomass potential as defined by Eq. (1-18) and ultimate biochemical oxygen demand of the organic wastes; i.e., the decay and decomposition of one gram of biomass potential requires one gram of dissolved oxygen.

The impairment of a stream's aesthetic qualities or its utility for recreation or water supply caused by given input (MT^{-1}) or concentration (ML^{-3}) of biomass potential is dependent on the stream size, shape, temperature and flow. Assuming that toxic materials are not present, an impairment index I, of a river system due to the impact of degradable organics containing BOD, TRN and P can be defined as a function of the added biomass potential, the mean residence time of nutrients in the river system, and the volume and surface area of the river system.

$$I = \left(\frac{\mathrm{BP}}{Q}\right)\left(\frac{X}{U}\right)^{\delta}\left(\frac{\text{volume of affected reaches}}{\text{surface area of affected reaches}}\right)^{\mu} \tag{1-19}$$

In Eq. (1-19), I is the impairment index, Q is the average flow in the river section from the outfall to the basin outlet, X/U is the average time of flow in that river section, and weights δ and μ reflect the relative importance of flow times and stream geometry. This particular index was used in a proposed effluent charge program for water quality control.[15]

1-5.4 Time-Varying Models

When required, more realistic representations of natural river systems with steady-state flows can be obtained from integrating, or approximating (using finite differences or finite elements), the following form of the one-dimensional continuity or transport equation (Eq. (1-1))

$$\frac{\partial C}{\partial t} = \frac{1}{A}\frac{\partial}{\partial X}\left(EA\frac{\partial C}{\partial X}\right) - \frac{1}{A}\frac{\partial}{\partial X}(CQ) \pm \sum_k S_k(U, X, t) \tag{1-20}$$

where again C is the constituent (degradable or nondegradable) concentration (ML^{-3}), t is time (T), E is the dispersion coefficient (L^2T^{-1}), A is the cross-section area (L^2) at location X, X is the longitudinal river distance (L), Q is the net downstream freshwater flow (L^3T^{-1}) and $\sum_k S_k$ are the sources and sinks ($ML^{-3}T^{-1}$) of the constituent. The problem is to find a general solution for the concentration C at all locations X at times t.

Investigations reported by O'Connor[16,17] and Tracor Associates,[18] are among many that have applied Eq. (1-20), or modifications of it, to river systems having steady-state flows. For example, Eq. (1-20) can be transformed to the following form for predicting biochemical oxygen demand, BOD(X, t) and dissolved oxygen concentrations, DO(X, t), in river systems.[5,16]

$$\frac{\mathrm{BOD}(X,t)}{\partial t} = \frac{1}{A}\frac{\partial}{\partial X}\left[EA\frac{\partial \mathrm{BOD}(X,t)}{\partial X}\right] - \frac{1}{A}\frac{\partial}{\partial X}\left[Q(X)\cdot \mathrm{BOD}(X,t)\right] - K[\mathrm{BOD}(X,t)] + \frac{\mathrm{BOD}_r}{A}\left[\frac{\partial Q(X)}{\partial X}\right] \tag{1-20a}$$

$$\frac{\partial \mathrm{DO}(X,t)}{\partial t} = \frac{1}{A}\frac{\partial}{\partial X}\left[EA\frac{\partial \mathrm{DO}(X,t)}{\partial X}\right] - \frac{1}{A}\frac{\partial[\mathrm{DO}(X,t)Q(x)]}{\partial X} + K_a[\mathrm{DO}_{\mathrm{SAT}}(X,t) - \mathrm{DO}(X,t)] - K^c\mathrm{BOD}^c(X,t) - K^n\mathrm{BOD}^n(X,t) + P(X,t) - R(X,t) - S_B(X,t) + \frac{\mathrm{DO}_r}{A}\left[\frac{\partial Q(X)}{\partial X}\right] \tag{1-20b}$$

where the terms, in units of $ML^{-3}T^{-1}$, in the right-hand side of Eq. (1-20a), from left to right, are the tidal dispersive and advective transport, the decay, and the BOD addition (BOD_r in ML^{-3}) from distributed runoff along the river. The right-hand side terms of Eq. (1-20b) in the same units and also from left to right, are the longitudinal dispersive and advective transport, reaeration as a function of the dissolved oxygen deficit (saturation concentration less the actual concentration), deoxygenation from carbonaceous and nitrogenous BOD, DO addition due to photosynthesis and reduction due to respiration and from BOD in benthal or bottom deposits, and DO addition (DO_r in ML^{-3}) from distributed runoff along the river.

Equations (1-20) can be integrated to give the one-dimensional time-varying longitudinal distribution of dissolved oxygen deficit concentrations, $D(X,t)$, in nontidal streams having steady-state flows $Q(X)$.[5,17]

$$D(X,t) = D(0,t)e_a \tag{1-21a}$$

$$+ \frac{\mathrm{BOD}^c(0,t)}{Q(X)}\left(\frac{K^c}{K_a - K^c}\right)(e_c - e_a) \tag{1-21b}$$

$$+ \frac{\mathrm{BOD}^n(0,t)}{Q(X)}\left(\frac{K^n}{K_a - K^n}\right)(e_n - e_a) \tag{1-21c}$$

$$+ \mathrm{BOD}_r^c\left(\frac{1-e_a}{K_a} - \frac{e_c - e_a}{K_a - K^c}\right) \tag{1-21d}$$

$$+ \frac{R - P + S}{K_a}(1 - e_a) \tag{1-21e}$$

$$+ F(K_a, U, X, p, t) \tag{1-21f}$$

where $D(X, t)$ = dissolved oxygen deficit concentration at location X and time $t(ML^{-3})$

$D(0, t)$ = initial ($X = 0$) deficit concentration, including that from waste flow (ML^{-3})

$e_a = \exp(-K_a X/U)$

U = average velocity from 0 to $X(LT^{-1})$

K_a = reaeration rate constant for flow conditions from 0 to X (T^{-1})

$\mathrm{BOD}^c(0, t)$ = rate of carbonaceous BOD input at $X = 0$ at time $t(MT^{-1})$

$Q(X)$ = streamflow at location $X(L^3T^{-1})$

K^c = deoxygenation rate constant for carbonaceous BOD in river (T^{-1})

$e_c = \exp(-K^c X/U)$

$\mathrm{BOD}^n(0, t)$ = rate of nitrogenous BOD input at $X = 0$ at time $t(MT^{-1})$

$e_n = \exp(-K^n X/U)$

BOD_r^c = uniformly distributed carbonaceous BOD input in runoff along river $(ML^{-3}T^{-1})$

R = average oxygen reduction rate from respiration $(ML^{-3}T^{-1})$

P = average oxygen production rate from photosynthesis $(ML^{-3}T^{-1})$

S = average oxygen reduction rate from benthal demand $(ML^{-3}T^{-1})$

$F(\cdot)$ = Fourier function defining the cyclical variation in dissolved oxygen concentration throughout the day at a site $X(ML^{-3})$

p = period of the photosynthetic wave

t = time (T)

Portions of Eq. (1-21) represent different effects. Part (a) is the contribution to the downstream dissolved oxygen deficit from initial deficit conditions. Parts (b) and (c) are the contributions resulting from point source discharges of carbonaceous and nitrogenous BOD wastes. Part (d) represents the deficit contribution from uniformly distributed carbonaceous BOD sources such as from runoff, when the quantity of runoff is small relative to the streamflow. A similar expression could apply to nitrogenous BOD also. Part (e) is the net deficit resulting from the time and distance averaged respiration, photosynthesis, and benthal demand effects on the deficit concentration. Part (f) accounts for the daily cyclical variation due to plant photosynthesis and has taken on a variety of forms. Because each part of Eq. (1-21) is linear, those parts that are applicable for a particular river system can be added together to account for all contributions to the oxygen deficit.

If the time variation of wastewater inputs, $\mathrm{BOD}(X, t)$, is substantial, the above equation for nondispersive streams may not be a very good predictor, simply because there will be dispersion. But in any case the time parameter t of the time-varying wastewater inputs, $\mathrm{BOD}(0, t)$, must be considered together with the time of flow, X/U, in order to achieve a reasonable prediction of the effects of time-varying waste inputs at various locations X.

Equation (1-21) illustrates the continuous solution approach to water quality modeling. The finite section solution approach involves dividing each river system into reaches or finite sections, developing a continuity equation for each reach, and solving all reach equations simultaneously.

In a finite section approach, each reach is assumed to be completely mixed. The mass balance equation for the ith reach is written as

$$V_i \frac{dC_i}{dt} = Q_{i-1,i}(\alpha_{i-1,i}C_{i-1} + \beta_{i-1,i}C_i) - Q_{i,i+1}(\alpha_{i,i+1}C_i + \beta_{i,i+1}C_{i+1}) + E'_{i-1,i}(C_{i-1} - C_i) + E'_{i,i+1}(C_{i+1} - C_i) - K_i V_i C_i + W_i \pm \sum_k S_{ki} \tag{1-22}$$

where V_i = volume of water in reach $i(L^3)$
C_i = concentration of constituent in reach $i(ML^{-3})$
$Q_{i,i+1}$ = flow between reaches i and $i+1(L^3T^{-1})$
$E'_{i,i+1}$ = exchange coefficient over reaches i and $i+1(L^3T^{-1})$
$\alpha_{i,i+1}$ = dimensionless mixing coefficient between reaches i and $i+1$
$\beta_{i,i+1} = 1 - \alpha_{i,i+1}$
K_i = reaction rate constant for constituent in reach $i(T^{-1})$
W_i = rate of constituent addition to reach $i(MT^{-1})$
$\sum_k S_{ki}$ = sum of sinks and sources for each reach $i(MT^{-1})$

These continuous and finite discrete models permit an estimation of the spatial and time distribution of water quality constituents in terms of various rate constants. The problem of forecasting water quality involves measuring these rate constants in the field for a particular river system, developing the model in a form that can be solved, calibrating and verifying it, and then interpreting the results of various solutions based on alternative management plans.

1-6 AQUATIC ECOSYSTEM SIMULATION MODELS

Over the past decade there has been an increasing emphasis on the effects of various constituents, especially nutrients, on the aquatic ecosystem, i.e., on the production of phytoplankton, zooplankton, fish and other trophic levels within natural water bodies.[6] Perhaps typical of many of the operational aquatic ecosystem simulation models is QUAL II, developed for the U.S. Environmental Protection Agency by Water Resources Engineers, Inc.[19] QUAL II numerically integrates Eq. (1-1) or Eq. (1-20) for a variety of water quality constituents, including conservative substances; chlorophyll a; ammonia, nitrite and nitrate nitrogen; phosphorus; carbonaceous biochemical and benthic oxygen demand; dissolved oxygen; coliforms and radio nuclides. The following few paragraphs define the mathematical relationships that describe the individual reactions and interactions among these constituents.

Chlorophyll a concentration C is considered to be proportional to the concentration of phytoplanktonic algal biomass, A.

$$C = \alpha_0 A \tag{1-23}$$

The time-varying (dA/dt) growth and production of algal biomass, A, is dependent on the growth rate of algae, $\mu_A(T^{-1})$, the respiration rate of algae, $\rho(T^{-1})$, the settling rate for algae, $\sigma_1(LT^{-1})$, and the average stream depth, $D_a(L)$, all at a particular location X in the river system. Although not in the QUAL II model, the concentration of algae is also a function of mortality, M, and of grazing, G_Z, by higher trophic levels such as zooplankton, Z. Hence,

$$\frac{dA}{dt} = \mu_A A - \rho A - \frac{\sigma_1 A}{D_a} - MA - G_Z \tag{1-24}$$

The growth rate of algae is dependent on the temperature and availability of nutrients (nitrogen, carbon, phosphorus) and light. The standard Michaelis–Menten formulation defines the specific growth rate at a given site in a river system.

$$\mu_A = (\mu_{A,20}\theta^{T-20})\left(\frac{\mathrm{NO_3}}{\mathrm{NO_3} + K_{\mathrm{NO_3}}}\right)\left(\frac{\mathrm{P}}{\mathrm{P} + K_{\mathrm{P}}}\right)\left(\frac{\mathrm{C}}{\mathrm{C} + K_{\mathrm{C}}}\right)\frac{1}{\lambda D_a}\ln\left(\frac{K_L + L}{K_L + Le^{-\lambda D_a}}\right) \tag{1-25}$$

where

μ_A = the specific growth rate of algal biomass (T^{-1})
$\mu_{A,20}$ = the maximum growth rate of algal biomass at 20°C (T^{-1})
θ = a temperature coefficient ranging from 1.02 to 1.06
T = the actual temperature in °C
$\mathrm{NO_3}$, P, C = the nitrate-nitrogen, orthophosphate-phosphorus and carbon concentrations (ML^{-3})
K_{C}, $K_{\mathrm{NO_3}}$, K_{P} = the temperature-dependent half-saturation coefficients (ML^{-3})
L = the light intensity (Langleys/T)
K_L = half-saturation coefficient for light (Langleys/T)
λ = the light extinction coefficient in the river (L^{-1})
D_a = the average stream depth (L)

Note that if any of the critical growth elements are 0, the algal growth rate μ is zero. Equation (1-24) couples algal production to the available supply of nutrients, light and temperature; hence algae and chlorophyll a will vary in time and space as the elements needed for growth change. Respiration is also temperature dependent

$$\rho_T = \rho_{20}\theta^{T-20} \tag{1-26}$$

where again T is the temperature in °C, θ is a constant, and ρ_{20} is the respiration rate (T^{-1}) at 20°C. All rate constants that are temperature dependent are defined by equations having the form of Eq. (1-26), where the constant θ is defined for each rate constant.

The mortality rate, $M(T^{-1})$ of algal biomass can be expressed as a linear function of the natural mortality rate, M_N, and the mortality caused by the toxicity of the water, βC,

$$M = M_N + \beta C \tag{1-27}$$

where β is a toxicity coefficient (rate of mortality per unit concentration, $(T^{-1}M^{-1}L^3)$ and C is the constituent or toxicity content (ML^{-3}) of the water in the river at location X.

The growth rate of zooplankton, μ_Z, can be defined by

$$\mu_Z = (\mu_{Z,20}\theta^{T-20})\left(\frac{A}{K_A + A}\right) \tag{1-28}$$

where K_A is the temperature-dependent half-saturation rate constant for algal biomass (ML^{-3}). The growth rate of zooplankton, $\mu_Z(T^{-1})$, times the zooplankton concentration, $Z(ML^{-3})$, times a conversion coefficient, $F_{A,Z}$, that indicates the mass of algal biomass required per unit mass of zooplankton, estimates the loss in algal biomass due to zooplankton grazing, G_Z. The reliability of current models for predicting trophic levels higher than phytoplankton (such as zooplankton and fish) is relatively poor;[20] hence the omission of these higher trophic levels from operational models such as QUAL II.

The nitrogen cycle in QUAL II is described by differential equations governing the transformation of nitrogen from one form to another. For ammonia-nitrogen, NH_3,

$$\frac{dNH_3}{dt} = \alpha_1 \rho A - \beta_1 NH_3 + \frac{\sigma_3}{A_X} \tag{1-29}$$

where NH_3 = the concentration of ammonia-nitrogen (ML^{-3})
A = algal biomass concentration (ML^{-3})
α_1 = fraction of respired algal biomass resolubilized as ammonia-nitrogen by bacteria
σ_3 = benthos source rate for $NH_3(MT^{-1}L^{-1})$
A_X = average stream cross-sectional area at location $X(L^2)$
β_1 = temperature-dependent rate of biological oxidation of $NH_3(T^{-1})$
ρ = temperature-dependent respiration rate (T^{-1})

For nitrite nitrogen, NO_2,

$$\frac{dNO_2}{dt} = \beta_1 NH_3 - \beta_2 NO_2 \tag{1-30}$$

where NO_2 = the concentration of nitrite-nitrogen (ML^{-3})
β_1 = the rate of oxidation of NH_3 to $NO_2(T^{-1})$
β_2 = the rate of oxidation of NO_2 to $NO_3(T^{-1})$

For nitrate nitrogen NO_3 concentration (ML^{-3})

$$\frac{d\mathrm{NO}_3}{dt} = \beta_2 \mathrm{NO}_2 - \alpha_1 \mu_A A \tag{1-31}$$

in which the parameters and variables are as defined above. These equations, (1-29), (1-30) and (1-31), together with Eq. (1-26), close the loop of the nitrogen cycle.

The phosphorus cycle is modeled in a less detailed manner than the nitrogen cycle. Only the interaction of phosphorus and algae, plus a sink term, are considered. Thus the differential equation describing the time-varying concentration of orthophosphate-phosphorus, $\mathrm{P}(ML^{-3})$, is written as

$$\frac{d\mathrm{P}}{dt} = \alpha_2 A(\rho - \mu_A) + \frac{\sigma_2}{A_X} \tag{1-32}$$

where α_2 = the fraction of algal biomass that is phosphorus
σ_2 = the benthos source rate for phosphorus $(MT^{-1}L^{-1})$

Carbonaceous biochemical oxygen demand, BOD^c, (ML^{-3}), is formulated as a first-order reaction

$$\frac{d\mathrm{BOD}^c}{dt} = -K^c\mathrm{BOD}^c - K_s\mathrm{BOD}^c \tag{1-33}$$

where K^c = temperature-dependent deoxygenation or decay rate of carbonaceous $\mathrm{BOD}(T^{-1})$
K_s = rate of loss of carbonaceous BOD due to settling (T^{-1})

The benthic oxygen demand, $\mathrm{BOD}^b(ML^{-3})$ is assumed to be a fixed demand, dependent on the cross-sectional area, A_X, at location X.

$$\frac{d\mathrm{BOD}^b}{dt} = K_b/A_X \tag{1-34}$$

where K_b is a constant benthic source rate $(MT^{-1}L^{-1})$.

The differential equation that describes the rate of change in dissolved oxygen is

$$\frac{d\mathrm{DO}_X}{dt} = K_a(\mathrm{DO}_X^S - \mathrm{DO}) + (\alpha_3\mu_A - \alpha_4\rho)A - K^c\mathrm{BOD}^c - K_b/A_X - \alpha_5\beta_1\mathrm{NH}_3 - \alpha_6\beta_2\mathrm{NO}_2 \tag{1-35}$$

where, in addition to the terms defined above,

DO_X^S = the temperature-dependent dissolved oxygen saturation concentration at location $X(ML^{-3})$
DO_X = the dissolved oxygen concentration at location $X(ML^{-3})$

K_a = the temperature-dependent reaeration rate constant (T^{-1})
α_3 = rate of oxygen production through photosynthesis per unit of algal biomass (MM^{-1})
α_4 = rate of oxygen uptake from respiration per unit of algal biomass (MM^{-1})
α_5 = rate of oxygen uptake per unit of oxidation of ammonia nitrogen (MM^{-1})
α_6 = rate of oxygen uptake per unit of nitrite nitrogen oxidation (MM^{-1})

For the most probable number (MPN) of coliforms, the rate of change with respect to time equals

$$\frac{dF}{dt} = -K_d F \tag{1-36}$$

where F is the coliform MPN and K_d is the die-off rate (T^{-1}).

The differential equation describing the rate of change in radio-nuclides, R, is

$$\frac{dR}{dt} = -\gamma_r R - \gamma_a R \tag{1-37}$$

where γ_r = radioactive decay rate (T^{-1})
γ_a = radioactive absorption rate (T^{-1})

Table 1-1 summarizes the complete set of equations numerically solved by QUAL II except the temperature relationships of the type defined by Eqs. (1-26). These equations include the effects of dispersion, advection, constituent reactions and interactions up through the phytoplankton trophic level, and a source term, $S_C(MT^{-1})$, that is assumed uniform over length ΔX of the river section at location X.

Note that if all left-hand terms of the equations listed in Table 1-1 are 0, this is a steady-state model. Both steady-state and time-varying models defined by these equations can be solved by numerical solution procedures that eliminate the need of simplifying the necessary differentials dC/dt and dC/dx for each constituent C. This permits a more rigorous treatment of the coupling effects among all constituents and the factors that characterize their aquatic environments.

Table 1-2 lists the input parameters defined above, the range of values of those parameters, whether or not they vary with location and/or temperature, and their relative reliability.[19] The relative reliabilities of various constituent concentration predictions provided by current state-of-the-art water quality models are listed in Table 1-3. Table 1-3 also summarizes the major impacts of various constituents in natural river systems.

Table 1-1 SUMMARY OF DIFFERENTIAL EQUATIONS SOLVED BY QUAL II SIMULATION MODEL[19]

Conservative mineral (C)	$\frac{\partial C}{\partial t} = \frac{\partial\left(A_X E \frac{\partial C}{\partial X}\right)}{A_X\,\partial X} - \frac{\partial(A_X U C)}{A_X\,\partial X} + \frac{S_C}{A_X\,\Delta X}$
Algae (A)	$\frac{\partial A}{\partial t} = \frac{\partial\left(A_X E \frac{\partial A}{\partial X}\right)}{A_X\partial X} - \frac{\partial(A_X U A)}{A_X\,\partial X} + \frac{S_A}{A_X\,\Delta X} + \left(\mu_A - \rho - \frac{\sigma_1}{D_a}\right)A$
Ammonia nitrogen (NH_3)	$\frac{\partial \mathrm{NH_3}}{\partial t} = \frac{\partial\left(A_X E \frac{\partial \mathrm{NH_3}}{\partial X}\right)}{A_X\,\partial X} - \frac{\partial(A_X U \mathrm{NH_3})}{A_X\,\partial X} + \frac{S_{\mathrm{NH_3}}}{A_X\,\Delta X} + \left(\alpha_1 \rho A - \beta_1 \mathrm{NH_3} + \frac{\sigma_3}{A_X}\right)$
Nitrite nitrogen (NO_2)	$\frac{\partial \mathrm{NO_2}}{\partial t} = \frac{\partial\left(A_X E \frac{\partial \mathrm{NO_2}}{\partial X}\right)}{A_X\,\partial X} - \frac{\partial(A_X U \mathrm{NO_2})}{A_X\,\partial X} + \frac{S_{\mathrm{NO_2}}}{A_X\,\Delta X} + (\beta_1 \mathrm{NH_3} - \beta_2 \mathrm{NO_2})$
Nitrate nitrogen (NO_3)	$\frac{\partial \mathrm{NO_3}}{\partial t} = \frac{\partial\left(A_X E \frac{\partial \mathrm{NO_3}}{\partial X}\right)}{A_X\,\partial X} - \frac{\partial(A_X U \mathrm{NO_3})}{A_X\,\partial X} + \frac{S_{\mathrm{NO_3}}}{A_X\,\Delta X} + (\beta_2 \mathrm{NO_2} - \alpha_1 \mu_A A)$
Phosphate phosphorus (P)	$\frac{\partial \mathrm{P}}{\partial t} = \frac{\partial\left(A_X E \frac{\partial \mathrm{P}}{\partial X}\right)}{A_X\,\partial X} - \frac{\partial(A_X U \mathrm{P})}{A_X\,\partial X} + \frac{S_{\mathrm{P}}}{A_X\,\Delta X} + \left[\alpha_2(\rho - \mu_A)A - \frac{\sigma_2}{A_X}\right]$

Biochemical oxygen demand (BOD^c)
$$\frac{\partial BOD^c}{\partial t} = \frac{\partial\left(A_X E \frac{\partial BOD^c}{\partial X}\right)}{A_X\, \partial X} - \frac{\partial(A_X U BOD^c)}{A_X\, \partial X} + \frac{S_{BOD^c}}{A_X\, \Delta X} - (K^c + K_s) BOD^c$$

Benthic oxygen demand (BOD^b)
$$\frac{\partial BOD^b}{\partial t} = \frac{\partial\left(A_X E \frac{\partial BOD^b}{\partial X}\right)}{A_X\, \partial X} - \frac{\partial(A_X U BOD^b)}{A_X\, \partial X} + \frac{K_b}{A_X}$$

Dissolved oxygen (DO)
$$\frac{\partial DO}{\partial t} = \frac{\partial\left(A_X E \frac{\partial DO}{\partial X}\right)}{A_X\, \partial X} - \frac{\partial(A_X U DO)}{A_X\, \partial X} + \frac{S_{DO}}{A_X\, \Delta X} + \left[K_a(DO_X^{SAT} - DO) + (\alpha_3 \mu_A - \alpha_4 \rho)A - K^c BOD^c - \frac{K_b}{A_X} - \alpha_5 \beta_1 NH_3 - \alpha_6 \beta_2 NO_2\right]$$

Coliform (F)
$$\frac{\partial F}{\partial t} = \frac{\partial\left(A_X E \frac{\partial F}{\partial X}\right)}{A_X\, \partial X} - \frac{\partial(A_X U F)}{A_X\, \partial X} + \frac{S_F}{A_X\, \Delta X} - K_d F$$

Radioactive material (R)
$$\frac{\partial R}{\partial t} = \frac{\partial\left(A_X E \frac{\partial R}{\partial X}\right)}{A_X\, \partial X} - \frac{\partial(A_X U R)}{A_X\, \partial X} + \frac{S_R}{A_X\, \Delta X} - \gamma_r R - \gamma_a R$$

Table 1-2 INPUT PARAMETERS FOR QUAL II[19]

Names in equation	Description	Units	Range of values	Variable by reach	Temperature dependent	Reliability
α_0	Ratio of chlorophyll *a*	$\frac{\mu g\ Chl\ a}{mg\ A}$	50–100	Yes	No	Fair
α_1	Fraction of algae biomass which is NO_3	$\frac{mg\ N}{mg\ A}$	0.08–0.09	No	No	Good
α_2	Fraction of algae biomass which is P	$\frac{mg\ P}{mg\ A}$	0.012–0.015	No	No	Good
α_3	O_2 production per unit of algae growth	$\frac{mg\ O}{mg\ A}$	1.4–1.8	No	No	Good
α_4	O_2 uptake per unit of algae respired	$\frac{mg\ O}{mg\ A}$	1.6–2.3	No	No	Fair
α_5	O_2 uptake per unit of NH_3 oxidation	$\frac{mg\ O}{mg\ N}$	3.0–4.0	No	No	Good
α_6	O_2 uptake per unit of NO_2 oxidation	$\frac{mg\ O}{mg\ N}$	1.0–1.14	No	No	Good
μ_{max}	Maximum specific growth rate of algae	$\frac{1}{day}$	1.0–3.0	No	Yes	Good
ρ	Algae respiration rate	$\frac{1}{day}$	0.05–0.5	No	Yes	Fair
β_1	Rate constant for biological oxidation of $NH_3 \rightarrow NO_2$	$\frac{1}{day}$	0.1–0.5	Yes	Yes	Fair
β_2	Rate constant for biological oxidation of $NO_2 \rightarrow NO_3$	$\frac{1}{day}$	0.5–2.0	Yes	Yes	Fair

σ_1	Local settling rate for algae	$\frac{\text{ft}}{\text{day}}$	0.5–6.0	Yes	No	Fair
σ_2	Benthos source rate for phosphorus	$\frac{\text{mg P}}{\text{day-ft}}$	*	Yes	No	Poor
σ_3	Benthos source rate for NH_3	$\frac{\text{mg N}}{\text{day-ft}}$	*	Yes	No	Poor
K^c	Carbonaceous BOD decay rate	$\frac{1}{\text{day}}$	0.1–2.0	Yes	Yes	Poor
K_a	Reaeration rate	$\frac{1}{\text{day}}$	0.0–100	Yes	Yes	Good
K_s	Carbonaceous BOD sink rate	$\frac{1}{\text{day}}$	*	Yes	No	Poor
K_b	Benthos source rate for BOD	$\frac{\text{mg}}{\text{day-ft}}$	*	Yes	No	Poor
K_d	Coliform die-off rate	$\frac{1}{\text{day}}$	0.5–4.0	Yes	Yes	Fair
γ_r	Radionuclide decay rate	$\frac{1}{\text{day}}$	*	No	No	Poor
γ_a	Radionuclide absorption rate	$\frac{1}{\text{day}}$	*	No	No	Poor
K_{NO_3}	Nitrate–nitrogen half-sat. constant for algae growth	$\frac{\text{mg}}{\text{l}}$	0.2–0.4	No	No	Fair to good
K_P	Phosphorus half-saturation constant for algae growth	$\frac{\text{mg}}{\text{l}}$	0.03–0.05	No	No	Fair to good
K_L	Light half-saturation constant for algae growth	$\frac{\text{Langleys}}{\text{day}}$	260	No	No	Good

* highly variable

Table 1-3 QUALITY IMPACTS AND CURRENT STATE OF MODELING[21]

Component	Quality problem use impacts	Modeling of streams	Modeling of estuaries
Transport, steady state		Good	Good
Transport, dynamic		Marginal	Moderate
Conservative substances	Water supplies	Good	Good
Total dissolved solids	Water supplies	Moderate	Good
Suspended solids	Water supplies Recreation	Marginal	Marginal
Bacteria	Water supplies Recreation	Moderate	Moderate
BOD-dissolved oxygen	Aquatic ecosystem	Good	Good
Simple industrial chemicals	Water supply Ecosystem	Moderate	Moderate
Simple metals	Water supply Ecosystem	Marginal	Marginal
Synthetic chemicals Complex metals	Water supply Ecosystem	Poor	Poor
Nutrients	Aquatic ecosystem Recreation	Moderate	Moderate
Eutrophication (algae)	Recreation	Marginal	Marginal
Zooplankton and fish	Recreation	Unsatisfactory	Unsatisfactory
Temperature	Aquatic ecosystem	Moderate	Marginal
Virus	Water supply Recreation	Unsatisfactory	Unsatisfactory
Floating substance	Recreation	Unsatisfactory	Unsatisfactory
Color and turbidity	Recreation Water supply	Unsatisfactory	Unsatisfactory

1-7 WATER QUALITY MANAGEMENT ALTERNATIVES

Thus far the discussion has focused on water quality predictive models of various levels of sophistication and complexity. These predictive models have been developed to assist in estimating the spatial and in some cases temporal distribution of various constituents in river systems, given various inputs of these constituents and given the river flows. These predictive models provide the core of any water quality management model that is designed to evaluate various alternatives for controlling river water quality.

Water quality management alternatives might include flow augmentation for the nonestuarine portions of a river system, wastewater treatment and diversions (bypass piping) at various point discharge sites, effluent storage lagoons for both nonpoint source runoff and for treated wastewater effluent, wastewater disposal on land, and instream artificial aeration. Various combinations of these alternatives may be appropriate for particular river systems. The questions to be answered include not only which of these management options to consider in a specific situation, but what should be their capacity, location, efficiency, etc., if they are implemented.

Each management alternative will have an effect on one or more water quality constituents, and each will involve a cost. One way to evaluate the advantages or disadvantages of each management alternative and to identify those that are more desirable than others for a given river system is to include these management alternatives as unknown variables within a water quality management model. Just how this might be done is briefly outlined below.

1-7.1 Low-Flow Augmentation

Low-flow augmentation models differ depending on how the streamflows associated with water quality standards are defined.[22] Low-flow augmentation models are usually developed to estimate upstream reservoir storage locations and capacities and reservoir release operating policies required to augment natural streamflows. Flow augmentation changes the streamflow at any location X along a river, and hence changes the flow Q and velocity U variables in the water quality prediction models. Since these variables are subject to some control, especially during low-flow conditions, it is necessary to solve for those augmented flows which, together with other management alternatives, best meet the water quality objectives.

Cost minimization is usually one (but not the only) objective of water quality planning. The cost incurred for low-flow augmentation includes not only any additional required reservoir storage capacity and operating costs, but also the loss incurred, if any, from diverting the water from other beneficial uses.

To determine the storage capacity and operating policy for reservoir releases needed at each reservoir site, a series of continuity and capacity constraints are required for various periods t of record. Continuity constraints equate the initial storage volume plus the inflow less the release and the evaporation and seepage losses to the final storage volume in any period t.

$$\begin{aligned}(\text{Initial storage volume})_t + (\text{inflow})_t - (\text{release})_t - (\text{losses})_t \\ = (\text{final storage volume})_t \qquad (1\text{-}38) \\ = (\text{initial storage volume})_{t+1}\end{aligned}$$

The probability distribution of releases determines the reliability of the augmented flow, and this reliability can be specified in advance, if appropriate.[22,23] Capacity constraints insure that the reservoir storage volumes in each period do not exceed the storage capacity.

$$(\text{Initial storage volume})_t \leqslant (\text{storage capacity}) \qquad (1\text{-}39)$$

Further details on the formulation and use of various types of reservoir design and operating models are found in numerous references.[24,25,26]

1-7.2 Wastewater Treatment

The concentration of any constituent C_i in wastewater produced at a site i can be reduced by various discrete unit treatment processes, k. The quantity of wastewater, QW_i^k, routed through each unit treatment process k will determine the cost and removal efficiency of the overall wastewater treatment facility. To illustrate how this overall efficiency and cost can be estimated when the wastewater flows are constant over time, assume there are k_i possible unit treatment operations at site i (e.g., grit chamber, primary and secondary sedimentation basins, activated sludge units, chlorination, etc.). The cost of each unit process is primarily a function of its hydraulic capcity QW_i^k. The problem is to find those hydraulic capacities of all unit processes k at each site i that, together with other management alternatives, will provide the desired water quality and will meet other water quality management objectives.

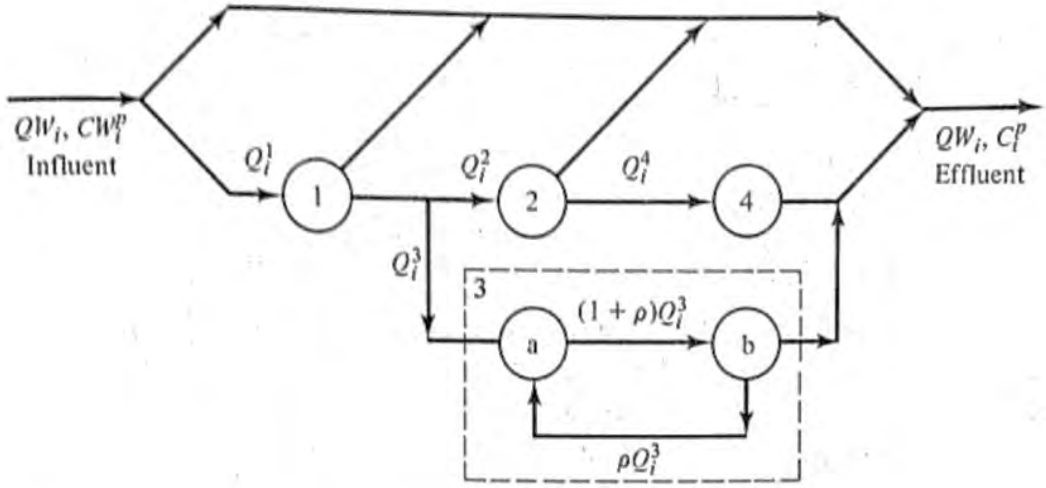
The concentration, C_i^p, of each constituent p in the effluent of a treatment facility depends on the quantity of wastewater influent routed through each unit treatment process and on the removal efficiency of that process. At hydraulic capacity, the removal efficiencies, f_k^p, of each process k are known for each constituent p. Mass balance equations for both flows QW_i^k, and constituent masses C_{ik}^p, are easily written for any proposed configuration of unit treatment processes. Such equations define the mass and concentration of each constituent in the effluent of the treatment facility. An example of how these mass balance equations can be written for unit processes in series and in parallel, with or without recycling, is illustrated in Fig. 1-1. In this diagram, process 3 might be considered an alternative to processes 2 and 4.

Each of the expressions in Fig. 1-1 is linear with respect to the unknown flows, QW_i^k, in each link of the treatment system. Since the final effluent flow equals the known influent flow, the effluent concentrations C_i^p of each constituent p can be determined from a knowledge of the unit process efficiencies and hydraulic flows.

1-7.3 Effluent Storage Lagoons

The ability to temporarily store treated wastewater effluent or nonpoint surface runoff when the assimilative capacity of the receiving land or water is low, and to release it when the assimilative capacity is high, may permit some reduction in the total cost of water quality management. To determine whether or not such an alternative is desirable, effluent storage lagoons can be modeled and included within the overall planning model.

Storage lagoons can achieve some removal of certain nonconservative constituents. Relationships between the removal of constituents and lagoon detention times are not always easy to obtain, and in any event are subject to considerable variation due to incomplete mixing and varying temperatures and volumes. Assuming first-order reaction rate constants K_l^p for each relevant



Process k	Influent flow	Effluent concentration C_{ik}^p
1	$Q_i^1 \leqslant QW_i$	$CW_i^p(1-f_1^p) = C_{i1}^p$
2	$Q_i^2 \leqslant Q_i^1 - Q_i^3$	$CW_i^p(1-f_1^p)(1-f_2^p) = C_{i2}^p$
3	$Q_i^3 \leqslant Q_i^1 - Q_i^2$	$CW_i^p(1-f_1^p)\left[\dfrac{(1-f_a^p)(1-f_b^p)}{1+\rho-\rho(1-f_a^p)(1-f_b^p)}\right] = C_{i3}^p$ †
4	$Q_i^4 \leqslant Q_i^2$	$CW_i^p(1-f_1^p)(1-f_2^p)(1-f_4^p) = C_{i4}^p$
System	QW_i	$[CW_i^p(QW_i - Q_i^1) + C_{i1}^p(Q_i^1 - Q_i^2 - Q_i^3) + C_{i2}^p(Q_i^2 - Q_i^4) + C_{i3}^p Q_i^3 + C_{i4}^p Q_i^4]/QW_i = C_i^p$

† Obtained by equating the recycled concentration to the effluent concentration.

FIGURE 1-1
Flows and pollutant concentrations throughout a potential wastewater treatment system.

constituent p, the change in mass of a constituent in a period t can be estimated using mass balance equations for each within-year period t.

$$S_{t+1}C_{t+1}^p - S_t C_t^p = Q_{\text{in},t}C_{\text{in},t}^p - Q_{\text{out},t}\left(\frac{C_{t+1}^p + C_t^p}{2}\right) - K_t^p\left(\frac{C_{t+1}^p + C_t^p}{2}\right)\left(\frac{S_{t+1} + S_t}{2}\right) \tag{1-40}$$

where S_t = lagoon storage volume at beginning of period $t(L^3)$
C_t^p = concentration of constituent p in the lagoon at beginning of period $t(ML^{-3})$
$Q_{\text{in},t}$ = influent to lagoon in period $t(L^3T^{-1})$
$C_{\text{in},t}^p$ = influent concentration of constituent p in period $t(ML^{-3})$
$Q_{\text{out},t}$ = effluent flow from the lagoon in period $t(L^3T^{-1})$
K_t^p = decay rate constant for constituent p in period $t(T^{-1})$

The left-hand side of Eq. (1-40) is the change in mass per period t. The initial storage volumes S_t are obtained from continuity equations similar to (1-38) and (1-39) above. Other equations may be needed to restrict the depth of the lagoon volume to prevent anaerobic conditions, and to insure certain minimum detention times. The attractiveness of storage lagoons is directly dependent on the variability of influent and/or the variability of the effluent released from the lagoon. Lagoon effluent variability is in turn dependent on the assimilative capacity of the receiving water body or land disposal site. Some methods for estimating land assimilative capacities, similar to the methods discussed in this paper for estimating water assimilative capacities, are found elsewhere.[27,28]

1-7.4 Artificial Instream Aeration

Instream aeration is another method of increasing the dissolved oxygen concentration of river systems. This is usually accomplished by injecting oxygen or air into water through a network of perforated pipes or by rotating devices that cause surface turbulence, thereby increasing the area over which oxygen transfer can occur. These methods may be particularly efficient for the temporary improvement of near anaerobic conditions, i.e., at sites where the dissolved oxygen deficits are relatively high, but they require energy and may cause excessive noise.

The oxygen transfer rate due to artificial aeration varies directly with the oxygen deficit D, with the water quality and temperature, and with the flow. Assuming the rate of oxygen transfer per unit of power input is constant over a period t, and that the aerators are operating at capacity, the quantity of oxygen transferred in period t at site i equals

$$\text{OR}_{ti} = k_{ti}(D_{ti}) \cdot (kA_i)$$

where OR_{ti} = oxygen transfer rate at site i in period $t(MT^{-1})$
$k_{ti}(\cdot)$ = rate of oxygen transfer per unit of power input as a function of oxygen deficit (L^2M^{-1})
D_{ti} = dissolved oxygen deficit concentration in period t at site i (ML^{-3})
kA_i = aerator capacity at site $i(LMT^{-1})$

The cost of aerators operated at capacity can be expressed as a function of their capacity, kA. These functions can account for the down time due to maintenance and repair. If the aerators are not operated at capacity, additional equations will be needed to account for both the capacity and the operating costs.[29]

1-8 CONCLUDING REMARKS

It would be ideal if one could say that water quality models such as those reviewed in this paper could be used directly to identify optimal, or at least improved, solutions to river water quality management problems. Perhaps they can in situations where there exist sufficient data and sufficient modeling expertise to develop, calibrate and verify models and their solutions and where the objectives of those responsible for water quality planning and management are clearly defined. But what about those situations where these ideal conditions do not exist? It is for these situations that many of the models, especially the simpler ones, outlined in this paper are most useful.

The true purpose of such models, at least in a planning and management context, is to develop insights into the behavior of the river system, to determine the significance or importance of having more accurate or more detailed data, and to assess the likely consequences of alternative policies that might be considered in an effort to improve or maintain river water quality. These insights can be used to guide the development of effective plans and decisions. Policy makers must try to understand not only *what* the best solution is, given a set of input data and assumptions, but more importantly *why* it is the best.

Water quality modeling, if done well, can give an understanding of why some management alternatives are better than others for a particular river basin. Modeling can provide one with estimates of how the river system will respond, at least in a relative sense, to different waste discharges. In addition, models can be used to help identify some preferred management plans given various management objectives and assumptions concerning future resource costs, technology, and social and legal requirements.

In acknowledging the role water quality models can and should play in the planning process, one must also recognize the inherent limitations of models as representatives of any real problem. The input data, including management objectives and assumptions concerning the physical, biological and chemical reactions that take place in the river, may be controversial or uncertain. Of course, the input affects the output. While the input data and model may be the best available, one's knowledge about the prototype (the actual river basin) and about how future events may alter its behavior will always be limited. In addition, since public water quality objectives change, water quality models must be viewed as flexible tools adaptable to changing circumstances as they are perceived and to changing data as they become available. Water quality models can, at best, provide information, not solutions.

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Readers familiar with water quality modeling literature will recognize that the writer has drawn upon the works of O'Connor, Thomann and DiToro, and Chen and Orlob in compiling this chapter. Their models, or modifications of them, have been used by analysts throughout much of the world. I am also indebted to numerous individuals at Cornell University who helped point out some problems on earlier drafts, but who bear no responsibility for any remaining errors or opinions.

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2
MODEL OF THE TRENT RIVER SYSTEM, ENGLAND

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2-1 THE TRENT BASIN

The river Trent (Fig. 2-1), the third longest river in the United Kingdom, is of importance in the national planning of water resources because it is one of the few major rivers flowing from the wetter west to the drier east side of the country. The main river is 274 km long and its catchment area is 10,450 km^2.

The river rises 11 km to the north of Stoke-on-Trent (Fig. 2-2) and flows in a southerly direction through that city until its confluence at Wychnor with the first tributary of importance—the river Tame. It then turns north-easterly flowing through Burton-on-Trent after which it is joined by the clean river Dove, the river Derwent from the north and the moderately polluted river Soar from the south. Downstream of Newark, at Cromwell Lock, it becomes tidal and eventually joins the Yorkshire Ouse to form the river Humber at Trent Falls.

Both the Trent and Tame have their sources in the Coal Measures and after 34 km and 27 km respectively pass from the Coal Measures to the Keuper Marl. The Tame catchment contains outcrops of the country's second most important aquifer, the Bunter Sandstone, but upstream of Water Orton, in the Birmingham and industrial West Midlands areas, the man-made impermeable area is estimated to be 20 percent of the total catchment area. This estimate is based on a population of two million and 42 m^2 of impermeable area per person.[2]

FIGURE 2-1
The River Trent.

The rivers Dove and Derwent have their sources in the Millstone Grit of the Peak District but also drain a central area of Carboniferous Limestone before flowing on to the Keuper Marl of the Trent Valley in the Ashbourne and Derby area. The rivers Soar and Anker drain the Keuper Marl of Leicestershire and the Coal Measures of Warwickshire respectively.

As well as the outcrops of the Bunter Sandstone aquifer in the West Midlands and Birmingham areas mentioned earlier, there are substantial outcrops also in Nottinghamshire, running north to south. Further east there are also outcrops of Magnesian Limestone.

The average flow of the river Trent at Wychnor immediately upstream with its confluence with the river Tame for the ten year period 1960–1969 was

FIGURE 2-2
The Trent Basin.

1,123 Ml/d (megalitres per day) while the river Tame had an average flow of 1,668 Ml/d of which 364 Ml/d was due to the discharge of effluents derived from the importation of water from the west into south Staffordshire and Birmingham. The influx of the river Dove increases the average flow of the river Trent by 1,373 Ml/d* and the Derwent by 6,123 Ml/d. The river Soar contributes some 582 Ml/d so that at Trent Bridge, Nottingham, the average flow of the river is 7,274 Ml/d. The flow at Trent Bridge varies from a low summer flow of 1,592 Ml/d to a winter average of 13,638 Ml/d.

2-2 SEWAGE AND INDUSTRIAL EFFLUENT DISPOSAL

The daily volumes of sewage and industrial effluent discharged to the river Trent in 1965 are given in Table 2-1, together with estimated discharges for 1912 and 2001.[3]

Table 2-1 EFFLUENTS DISCHARGED TO THE RIVER TRENT SYSTEM[4]

Year	Sewage Ml/d	Industrial† Ml/d
1912 estimated	341	59
(2.4 percent compound decrease)		
1965 (actual)*	1,200	205
2001 estimated	2,819	482
(2.4 percent compound increase)[3]		

* 232 Ml/d was discharged below Trent Bridge.
† Excluding cooling water and mine water.

While the figures estimated for 1912 for the discharge of sewage effluent have been corroborated from the records of the City of Birmingham Water Department, those for industrial effluent discharged, although calculated in the same way, must be much more speculative because of their erratic growth and much industrial effluent that was previously discharged directly to the river is now passed into the foul sewerage system.

The proportion of sewage and industrial effluent at Trent Bridge in the past, present and as forecast in the future under average and dry weather flows is shown in Table 2-2.

Table 2-2 RATIO OF RIVER FLOW TO EFFLUENT AT TRENT BRIDGE, NOTTINGHAM[4]

	1912	1965	2001
Average river flow	16.6:1	5.1:1	2.3:1
Dry weather flow	7.5:1	2.5:1	1.6:1

* Average based on 1962–1969 flow figures.

In this table, the dry weather flow figures for sewage in Table 2-1 were converted to average figures by multiplying by a factor of 1.25 for the first line. Also the ten year average and dry weather flows were used for 1965 and adjusted for 1912 and 2001 by subtracting and adding respectively for imported water and estimated growth of discharges of effluents.

Some 40 percent by volume of sewage effluent discharged to the river system originates from industrial effluents discharged to foul sewers and because of their importance and diversity, a description of the major categories and their location at the time of the investigation follows.

In Stoke-on-Trent and the Potteries, the principal discharge to both sewer and river was of settled potters' slip—a suspension of clay, but in addition there were typical discharges from the steel and engineering industry. From the vicinity of Stafford the major discharge was from a salt works (since closed down) and from Birmingham and the industrial West Midlands, effluents from the metal finishing industry predominated although there were discharges from almost every other type of industry. In Burton-on-Trent the major effluent was from the brewing industry while in the East Midlands other organic-type effluents had their origin in sugar-beet factories. The greatest quantity of effluent in the East Midlands was, however, due to the textile dyeing and finishing industries and there were also significant discharges from the chemical and pharmaceutical industries.

The coal fields within the Trent basin were among the most productive in the country, producing about 52 million tonnes or just under 30 percent of the national output. This gave rise to about 91 Ml/d of minewater and coal washing plant effluents discharge within the basin, their salinity increasing significantly the total dissolved solids content of many of the streams to which they were discharged.

The combination of the proximity of industry, the availability of coal locally and cooling water from the river, the geographical location being midway between north and south England, were the reasons for the installation of electricity generating stations along the river Trent. There is a station every 13 km and in 1970 the installed capacity was 15,000 MW or about one-third of the national generating capacity. The earlier, smaller stations of up to 500 MW are cooled either by circulating river water or use of this method in combination with cooling towers. The more modern stations of 2,000 MW capacity, however, depend entirely on cooling towers. The former increase the sensible heat of the river water, raising the temperature by about 5°C though in the middle Trent at Shardlow, temperatures of up to 30°C are recorded. The latter evaporate up to 386 Ml/d of pure water, the soluble solids being discharged to the river in purging operations.

2-3 AMENITY USES OF THE RIVER SYSTEM

Notwithstanding the extensive use made of the river system by industry for abstracting water for process and cooling purposes and as a vehicle for the disposal of waste, it is also widely used for recreation and amenity purposes. Fishing is the most popular sport and about 175,000 anglers are licensed each year. These anglers, together with unlicensed anglers under sixteen years old, probably visit the main river and its tributaries about 5 million times a year. Coarse fisheries predominate in the basin, although there are excellent trout fisheries in the Dove, the Derwent, and some smaller streams such as the Wye, Lathkil Manifold, Blithe, and Sence. Water supply reservoirs are also stocked with trout.

Apart from the 34 km of the river Trent below Stoke-on-Trent and the 18 km between the Tame and Dove confluences, the Trent itself is a good coarse fishery. The other tributaries have fisheries whose value is proportional to their size but the Tame, Erewash, and Fowlea Brook and Bottesford Beck are wholly or mostly fishless. There are other smaller lengths of some other tributaries which are also fishless.

There is also an extensive canal system in the basin of some 688 km and while, generally, canals in the Birmingham area do not support fish life, most of the remaining canals are reasonable fisheries.

There is an increasing use of the Trent and any other suitable water by power and sailing boats, by water skiers and by people who just want to park their cars, walk or picnic by water.

2-4 WATER QUALITY

The map of the river system (Fig. 2-2) shows that most of the centres of population are situated by the headwaters of the Trent or one of the tributaries. The inhabitants of the catchment number over 5 million, of whom some 2.1 million live in the industrial West Midlands area whose centre is Birmingham. Consequently sewage, industrial effluent and storm-water runoff generated by these conurbations pass through a large part of the main river. Only two of the larger tributaries, the Dove and Derwent, are unaffected in this way. Table 2-3 shows in detail the quality of the water of the Trent and its tributaries, 1968–1970, while Table 2-4 summarizes the yearly changes in Amm N, BOD, DO and chloride at Trent Bridge, Nottingham together with the relevant flows.

Although it is not possible to go into great detail about the quality of the water of the Trent river system, there was a steady improvement in both the quality and appearance of the river water from the early 1960s. This was in the main due to the building of new sewage treatment works, the extension of existing works and sewerage systems, the installation of effluent treatment plants by industry before discharge to the river, to the sewerage system and to soakaways, the diversion of industrial effluent to the sewers and lastly, but by no means least, the reduction in the carbonization of coal for the production of gas, as the production of North Sea gas and the gassification of oil increased.

Table 2-3 AVERAGE ANALYTICAL RESULTS FOR THE TRENT AND ITS TRIBUTARIES 1968–1970[6]

Point on river system	No. of samples	Permanganate value (4-hr) (range) mg/l	BOD (5-day) (range) mg/l	Ammoniacal nitrogen as N (range) mg/l	Nitrate as N (range) mg/l	Chloride as Cl (range) mg/l	Dissolved oxygen (range) mg/l	Temperature °C (range)
Trent above Tame confluence	40	6.1	5.6	0.7	6.2	187	7.6	12.0
at Wychnor		(2.6–18.4)	(1.2–16.2)	(Nil–2.6)	(3.5–8.8)	(63–360)	(3.4–11.2)	(3.0–23.0)
Tame above Trent confluence	148	11.7	17.5	6.5	7.7	86	3.9	12.3
at Chetwynd		(6.0–31.0)	(6.0–77.0)	(1.4–13.2)	(0.1–13.3)	(47–365)	(Nil–10.5)	(2.0–24.5)
Trent below Tame confluence	272	9.4	13.4	3.9	7.1	128	5.1	11.8
at Walton		(3.0–23.0)	(4.0–52.0)	(0.4–9.5)	(0.3–15.2)	(58–304)	(0.1–10.4)	(0.0–25.0)
Dove above Trent confluence	196	4.5	3.3	0.3	3.0	32	10.2	10.2
at Monksbridge		(1.0–16.2)	(1.0–14.0)	(Nil–1.6)	(1.7–4.9)	(18–81)	(6.3–15.0)	(0.0–22.1)
Derwent above Trent confluence	310	4.1	5.0	0.9	3.2	62	7.7	13.8
at Wilne		(1.8–19.0)	(2.0–15.0)	(0.2–3.1)	(2.0–4.6)	(28–156)	(2.7–11.9)	(4.0–24.5)
Soar above Trent confluence	251	6.0	7.4	3.5	7.1	81	8.9	11.5
at Kegworth		(3.0–13.0)	(1.8–21.3)	(0.3–11.0)	(4.2–13.5)	(29–152)	(5.1–12.2)	(1.5–24.0)
Erewash at Barton Ferry	151	9.4	14.0	4.9	8.2	231	7.1	11.9
		(4.0–44.4)	(2.0–34.0)	(0.5–12.9)	(1.6–17.3)	(81–494)	(1.5–20.0)	(1.5–25.0)
Trent at Trent Bridge,	333	6.3	8.6	1.1	6.8	97	7.5	15.3
(Nottingham)		(3.0–14.0)	(3.4–22.0)	(0.2–3.9)	(3.1–11.3)	(40–162)	(4.2–10.9)	(5.0–27.0)

Table 2-4 QUALITY OF RIVER TRENT AT TRENT BRIDGE, NOTTINGHAM[4]

Year	No. of samples	Ammoniacal nitrogen (mg/l) Average (range)	BOD (mg/l) Average (range)	Dissolved oxygen (mg/l) Average (range)	Chlorides (mg/l) Average (range)	Average flow m^3/s mgd
1962–1964	190	3.5	13.1	6.7	101	64.2
		(0.9–9.3)	(3.3–34.3)	(2.6–10.6)	(39–167)	(1,220)
1965	49	3.0	11.6	7.2	96	101.3
		(0.8–6.2)	(4.4–28.8)	(3.8–14.8)	(42–154)	(1,925)
1966	31	2.0	10.7	7.4	83	119.4
		(0.6–4.6)	(4.8–20.4)	(5.0–11.2)	(44–143)	(2,270)
1967	47	1.7	9.6	7.0	104	83.6
		(0.6–5.7)	(5.1–25.2)	(4.6–10.6)	(53–195)	(1,590)
1968	46	1.4	9.1	7.6	89	89.4
		(0.4–3.9)	(3.8–22.5)	(4.5–10.9)	(53–147)	(1,700)
1969	108	0.9	7.9	7.0	103	96.0
		(0.2–2.8)	(4.2–19.0)	(4.7–10.8)	(40–162)	(1,824)
1970	155	1.1	8.8	7.8	101	81.5
		(0.2–3.6)	(3.4–19.5)	(4.2–10.8)	(43–150)	(1,555)

2-5 WATER RESOURCES

Because of the pollution it receives, the Trent river itself is not used as a source of water for potable supply purposes, but many of its tributaries are, either by direct abstraction or through the construction of impounding reservoirs. Table 2-5 shows the major sources of water in the Trent system which are used for potable purposes.

In June 1968 the Trent River Authority published "Water Resources: a preliminary study"[5] in which the forecast demands for water and a possible allocation of potential resources to meet these forecast demands, was made. This is summarized in Table 2-6. The figures are, however, subject to periodical revision and while the pattern of demand remains constant, the actual figures vary. Thus the 1970 figure for the estimated quantity of water required for potable purposes has fallen to 614 Ml/d from the 968 Ml/d quoted in the table.

The study also concluded that the direct abstraction of water by industry could be expected to increase from 600 Ml/d to 1,709 Ml/d by the year 2000; that the Central Electricity Generating Board's demand for cooling water which would be lost by evaporation would increase from 386 Ml/d to 909 Ml/d and that the demand for water for irrigation purposes was estimated to increase from 164 Ml/d to 477 Ml/d.

The report concluded that:

> . . . provided demands in the western area of the catchment can be met by importing water from further west and provided that the quality of the river Trent can be improved to acceptable standards for its use to augment public water supply, the resources of the basin are sufficient to meet all future requirements for water to the end of this century and beyond.

Table 2-5 MAJOR SOURCES OF POTABLE WATER SUPPLY FROM THE TRENT SYSTEM[4]

River	Source	Authority	Source yield m^3/s
Blithe	Blithfield Reservoir	South Staffs. Waterworks Company	0.79
Churnet	Tittesworth Reservoir	Staffs. Potteries Water Board	0.39
Dove	Abstraction/Staunton Harold Reservoir and Foremarke Bottom	River Dove Water Board	2.00
Derwent	Derwent Valley reservoirs	Derwent Valley Water Board	2.58
Amber	Ogston Reservoir (including pumped storage from river Derwent)	North Derbyshire Water Board	0.51
Derwent	Direct abstraction/Little Eaton	South Derbyshire Water Board	0.49
Derwent	Abstraction/Wilne Reservoir (under construction)	City of Nottingham Water Department	0.79
Bourne	Abstraction/Shustoke Reservoir	City of Birmingham Water Department	0.28
Blythe	Abstraction/Shustoke Reservoir	City of Birmingham Water Department	

Table 2-6 FORECAST DEMAND AND POSSIBLE RESOURCE ALLOCATION[4]

		Resources			
Year	Demand m^3/s	Groundwater m^3/s	Dove/Derwent m^3/s	Local surface m^3/s	Trent m^3/s
1975	2.42	0.63	1.32	0.05	0.42
1985	8.47	0.79	3.68	0.16	3.84
2000	17.8	0.95	5.16	0.47	11.21

The River Authority decided therefore that the logical course of action was to develop the resources of the clean tributaries Dove and Derwent as far as possible and thereafter to utilize the water of the Trent itself. The research programme was commissioned to investigate the use of river purification lakes to alleviate the pollution caused by storm runoff, to investigate the recharge of the Bunter Sandstone by river water in the times of high river flows and to assess the possible exploitation of the river gravels as a source of water. It was also proposed to construct a pilot water treatment plant to determine any special treatments that would still be needed after the quality of the middle and lower Trent had been improved to the "acceptable standard" mentioned in the Preliminary Study.

2-6 THE TRENT RESEARCH PROGRAMME

Discussions were also held about these proposals with the Ministry of Housing and Local Government and, at their suggestion the Water Resources Board. They included the possibility of raising all effluent standards to the 30:20 norm and in some cases to a higher quality in the future. The Water Resources Board thought, however, that nothing less than a comprehensive study of the economics of the development of the resources of the Trent river system was required. Thus the Trent Research Programme was born.

Taking into account the amount and expected growth of population and industry in the upper and middle reaches of the catchment, the objectives of the Research Programme were stated to be:[6]

(1) To determine the different ways in which the river Trent, its tributaries and other waters in, or which could be brought into the area, could be used to satisfy the expected demands in that area or elsewhere, for water for domestic, industrial, agricultural and amenity use, of a quality suitable for each.
(2) To evaluate the costs and benefits of each of these ways as a guide to determining the most efficient solution.

The Research Programme which was concerned with both practicability and cost was, for efficiency, broken down into a number of projects, each of which was tackled by a team of experts, the activities of whom were coordinated by the Water Resources Board. The Programme was guided by a Steering Committee chaired by the Water Resources Board and on which were representatives from the Ministry of Housing and Local Government (now the Department of the Environment), Water Pollution Research Laboratory (now Water Research Centre, Stevenage Laboratory) and the Trent River Authority (now Severn–Trent Water Authority).

These projects were:

1 The estimation of demand for water in, and to be supplied from, the Trent area.[7]

2 The assessment of river flow augmentation and of the water available for abstraction.[7]

3 The cost of waste water treatment.[8]

4 The effects of pollution on river water quality.[9]

5 The costs of river water treatment at the present and future specified improved qualities.[10]

6 The investigation and assessment of the benefits of river purification lakes on the river Tame.[11]

7 Investigation of the cost and practicability of recharging:

a The Bunter Sandstone.[12]

b The river gravels.[13]

8 Investigation into the use of non-potable standard water for industry.[14]

9 Investigation of the demand for and the benefits of water-based recreation.[15]

10 The construction of an economic model.[16]

11 The evaluation of the economic model.[17]

From the references, full details of the projects may be obtained; here only the important aspects of each will be highlighted.

2-6.1 Future Demands for Water

In their study of water resources in Wales and the Midlands[18] the Water Resources Board brought up to date the assessment of the expected demand for water made by the Trent River Authority in 1968.[5] These figures are given in Table 2-7.

In 1967, the total demand for water on the twelve authorities whose areas cover most of the Trent basin amounted to 1,436 Ml/d. This figure is equivalent to a daily per capita consumption of 240 litres. By the end of the century it is estimated that the equivalent figures will be 3,912 Ml/d which is equivalent to 455 litres per head per day. These figures represent an estimated rise in demand of about 3 percent per annum compound. Of these demands, some 37 percent are to meet the estimated requirements of industry for metered supplies of potable water from the water undertakings.

More than 80 percent of the water abstracted directly by industry is used for cooling purposes and is frequently returned to the same source almost immediately virtually undiminished in quantity, but perhaps with some deterior-

Table 2-7 CURRENT AND FUTURE DEMANDS FOR WATER[18]

Use	Ml/d			
	Resources	Demand 1967	Estimated Demand 1981	Estimated Demand 2001
Public water supply	1923	1436	2342	3912
Water mills	2070	2070	2070	2070
CEGB*				
direct cooling	9380	7464	3498	1816
evaporation	370	208	1045	1818
Other industry				
cooling	950	600	826	1452
other	520	322	445	810
Agriculture				
normal	69	69	69	69
spray irrigation	26	26	54	82

* CEGB = Central Electricity Generating Board

ation in quality. The major user of cooling water is the Central Electricity Generating Board but it is expected that their demand for water for direct cooling will diminish from over 7,450 Ml/d to about 1,820 Ml/d by the year 2000 because of the installation of additional cooling towers for evaporative cooling purposes. The evaporative losses are however expected to increase substantially—from about 208 Ml/d to 1,820 Ml/d by the end of the century and about one-third of this will be lost from the basin altogether. New evaporative power generating stations will probably be sited in the middle or lower reaches of the Trent and therefore will be unlikely to create a water resource problem.

Of the residual industrial demand, no less than 2,070 Ml/d is for motivating water wheels in water mills and while this figure is unlikely to increase—in fact it is expected that it will decrease as more mills turn to electricity for power—the increased use of evaporative cooling could lead to an increase in net demand. Other demands, amounting to some 322 Ml/d have, like the demand for industrial cooling water (other than CEGB), been estimated to increase at the rate of 3 percent per annum compound until the end of the century.

The demand for water for agricultural purposes is, by comparison, small—some 70 Ml/d and is not expected to increase significantly. While the potential demand for spray irrigation purposes is high, it is likely to be limited by economic factors and met by the provision of local storage.

2-6.2 River Flow Augmentation

This investigation examined various ways of obtaining additional supplies from the rivers within the basin. Of these the best quality water was in the rivers Dove and Derwent and the possibility of augmenting their flows was therefore examined in detail.

Six methods of improving the yield obtainable from the rivers were postulated:

1 by constructing new regulating reservoirs in the upper catchments;
2 by increasing the capacity of existing reservoirs;
3 by converting existing direct supply reservoirs to regulating reservoirs;
4 by constructing pumped storage reservoirs in the lower part of the catchments;
5 by constructing tidal type sluices upstream of their confluence with the Trent and
6 by redeploying the compensation water from the existing direct supply reservoirs.

The results of the investigation were reported in full by the Trent River Authority[19] but for the economic model the Authority provided the following cost and yield information for the conjunctive development of the rivers Dove and Derwent:

1 The cost of constructing an upland reservoir at Brund in the Dove catchment with an estimated yield of 210 Ml/d.
2 The cost of constructing a pumped storage reservoir at Hassop in the Derwent catchment with an estimated yield of 130 Ml/d.
3 The cost of constructing a pumped storage reservoir at Carsington in the Dove catchment with an estimated yield of 180 Ml/d. This would be a second development.
4 The costs of constructing a number of possible pumped reservoir sites in the vicinity of Derby which could be filled from either the Dove or the Derwent.

2-6.3 Costs of Wastewater Treatment

The costs of treating sewage to the standards actually being achieved in 1969 (ET1) were assessed and estimates made of the costs of plant designed to achieve the following standards which, it was specified, should be met 95 percent of the time:

1 A 30:20 effluent standard (30 mg/l suspended solids, 20 mg/l BOD) (ET2).
2 As in (1) above but fully nitrified (ET3).
3 A 10:10 fully nitrified effluent (ET4).

The cost estimates were based on a detailed investigation of the facilities (and their performance) at all the major sewage works. These accounted for about 80 percent by volume of the total effluent discharged to the system. Where actual costs were known, they were used, but where they were not available estimates were derived from various sources on the unit costs of the individual treatment processes. All costs were adjusted to 1967 prices and it was assumed that plant would be replaced by a similar unit at the end of its useful life, which was defined to be 30 years. Results were expressed in both the total capital investment necessary to achieve the specified effluent qualities and the total annual cost which included operating costs. The capital investment necessary to treat to ET2 standard was estimated to be £79 million in 1971 and £132 million in 2001. These figures included £55 million for plant already in situ. The equivalent annual costs, including operating costs were estimated to be £9.8 million and £16.2 million respectively. The estimates for ET3 and ET4 standards were some 15 percent and 25 percent greater respectively.

A similar approach was used to estimate the costs of treating industrial effluents which were discharged directly to the river system, the standards being approximately the same as those for sewage effluents. Treatment to ET4 standard was estimated to cost £3.6 million per annum in 1971 and that this figure would rise to £6.7 million per annum in 2001.

2-6.4 The Effects of Pollution on River Water Quality

Quality data supplied by the Trent River Authority supplemented by intensive surveys carried out by the Water Pollution Research Laboratory were used to estimate relationships between the polluting loads released to the river system and the concentrations at specified points throughout the river system of the sixteen quality determinands considered to be of most significance. These relationships were then used to estimate the quality distributions that would occur at specified river flows at various times until the year 2001 for the various effluent treatment standards described above. Relationships were also developed between water quality and the types of fishery to be expected.

These relationships formed the basis of the water quality simulation model which was part of the economic model. The formulation of the model necessarily involved the adoption of certain simplifying assumptions, the two principal ones being:

1 that the system would behave as though the flows and polluting loads were maintained at their arithmetic mean value;
2 that the rate of decay of non-conservative determinands would remain constant at the values derived from the analysis of data for 1969.

2-6.5 Costs of River Water Treatment

The feasibility of treating river Trent water to potable standards was examined through pilot plant experiments between 1968–1971 and the costs of so doing were assessed. From this work the following conclusions were drawn:

1 In view of the pollution of the river water, conventional treatment, i.e., flocculation, sedimentation, filtration, and disinfection would not be capable of producing a water of potable standard.

2 The recommended sequence of treatment to produce a potable, though not necessarily wholesome water was: storage, biological sedimentation for ammonia removal, coagulation and softening, sedimentation and filtration, pH correction, granular activated carbon treatment, and disinfection by chlorine.

3 The total costs (including operating costs) at 1967 prices were estimated to range from 2.2 pence per cubic meter to 1.8 pence cubic meter for plants varying in capacity from 21.5 Ml/d to 215 Ml/d.

For use in the model, eight treatment sequences were specified for use with different raw water qualities. They included conventional treatment alone and with various combinations of ammonia removal, treatment by the use of activated carbon and desalination. The total annual costs for these various combinations at 1967 prices varied from 1.4 to 4.5 pence per cubic meter.

2-6.6 River Purification Lakes

After a storm over the largely impermeable Birmingham area, the first flush of storm water is frequently extremely polluted and to protect the lower Tame and Trent from the wave of pollution the idea of diverting the river itself through retention lakes was proposed. These would allow a large proportion of the polluted suspended material to settle and, particularly in times of normal or low flow, would allow some of the biodegradable organic matter to be oxidized.

A small pilot plant treating some 11 Ml/d was constructed in 1967 at Elford on the Tame to test the proposal and was kept operational until 1970. Such was the improvement in quality from five days' retention that caged carp, which quickly died in the influent, survived for long periods in the effluent from the plant. Because of the encouraging results obtained, a larger plant was commissioned at Lea Marston in 1969. The plant, which was designed to treat 4.5 Ml/d passed abstracted river water through retention tanks with a residence time of $1\frac{1}{2}$ hours before being pumped to a worked out gravel pit which provided five days' retention. The chemical and biological changes which occurred were monitored at various stages through the system.

The observed effects of passing river water through the system were, in general terms, that the river water quality was improved by about 25 percent through the sedimentation tanks and a further 25 percent improvement was obtained in the retention lakes. Again, caged carp survived in the effluent from the lake before it rejoined the river. It also provided an effective buffer against pollution caused by storms.

The main conclusions drawn were threefold:

1 A purification lake system on the river Tame would be an effective way of protecting the river from storm pollution arising in the industrial West Midlands.

2 The principal problem associated with the system would be the removal, treatment and disposal of the sludge.

3 The annual running cost of a full scheme at 1967 prices would be of the order of £0.5 million.

2-6.7 Artificial Recharge

During periods of high flow, water from the Trent could be abstracted to recharge either the Bunter Sandstone aquifer from which water was already abstracted for public supply, or the extensive but shallow gravel beds close by the river. Three benefits would result:

1 An increased yield from the aquifer.

2 An improvement in quality of Trent water due to its passage through the sandstone or gravels.

3 The possibility of overcoming objections to the direct abstraction for potable purposes of water which contained large proportions of effluents.

Experiments were carried out in recharge basins in the Bunter Sandstone using water from the river Maun whose quality was somewhat poorer than the Trent at Nottingham. After settlement and aeration, recharge through a sand filter covering the floor of the basin was carried out at rates of up to 4.5 Ml/d. The average recharge rate was 0.35 m/d which implied a gross land requirement of 1 km^2 per 100 Ml/d.

Analyses of samples obtained from beneath the floor of the basin showed a substantial improvement in water quality. Suspended matter, including bacteria and viruses and ammonia were removed and a large percentage of the organic compounds was either absorbed or decomposed biologically. It was considered that there would be little danger of adversely affecting the quality of the groundwater outside the immediate locality of any prototype scheme. Contaminants introduced into the aquifer would mainly consist of inorganic salts at levels within the limits for potable purposes.

For use in the model, costs of a scheme yielding 290 Ml/d at a site 16 km from the river Trent were estimated. At full utilization and at 1967 prices, the capital costs were equivalent to £32,000 per Ml/d; at $8\frac{1}{2}$ percent interest unit costs were equivalent to 2 pence per cubic meter. Irrigation experiments were also carried out which produced even more improvement (in quality and significant increases) in vegetational yield, but the recharge rates achieved were less than half the results obtained using a recharge basin.

The gravel recharge experiments were less successful due to the variation in thickness of permeability. Nevertheless, treated water could be obtained on sites at a 1967 cost of about 2.4 pence per cubic meter from these sites each of which could produce about 45 Ml/d. It was considered that these yields, while being valuable as sources for local supplies, had limited application on a regional basis.

2-6.8 Dual Supply Systems

A dual supply system is one in which river water, treated to a lower standard than for potable supply is distributed to industrial premises in parallel with the potable system. For its success, it depends on the willingness of industrialists to accept the lower grade water in substitution for potable supply, or to meet additional future demands. The former case has the advantage that it releases water to meet additional domestic demands.

From the results of about 1,000 questionnaires it was found that about 300 Ml/d of existing potable supplies could be replaced by lower grade water if it were available at an attractive price. Further, depending on the degree of substitution that could be achieved, non-potable supplies of up to double this quantity could be introduced by the turn of the century.

A desk study was carried out and seven possible schemes designed in outline and costed. Although these provided no alternative to the development of new resources, there appeared to be potential for introducing substantial quantities of non-potable water in the Birmingham and South Staffordshire areas. Nottingham, Derby and perhaps Stoke were also possible candidates for dual supply systems.

2-6.9 Recreation Benefits

Because angling was the major amenity use made of the river system and because of the obvious importance to angling of river water quality, studies of the recreation benefits to be obtained from the river system were confined to angling. The benefit was effectively defined to be the willingness to pay for the pleasure of fishing. The area surrounding each fishery, in practice a stretch of river, was divided into zones and the anglers classified according to their zone of residence. The method used defined a demand curve which related the number of visits per year with cost and in this way the calculations could take account not only of entrance and other fees but also of travel costs which included the value of time spent traveling.

For the model's purposes lengths of rivers where there were fisheries were divided into stretches of about 16 km. A survey of anglers was carried out in 1969 which provided 1,600 fully completed questionnaires which represented about 1 percent of the angling license holders. Results were therefore multiplied by 100 to arrive at an estimate of the benefit.

Two equations were derived from these data based on alternative values allotted to "time" of 12.5 and 25 pence per hour. Both of these figures were, however, lower than the value of 36 pence per hour used by the government in similar surveys and this was the figure used in the model. The angling benefits calculated and adjusted to 1967 prices varied from nil to over £8,000 per mile per annum. Differences were due to factors such as the number of angling places per mile, the position of the fishery relative to population centres, the quality of the fishery and the availability of alternative attractions.

To allow for other forms of recreation, e.g., sailing and power boating, it was suggested that 30 percent should be added to the fisheries benefit.

2-6.10 Alternatives to the Conventional Development of Trent Basin Resources

Several alternatives to the development of conventional indigenous sources were considered. They included the direct importation of water from the West and North, desalination of sea and brackish waters and of certain intractable effluents and the possibility of removing some effluents from the river system altogether.

Water is, of course, imported into the Trent system indirectly via the public water supply to Birmingham and its environs and the sewerage system and sewage works of that area. A desk study estimated the costs of delivering clean water to augment the flow in the upper reaches of the Trent and Churnet from the rivers Severn and Dee and also from a Dee estuary conservation scheme. Similarly, the cost of importing water from a Morecambe Bay scheme was examined but found to be more expensive than that from the Dee estuary. Additionally, the cost of water abstracted at Upton on Severn and piped to Coventry and Leicester was estimated.

The desalination of sea water for use as both a base load supply and in conjunction with conventional sources was examined as was the desalination of the limited quantities of brackish water in the basin.

The cost of piping some 40 Ml/d of effluent to the coast for direct discharge into the sea was estimated.

None of these possibilities had costs which were competitive with the other alternatives considered and they were not therefore considered in the model.

2-7 THE MODEL

The principal objective of the model was to assess the costs of alternative strategies for meeting the additional demands on the water resources of the Trent basin. Of course, account was taken of specified minimum river water quality standards and other constraints. From these considerations the least cost solution could be identified.

The "least cost solution" was defined as the minimum total annual cash flow at the year 2001 and was the sum of the annual capital costs at a chosen interest rate of all effluent and water treatment plants, and trunk pipelines connecting sources of supply to centres of demand, but not inclusive of detailed distribution. The benefits from the recreational use of the river system were costed similarly, i.e., converted into an annual cash benefit and were deducted.

The model comprised three elements:

The river model which predicted how present river water quality conditions would change under alternative specified developments of effluent treatment facilities, water resources development, river flows, etc., and minimized the total annual costs of those developments.

The allocation model which selected the pipelines for public water supply to link sources to twelve population centres where the demands for additional water were expected to arise.

The timing of investment model which selected from the patterns of development proposed by the Allocation Model, the sequences of development to be followed which would minimize the total cost of meeting the additional demands within any specified constraints. For comparison purposes this was calculated as net present value.

2-7.1 The River Model

An inventory of river users was compiled which included all agencies which either affected or which were themselves affected by river water quality. Examples of users are:

1 Sewage and industrial treatment works
2 Brooks, streams and minor tributaries
3 River treatment lakes
4 Abstractions for storage and for water treatment plants
5 Abstractions for artificial recharge
6 Electricity generating stations
7 Fisheries
8 Weirs
9 Routine observation points (stations where water quality was sampled on a regular basis)

For some users, the model was allowed to exercise some choice over the type of use or the extent of the use; these choices were called decisions. For example, several possible effluent quality standards were postulated for effluent treatment works and alternative processes were specified for a water treatment plant. Users where different strategies could be exercised were called "active" users to distinguish them from "passive" users for which the model could offer no choice, e.g., brooks and streams.

The river system was divided into thirty-three reaches which could be identified geographically, and which were roughly uniform in flow and quality. Individual users were identified sequentially along each reach. The model started at its upstream boundary where it was given the relevant average flow and quality data and proceeded downstream from user to user adding up the effects on flow and quality of each user. The quality was described in terms of the mean concentrations of the following characteristics:

Biochemical oxygen demand
Ammoniacal nitrogen
Oxidized nitrogen
Dissolved oxygen
Suspended solids

Temperature
Total dissolved solids
Monohydric phenols
Chromium
Copper
Nickel
Zinc
Lead
Cadmium
Total organic carbon
Chloride

Statistical relationships—"qualigrams"—were devised where some indication was required of the variation in quality about the mean.[9]

Before being used in a predictive mode, the model was calibrated using 1969 data. For this purpose, mathematical functions (usually first-order equations) were derived to represent the effects of natural self-purification processes in the river. When a reasonable correlation had been obtained and the empirical equations were seen to be chemically reasonable, the model was tried under 1970 conditions for some reaches. Reasonable correlations between the observed and calculated data were obtained.

To simulate the benefits derived from fisheries, their quality was assumed to depend solely on water quality and five economic categories of fishery were identified:

Mixed, i.e., game and coarse fishery
Thriving coarse fishery
Good coarse fishery
Poor coarse fishery
No fishery

The type of fishery was predicted using a method developed by V. M. Brown[20] and algorithms developed in 1970 by I. C. Hart of the Water Pollution Research Laboratory (now Water Research Centre). This used an empirical relationship found between the calculated toxicity of a mixture of pollutants to rainbow trout and the presence or absence of fish in the rivers of the Trent system and was found to be highly successful.

Eight types of water treatment plant were specified to treat the expected range of raw water quality.

Also, five effluent discharge codes were used (see Table 2-9).

The action of the model is to travel through the stages successively starting in the headwaters, accumulating comprehensive information about the decisions available at each stage and the effects that these would have on river water quality taking into account the natural self-purification that occurs. The type of fishery that could be sustained in each reach was predicted and the type of water treatment plant selected appropriate to the quality of river water at the point where the abstraction was to be made.

Table 2-8 DEFINITION OF WATER TREATMENT PLANT TYPES
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Code	Treatment stages
WT1	River intake, storage, softening,* sedimentation, filtration, pH correction, chlorination and final storage
WT2	WT1 plus ammonia reduction or alternative disinfectant
WT3	WT1 plus granular carbon adsorption
WT4	WT2 plus granular carbon adsorption
WT5	WT1 plus desalination
WT6	WT2 plus desalination
WT7	WT3 plus desalination
WT8	WT4 plus desalination

* Sources from the river Trent (excluding Bunter recharge developments) were assumed to require 100 mg/l reduction in hardness

In the Trent system there are over 80 effluent discharges whose combined flow was in excess of 500 Ml/d each of which could be treated to one of the four standards defined above. Together these combined to produce about 10^{40} management choices, which was unmanageable, so to examine this range and to seek out optimal policies, a method of approximation based on the mathematical technique called "dynamic programming" was devised. Its success was due to the introduction of two concepts, river state and river stage.

The following definitions of the concepts are those of Warn.[21]

> For every point in the river system there is an amount of money—the total annual cash flow—which, within the decisions open to the model, is the maximum which could be spent on the effluents discharged upstream. Moving downstream, this cost increases because more effluents are discharged. Similarly for every point there is a minimum cash flow resulting from the treatment of each upstream effluent discharge to the standard associated with lowest cost. The range of costs between this minimum and the maximum represents the area within which the model is being allowed to exercise choice. To define the river state, this range of costs is divided into 30 equal segments, the number of segments defining the number of river states. The segments are numbered in order from 1 to 30 from minimum to maximum costs, and management policies whose expenditure on effluent treatment falls into a particular segment are said to produce the river state whose number is that of the segment.
>
> River stages are points in the river system at which the river state of the management policies is calculated. Stages are therefore the locations of each effluent treatment works for which costed decisions are specified. At each stage, the approximation afforded by dynamic programming is introduced.

The modus operandi of the river model can best be explained using an example which, for clarity, has only ten river stages (Fig. 2-3). At the upstream limit, no costs are attributable to the quality of water entering the section and up to the first stage, therefore, the river is said to be in state 1. At Stage 1 a discharge

Table 2-9 ASSUMED LEVELS OF WATER QUALITY PARAMETERS FOR EFFLUENT STANDARDS OF SEWAGE TREATMENT WORKS

Effluent standard code	Parameter levels (mg/l)						
	Biochemical oxygen demand (5-day)	Ammoniacal nitrogen	Oxidized nitrogen	Dissolved oxygen	Suspended solids	Total organic carbon	Percentile designation*
ET0	Corresponds to the quality of untreated sewage. Details not required by the model†						
ET1	Corresponds to the observed quality in 1969 as measured by the Trent River Authority. Quality varies for individual works‡						
ET2	11.0	20.0	5.0	6.0	17.0	20.0	50
	20.0	30.0	15.0	2.7	30.0		95
ET3	11.0	1.0	19.0	7.0	17.0	20.0	50
	20.0	3.5	28.0	3.6	30.0		95
ET4	5.0	1.0	19.0	8.0	4.0	10.0	50
	10.0	3.5	28.0	4.8	10.0		95

Other parameters: The ET1 values for temperature, total dissolved solids and chloride were assumed for the standards ET2, ET3, and ET4.

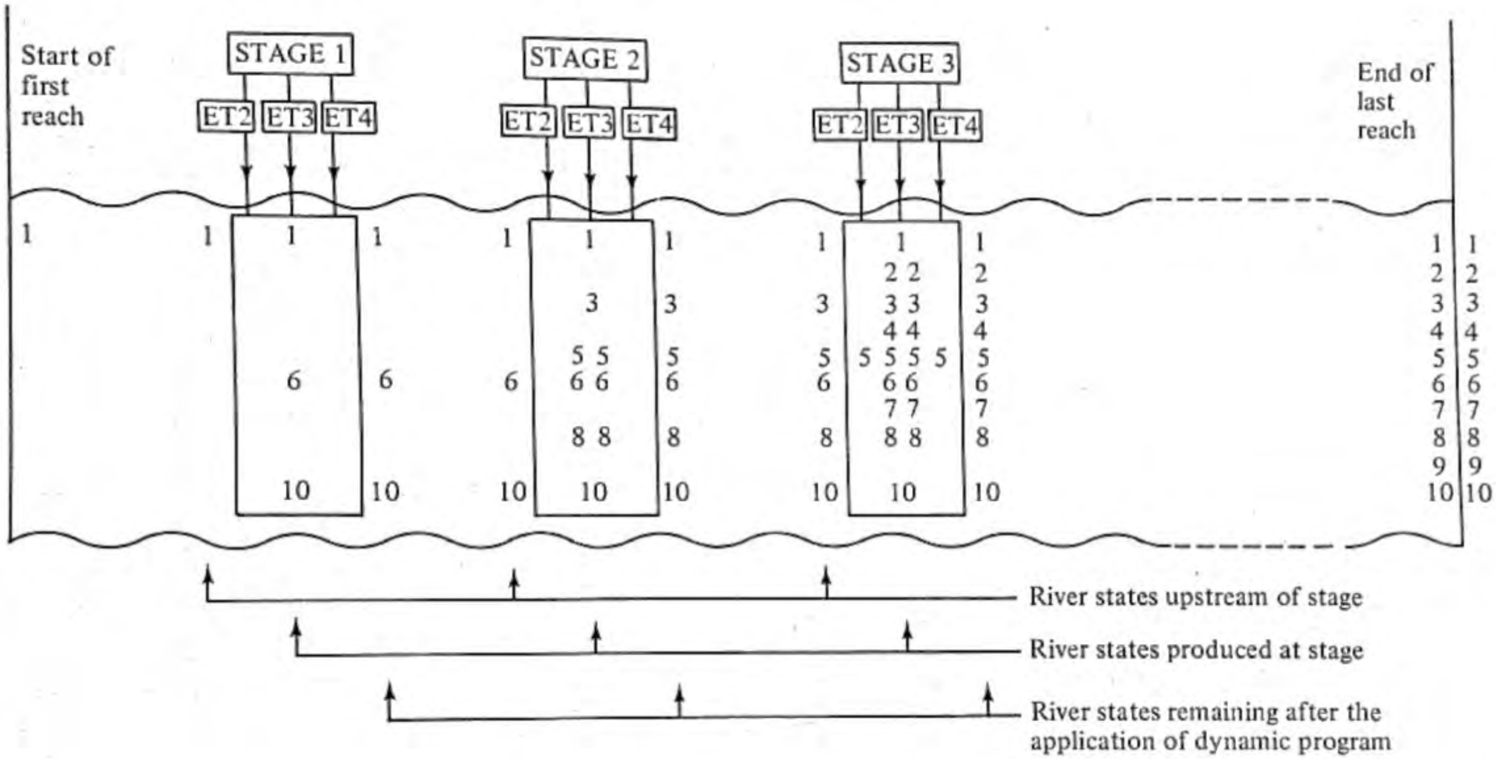
The ET1 value for monohydric phenols was assumed to reduce pro rata to 80 percent of any biochemical oxygen demand reduction on achieving the standards ET2, ET3, or ET4.

The ET1 values for the metals was likewise assumed to reduce pro rata to 60 percent of any reduction in suspended solids on achieving the standards ET2, ET3, or ET4.

* The 95 percentile values indicate the source of the model data. Thus the ET2 standard represents 95 percent compliance with the 20:30 standard. The model operates on the basis of annual mean quality (i.e., 50 percentile).

† Details are not required because "equal treatability" of raw sewage was assumed in the development of the cost functions for effluent treatment.

‡ The ET1 qualities vary for individual works as do the qualities for industrial effluents. Such figures are not amenable to concise summary and are given elsewhere.



The designation 5 5 etc, indicates that two separate sequences of decisions (policies) produce river state 5

FIGURE 2-3
Operation of the dynamic program of the river model.

of effluent is received into the river, the quality of which is determined by the choice of treatment plant ET2, 3 or 4. Each effluent standard has an associated cost and each produces a different quality of river water after mixing takes place. In the example, effluent treatment costs define the river states 1, 6 and 10. Downstream there may be a fishery or a water abstraction point. From the river quality associated with each state, the type of fishery may be predicted or for the abstraction, the type of water treatment plant determined. The type of fishery represents a financial benefit to the system and the water treatment plant a cost, which depends on the volume of the abstraction and the type of plant.

These benefits and costs are summed for each available river state and each state contains information about the choices made at effluent treatment works; the associated annual cash flow, the resulting quality of river water; the consequential decisions by downstream abstractors; the type of fishery and the total annual cash flow from the sum of the costs of all upstream river users.

At the second stage the process is repeated, but this time there are three possible qualities of river water to be taken into account. The procedure is repeated, resulting in nine possible choices covering the first two stages. These comprise nine possible water qualities with which nine costs are associated. From the diagram it will be seen that the nine alternatives are distributed between only six of the possible maximum ten states, but that more than one choice results in the same state. Where this happens, the dynamic program eliminates all but the least-cost way of achieving the particular state. Thus only six possibilities are passed forward to the next stage where the process is repeated.

When the downstream boundary is reached, the result is a limited number of policies or choices for the river system modeled, each of which has taken into account all the necessary constraints, etc. The state associated with the lowest total annual cash flow represents the optimal policy for the basin.

2-7.2 The Allocation Model

In addition to the data used in the river model, the allocation model was supplied with information about population projections, per capita consumption and industrial requirements for twelve demand centres located approximately at the principal centers of population of the area. From this information the demand for additional water and the consequent increase in effluent return was estimated for the beginning of each ten year time period up to the planning horizon at the year 2001.

Sites were specified at which water could be abstracted for public supply, most of which were on rivers, but some were to tap groundwater resources in the Bunter sandstone or the river gravels. In addition, imports of water from adjacent catchments were considered as alternatives to indigenous resources. Data on the costs of making specified quantities of water available at the abstraction points were included. These included the costs of reservoirs—either pumped storage or regulating—and the costs of transmission from the supply point to the center of demand. The latter included the cost of intake works, pumphouses and pipelines and the costs of pumping. The costs associated with the quality of the raw water and that of treatment to make it suitable for the purpose for which it was intended remained within the river model, which was used by the allocation model as a sub-routine.

In any one simulation, a choice of four sources was allocated to each demand center. The allocation model operates by assuming initially that all the specified links between the sources and demand centers have been constructed. The river model is then seen to establish the optimal distribution of costs between effluent and water treatment and the sum of these costs together with the supply costs (i.e., the storage plus transmission costs) provided a total annual cash flow for the network of supply–demand links. At this stage, the costs are, of course, totally unrealistic because each centre is being supplied several times over.

The model then isolates the link with the greatest total unit cost which may be removed from the network without leaving the demand center without a supply. A new total annual cash flow is calculated for the revised network and the model compares the actual saving brought about by deleting the link with the expected saving on the basis of unit cost, i.e.,

$$\text{Actual saving} = \text{TACF with link} - \text{TACF without link}$$

$$\text{Expected saving} = \text{Total unit cost} \times \text{demand}$$

These savings differ because the deletion of a link and the removal of its water treatment plant changes the balance of costs in the river model. The link is permanently deleted if the actual saving exceeds the expected saving but if the converse is true, the model investigates whether it would be preferable to delete the link with the second highest unit cost in the initial ranking. If the actual unit cost of the first link exceeds the expected unit cost of the second link, the model still deletes the first. Otherwise the first link is restored to the network and the search procedure repeated, comparing the second and third ranking links and so on until a permanent deletion is made.

The expected unit costs are then revised following a simulation of the new network and the most expensive of the remaining links investigated. This procedure is repeated until no redundant links remain, the final network being considered the optimum solution.

This operation was repeated for the years 1981, 1991 and 2001 independently. While in principle the model could have selected incompatible strategies, in practice it was found that a reasonable pattern of development usually evolved. If the model produced incompatible patterns, it could be forced to produce a coherent pattern by selecting the solution for one of the years, usually 2001 and constraining the model to produce compatible solutions for the other two.

2-7.3 The Investment Model

The purpose for which this model was designed was to determine the best schedule of resource development to meet the growing needs for water over the planning period. A series of programs was written, one for each type of installation—sewage works, water treatment works, etc., which calculated the costs of alternative construction strategies and selected the development schedule with the minimum net present value. In practice it proved to be more convenient to perform this calculation by hand.

The three elements of the model can be used in sequence as an integrated whole or two of them—the river and investment models—may be used independently. The allocation model, however, can only be used in conjunction with the river model which forms an essential subroutine of it.

Of the three it is perhaps the river model which will be found to be of most interest. It can be used on its own for a variety of purposes, from examining the overall water quality of the river system to predicting the local effects of a proposed new discharge and thus its effect on other users. While it was designed to operate using annual mean concentrations it can also be used for operating under average summer, spring, autumn or winter conditions as the data become available. The facility to simulate sixteen water quality characteristics simultaneously, to follow these with a further sixteen if necessary and to be able to exchange individual constituents to meet the particular requirements of the river system being modeled, represents a considerable advance in modelling techniques. It was through this facility that the model's prediction of the river's ability to sustain fisheries became possible; moreover, it was also possible to forecast with some accuracy the quality of those fisheries.

2-8 RESULTS

Many simulations were carried out using the model to produce both "global" and "local" results, but there were six principal runs of interest. In each of these the model was run for the years 1981, 1991, and the planning horizon, 2001. Each produced the least cost solution subject to the imposed constraints. The simulations were:

1 Maximum development of storage within the Trent basin.

2 Maximum use of the river Trent by direct abstraction only.

3 Maximum use of the river Trent by direct abstraction and recharge of the Bunter Sandstone.

4 Maximum importation of water, including augmentation of the flow of the river Dove with water from the Dee estuary.

5 Optimum solution given a free choice of those types of water supply considered in 1–4 above.

6 Optimum solution given a free choice subject to the constraint that one reservoir would be built in the Trent basin before 1980.

In every case, for practical reasons, the model was constrained to supply the four West Midland demand centers (Birmingham, Nuneaton, Walsall, and Wolverhampton) by imports of water from outside the Trent basin.

The results of the simulations were reported by the Water Resources Board[6] but the result of simulation run 5 is reproduced in Table 2-10.

The locations associated with the numbers given in Table 2-10 are shown in Fig. 2-4.

FIGURE 2-4
Demand centres and supply points.

Table 2-10 LEAST COST SOLUTION OF SIMULATION 5 AT YEAR 2001

Demand centre	Supply point	Type of supply	Water treatment type
Birmingham	Bewdley (23)	Import	WT1
Mansfield	Colwick (18)	Direct Trent	WT1
Leicester	Thrumpton (17)	Direct Trent	WT1
Chesterfield	Thrumpton (17)	Direct Trent	WT1
Nuneaton	Willey (24)	Import	WT1
Scunthorpe	Dunham (20)	Direct Trent	WT1
Loughborough	Thrumpton (17)	Direct Trent	WT1
Nottingham	Colwick (18)	Direct Trent	WT1
Derby	Swarkestone (13)	Direct Trent	WT1
Stoke	Yoxall (10)	Direct Trent	WT1
Wolverhampton	Coalport (22)	Import	WT1
Walsall	Coalport (22)	Import	WT1

Total annual cash flow (£ thousand):	1981	1991	2001
	2,610	7,230	13,380

Note: For 1981 and 1991 WT4 is necessary on Trent abstractions. The cash flow figures do not take account of the write-off of redundant plant. This is, however, brought into the calculation of present value (see Table 2-11)

The predicted distribution of fish if the development plan suggested by simulation run 5 were implemented is shown in Fig. 2-5. Due to the improved effluent treatment standards that would be in force by 2001, good quality fisheries would be expected in most parts of the river system. Exceptions would be the upper part of the river Tame, the Soar and the Erewash.

This may be contrasted with the substantially different pattern of fish distribution which would occur in 2001 if the volume of effluent increased as before, but the standard of treatment were maintained at the present day level. This is shown in Fig. 2-6. Due to the deterioration in river water quality because of the increased proportion of effluent, substantial lengths of the river system would be fishless, the best class of fishing being restricted to only a small length of the river Dove.

For each simulation the equivalent net present value of the assumed program of development was also calculated manually since this was the basis on which the Water Resources Board usually made comparisons. A summary of the costs of all the simulations is given in Table 2-11.

It should be emphasized that these simulations were made for the purposes of evaluating the model and did not imply any recommendation about future resource development. They were based on some fundamental assumptions, for example the type and extent of water treatment necessary to produce an acceptable supply from Trent river water, which have yet to be tried out in practice. Nevertheless, they established the usefulness of the model as a tool for assisting with decisions in the future.

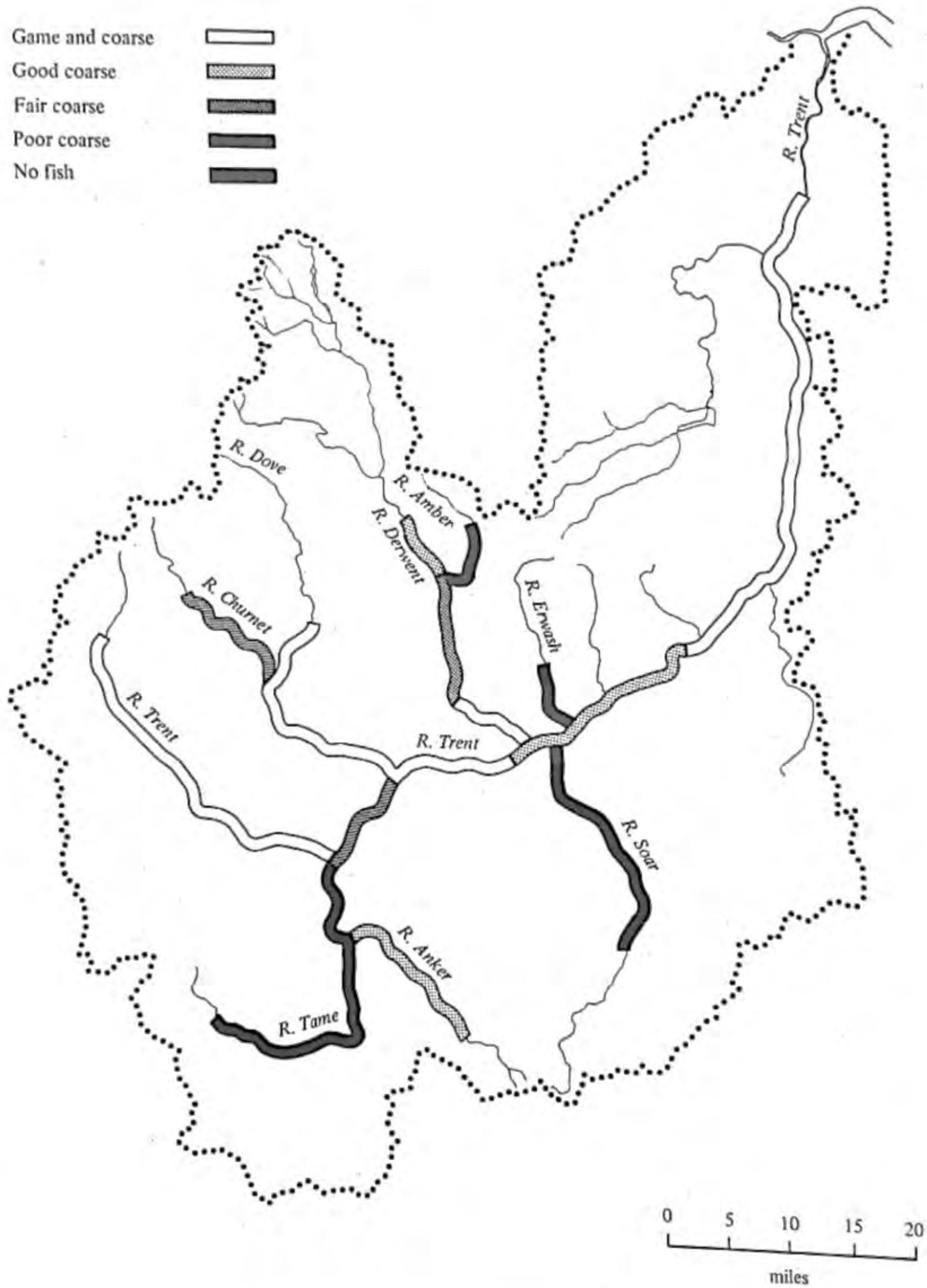


FIGURE 2-5
Predicted distribution of fisheries in 2001 assuming effluent treatment as in simulation 2.

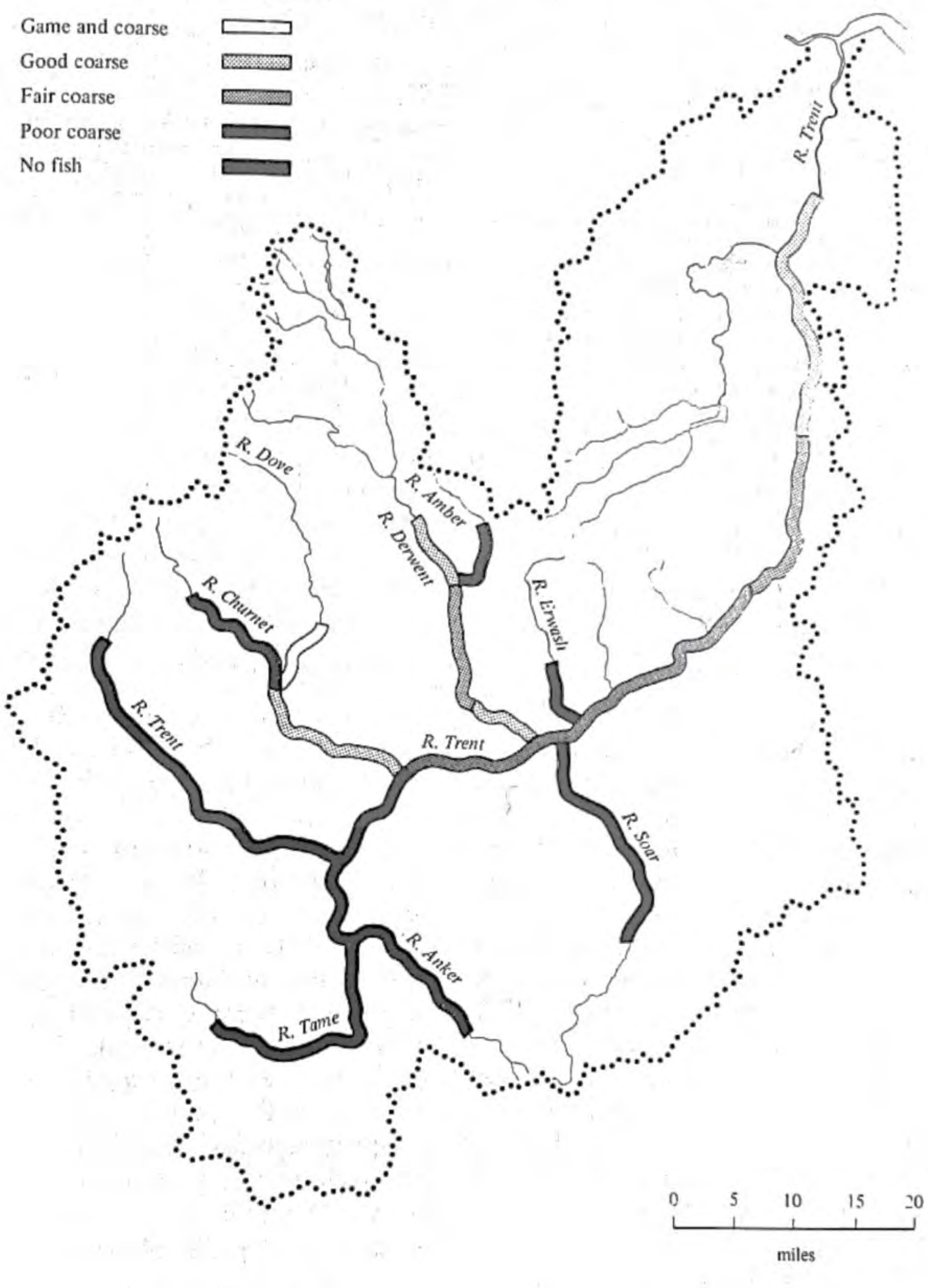


FIGURE 2-6
Predicted distribution of fisheries in 2001 assuming present standard of effluent treatment.

Table 2-11 COSTS ASSOCIATED WITH THE DIFFERENT DEVELOPMENT STRATEGIES

£ million

Simulation	Total annual cash flows 2001	Capital cost	Present values: Total cost	Fishery benefit	Net
1 Maximum internal storage	14.5	182	70.8	8.5	62.3
2 Maximum use of the Trent (Direct abstractions only)	13.4	184	70.8	10.9	59.9
3 Maximum use of the Trent (Direct abstractions and Bunter)	13.4	184	70.8	10.9	59.9
4 Maximum importation	17.8	213	86.7	8.5	78.2
5 Optimum development	13.4	184	70.8	10.9	59.9
6 Free choice after construction of one reservoir before 1980	14.3	199	73.4	10.0	63.4

2-9 CRITIQUE

The validity of the results obtained from any mathematical model is dependent on two principal factors:

1. The assumptions and approximations built into the model.
2. The accuracy of the input data.

The individual projects comprising the Trent Research Programme, while providing the input data and improving the assumptions incorporated into the model, played a much more important role. They increased the understanding and added to the fundamental knowledge of water quality relationships as well as about new techniques for use in water and effluent treatment. They also identified areas in which further research work was necessary.

The evaluation of the simulation runs revealed that the model was sensitive to water treatment costs but relatively insensitive to those of effluent treatment. If all effluents were treated to the ET2 standard as a desirable minimum, the marginal cost of achieving higher standards would be relatively small. Better effluent treatment would, of course, bring significant benefits to the river both in fisheries and in general amenity and it would also reduce the cost of the treatment necessary for water abstracted for public supply purposes. The model demonstrated that it could select those effluents which, by raising their standards, would have most effect on river water quality, thereby enabling valid comparison to be made of the influence of different discharges on the quality of the water at specific points in the river.

Given sufficient reliable data and through its ability to simulate sixteen water quality characteristics simultaneously, the river model is a powerful tool to help management formulate the most effective policies for the river system as a whole. Nevertheless, the model has limitations in this respect, for example, the use of average water quality values determined from the annual distribution of quality may not be sensitive enough for practical purposes. It may prove necessary to operate using seasonal averages to take account of seasonal variations in quality. Given the requisite data the model can do this. It may even be able to operate using average monthly quality and flow figures. It is not, however, suitable for operational management, nor will it "ration" water between demand centers—it always assumes that the demand can be met in full.

Although calibration was carried out using two years' data and the conditions were not very different in those years, subsequent work has shown that the relationships assumed are robust enough to reproduce the observed situation in the river when the flow conditions were very different. There are, however, still discrepancies in the mass balance of non-ferrous metals which need further study; one possible explanation is that it may be due to atmospheric deposition and storm or other discharges.

Not unexpectedly, the model proved to be very sensitive to the level of total organic carbon (TOC). This blanket measurement is not altogether satisfactory because of the lack of knowledge about the constituents giving rise to the measurement or the concentrations at which they would become harmful. Because of these uncertainties, a maximum level which could be tolerated in "potable" water was assumed and activated carbon treatment was considered to be necessary, although costly, to keep the level as low as possible. Despite these drawbacks it was thought essential to keep TOC in the model because of its importance in representing the overall level of organic materials present, including those unknown compounds which might be harmful.

The allocation model functioned well but could, under certain combinations of circumstances, give rise to problems. The mechanism for selecting links between sources and centers of demand worked successfully so long as the decision to remove a single link did not lead to repercussions in the remainder of the network, whose total cost was similar to the costs associated with the components which constituted the link. A similar situation could arise in processing the results of the allocation model into a plan of development through time. The modifications to cost introduced by this process were assumed initially not to be so substantial as to invalidate the selections made by the allocation model.

In practice it was easy to identify the areas where these types of assumption were critical and where there were particularly marginal decisions, these were subjected to further analysis in which the consequences of the individual selections were fully evaluated.

The timing of investment model was never used, as it was more convenient to perform the calculations manually using a desk top calculator.

Of the three elements of the Trent model, it is undoubtedly the river model which has aroused the most interest. This is perhaps because it has successfully incorporated new techniques and has proved to be useful in practice. Indeed it has been applied to the Bedford Ouse to facilitate a study of the present and future effluent disposal problems in that river due principally to the construction of a new town.[22] This model was also extended to include the Great Ouse so that water quality aspects of the feasibility of constructing storage reservoirs in the Wash could be studied.[23]

Finally, although the work was completed in 1970, it is still used by the Severn–Trent Water Authority for the investigation of specific quality problems in the river.

SUMMARY

The increasing demand for water for potable, industrial, and agricultural purposes within the Trent basin, the limited resources of the clean tributaries, the few reservoir sites available and the pollution load of the river, were causes of concern to the Trent River Authority. The demands for additional water might be met by a number of different methods, including the use of the polluted river itself.

There were many other facets of the problem to be taken into account, among them the actual and potential uses of the river system including recreation and the possible importation of water into the basin. Finally, and perhaps fundamentally, the basic conflict had to be resolved between those who used the river as an effluent carrier and those who wished to use it as a major source of water. To find the best way of using the resources of the whole basin the Trent Research Programme, a cooperative venture costing over half a million pounds and lasting three years, was designed and undertaken.

By the end of the program a mathematical model, whose success was based on the use of two principles—dynamic programming and quality states—had been developed. It had considerable complexity and flexibility and operated at three levels: river model level, which dealt with up to sixteen chemical quality determinands simultaneously; allocation level and optimization of capital investment level.[1]

ACKNOWLEDGEMENTS

Quotations from the papers listed in the bibliography have been freely made and the help of their authors is gratefully acknowledged. The opinions expressed are, however, those of the author and do not necessarily represent those of the Department of the Environment.

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3

MODEL OF THE SAINT JOHN RIVER, CANADA

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3-1 INTRODUCTION

During the latter half of the last decade it became increasingly evident that water pollution in Canada was rapidly becoming a problem of major national concern. It was in line with the popular feeling that the environment was deteriorating in many different ways, and that something should be done to reverse the process. A vocal and active group of citizens began to question the concept of accepting economic growth for its own sake; the undesirable side effects of decades of accelerated industrialization and urbanization were brought frequently to the attention of the public by the mass media, and there developed a deep concern about the negative environmental and ecological consequences of economic growth.

The United Nations Conference on the Human Environment, held at Stockholm in 1972, further highlighted both the need and the importance of environmental management. The "Earth Day" attracted an estimated 20 million participants in the United States alone, and because of increasing public concern about the environment, new Departments of the Environment were set up in many countries.

The newly created Department of the Environment in Canada started to investigate environmental degradation problems in many areas. In the field of water resources management, the most striking example of environmental degradation was in water quality. Most of the accessible lakes and rivers were deteriorating due to municipal, industrial and agricultural wastes. People became shocked to find that Canada's largest city, Montreal, had no sewage treatment facilities: everything was dumped straight into the Saint Lawrence River. A similar situation existed for several other centres of population.

It should be realized that the so-called "environmental crisis" of the early 1970s had developed not only because of the neglect of the overall environmental management process and increasing levels of waste discharge to the water bodies, but also partly due to the increasing perception of pollution problems resulting from society's need or demand for a better quality of life, which, in turn, was a by-product of increasing levels of affluence and education. Within a management context, the complementarity between the environment and development process began to be emphasized. It was argued, quite correctly, that for the development to continue and be self-sustaining, environmental implications must be investigated and appropriate steps should be taken to reduce the magnitude of unwanted side-effects.

For this reason the decision was made to develop mathematical models as a tool for water quality management of the Saint John River System.

3-2 DESCRIPTION OF THE STUDY AREA

The Saint John is an international river, the upper part of which is in the United States and the lower portion in Canada. Its length is approximately 700 km and it has a drainage area of 54,934 km^2. Nearly 36 percent of the drainage area is in the State of Maine in the United States, and the balance, 64 percent, is in Canada—13 percent in the Province of Quebec and 51 percent in the Province of New Brunswick, as shown in Fig. 3-1. Among its major tributaries are the Allagash, Tobique, Fish, Aroostook and Madawaska Rivers. Because of its geographical nature, i.e., first international and then interprovincial in Canada, the institutional arrangements necessary for rational management of its water resources have not been easy. The models developed and discussed in this chapter are for the Canadian section only: the modeling approaches used for the portion in the United States are discussed in Chapter 4.

In some ways, the Saint John can be considered to be a typical Canadian river. It is certainly a major Canadian river, both in terms of its physical size, i.e., drainage area and length, and its socio-economic importance to the region through which it flows. Like any other river, its water is used for a variety of purposes like hydropower generation, industrial and municipal uses, waste assimilation, recreation—including sports and fishing—navigation and flood control. The river has an added importance since it has been noted as a major spawning habitat for Atlantic salmon. Thus, significant conflicts exist at present between various water uses.

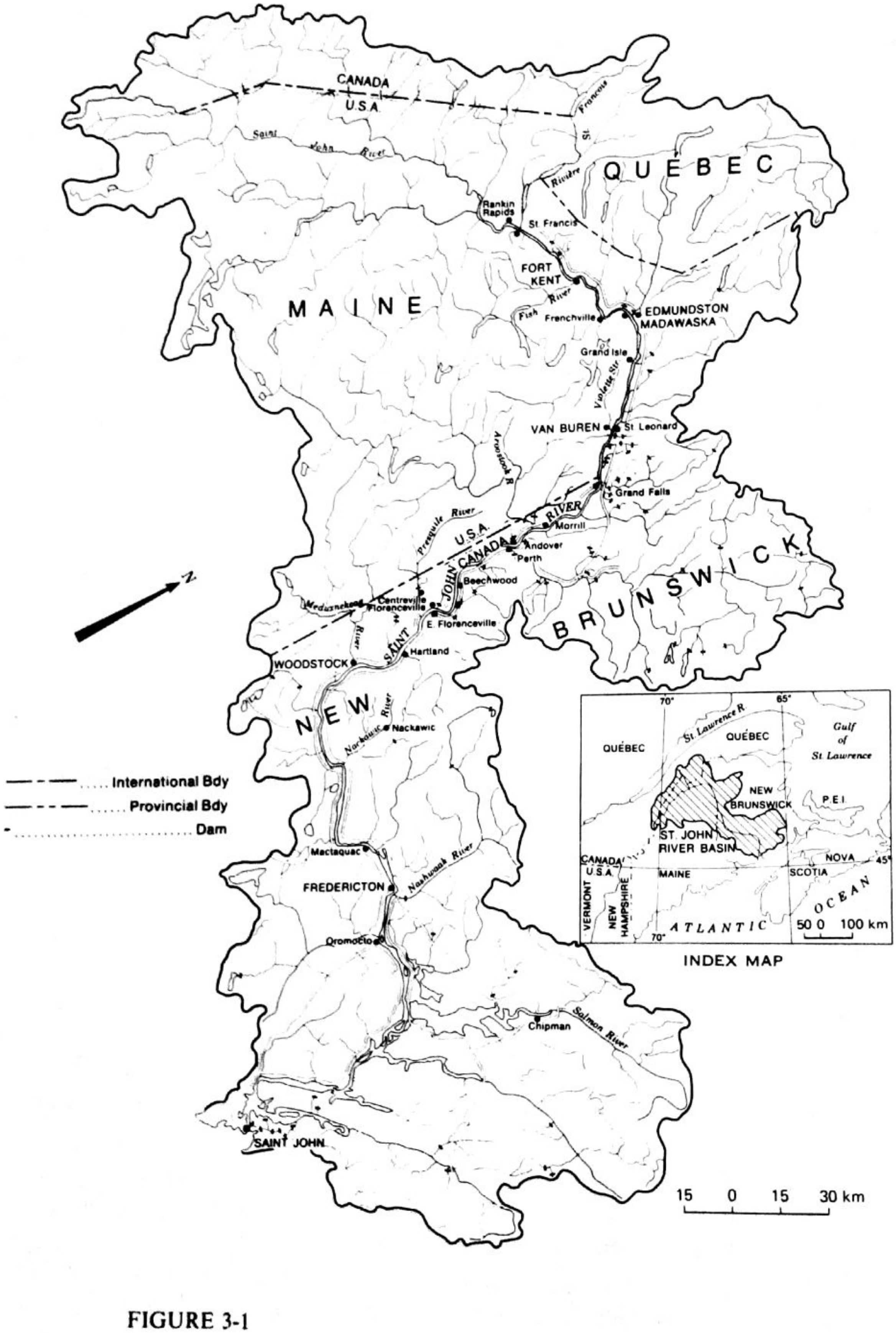

FIGURE 3-1
Saint John River Basin.

In terms of water quality, the problems are quite severe at present. During the study period fifty-nine significant sources of pollution were identified between the headwaters and Oromocto (Fig. 3-1). This did not include major sources of pollution that exist on the estuarine region near the City of Saint John, and numerous other minor sources. Although most of these are municipal effluents, several major sources of pollution arise from industry as briefly mentioned later.

The main industry in the Saint John River Basin has been the harvesting and processing of forest products. From water quality considerations, the major source of pollution has been the pulp and paper industry, which has three principal locations within the Canadian side of the river basin, at Edmundston-Madawaska, Saint Anne-Nackawic and Saint John. The pulp and paper industry has dominated the industrial scene for a long time, and accordingly there are several old mills which can be characterized by their somewhat low earnings. Economically it may not be viable for them to institute stringent pollution control measures. The newer mills, such as at Saint Anne de Nackawic that manufacture high-grade kraft and other kindred products, are in a better financial situation. Socio-economic analysis of the possible industrial scenario of the river basin, indicates that the pulp and paper industry will continue to play a dominant role in the general economy of the region.

After pulp and paper, the second most important industry within the study area at present is the food-processing industry, which is based mainly on potatoes. Basically two types of products are manufactured, food products—such as potato chips, french fries and mashed potato—and starch products. The former have experienced rapid growth during the last decade due to both economics and product innovation, but the latter has become a marginal, and it is highly likely that many plants will go out of production during the next few years.

The river has been used extensively for logging ever since the time of the early settlers, and also serves as a transportation link. Its tributaries form the most significant spawning ground for the Atlantic Salmon on the east coast of Canada,[1] and consequently the Saint John river system has been an important element for the fishing industry of the Maritime Provinces. The river has always been used for recreational purposes, and the fish, wildlife and water fowl of the area depend on the river and its tributaries for their survival.

The river flow is partially regulated for developing hydroelectric power, having a current total installed capacity of 530 MW.[2] Construction of the hydroelectric dams, significant increase in the amount of industrial wastes discharged to the river, and poor land use and forest product harvesting practices have all contributed to a substantial reduction in the spawning of the Atlantic salmon. This has naturally contributed to a significant decrease in the salmon run. Even though commercial salmon fishing plays a minor role in the overall economy of the basin, its importance as a sport and its sociological implications should not be underestimated. Hence, preservation of the salmon fishery remains an important objective so far as the general public is concerned.

The deterioration of the water quality of the river occurs due to a variety of causes: organic wastes, nutrients, floating and suspended solids, toxic chemicals, bacteria, etc. The most noticeable form of water pollution has been due to extensive discharge of organic wastes (BOD), which has resulted in rather low dissolved oxygen (DO) contents in many parts of the river. The total untreated daily load was estimated to be 1,438,000 lb of BOD, nearly 59 percent of which was contributed to by the pulp and paper industry at Edmundston–Madawaska region. Furthermore, potato-processing plants located primarily on the Aroostook and the Presquile Rivers, tributaries of the Saint John, are responsible for contributing significant amount of effluents between Edmundston and Woodstock.

3-3 MODEL DEVELOPMENT

While the study outline and the overall framework were developed by the author, the models themselves were developed by Acres Limited. The development was closely supervised by a steering committee, and the actual process used for model development has been discussed in detail elsewhere.[3]

During the very early stages of model development, it was decided to build two models—a programming or optimization model and a simulation or descriptive model—and to ensure the complementarity between the two models. In the context of the present discussion, these two types of model can be defined in terms of their relationships to problem-solving.[4] Programming models, for a given objective function that can be explicitly formulated in mathematical terms, attempt to derive the optimal policy. This means that the objective function can be reduced to maximizing a single variable in order that a unique solution can be obtained. Examples of this type of model are linear and nonlinear programming, stochastic programming and integer programming. Simulation models, on the other hand, attempt to predict possible future consequences due to a set of assumed exogenous variables and policy alternatives. They do not contain explicit definition of the objectives, nor can they be used to optimize the system. However, in certain situations an optimization subroutine can be used to provide partial optimization.

While programming and simulation models have been built extensively for water quality management purposes, their complementary use has not been that widespread in practice. Complementary use of such models was proposed by Loucks for the comprehensive planning of the Delaware River Basin, where three programming models and one simulation model were developed to analyze the problems of water supply, water quality, hydroelectric power generation and recreation.[5] Linear programming techniques were used for all three programming models, two of which were based on the concept of deterministic hydrology and the third one on stochastic hydrology. Deterministic models were used to carry out the preliminary screening, and then the stochastic model was used to carry out further screening of a reduced number of projects. Finally, detailed analyses of a select few designs and operating policies were carried out by the simulation model.

In spite of such advantages of using two complementary types of model, comparatively few cases of such usage currently exist. For the Saint John River Basin Study, it was decided to adopt this approach. Furthermore, major emphasis was placed on large effluent discharges which had a significant effect in the deterioration of water quality, rather than on other water use purposes like hydropower, water supply fisheries and recreation which could be indirectly considered.[7]

3-4 PROGRAMMING MODEL

The mathematical models developed use biochemical oxygen demand (BOD) and dissolved oxygen (DO) as water quality indicators in the river, and BOD as the measure of pollution in the waste effluent. This was because high organic loading is the most significant water quality problem in the basin, even though several other types of quality problem can be identified. The decay of degradable waste is more difficult to analyze than for nondegradable wastes since a reduction in pollution concentration may be due to dilution in the river as well as to the decay process.

As the organic wastes start to decay in the water, they begin to consume oxygen present in the water body, thus reducing the instream dissolved oxygen level. If the dissolved oxygen content of water continually remains above zero during this biodegradation process, the situation continues to be aerobic. However, at lower DO levels, sensitive types of game fish like salmon may have difficulty surviving. When the organic waste load is excessively high, the DO content effectively reduces to zero parts per million. It produces "septic" conditions, with resulting unsightliness, unpleasant odors and reduced waste stabilization, which have catastrophic effects on aquatic life. This is diagrammatically shown in Fig. 3-2 as the DO sag curves.

The programming model developed uses linear programming (LP) for the direct solution of the relation between BOD and DO. It was based on the classical Streeter–Phelps equation, having two first-order differential equations which define the mechanics of BOD–DO relations. Assuming steady-state conditions of given hydrologic regime and constant effluent discharge rates, it can be shown that the DO deficit at any point is the sum of the component DO deficits due to the individual sources. Expressed mathematically, it becomes

$$D_j = \sum_i d_{ij} L_i \tag{3-1}$$

where L_i = BOD load entering the river at point i

D_j = total DO deficit at quality point j

d_{ij} = transfer coefficient defining DO deficit at point j due to unit waste load entering at point i

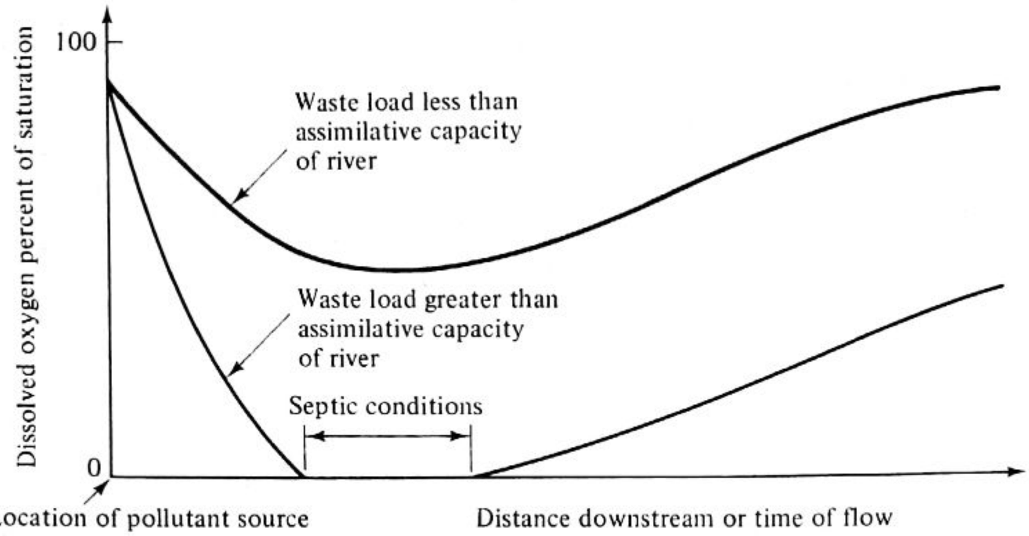

FIGURE 3-2
Oxygen sag curves: Effect of waste load on dissolved oxygen in the river.

The DO deficit D_j can be constrained so as to ensure that a specified level of DO is maintained, which can be represented by the following inequality:

$$D_j \leq S_j - \text{DOST}D_j - \rho_j \tag{3-2}$$

where S_j = saturation concentration of DO
$\text{DOST}D_j$ = specified DO standard at j
ρ_j = total DO deficit due to all uncontrollable sources upstream

If the above equations are combined, the constraint or requirement for a particular point j will be given by

$$\sum_i d_{ij} L_i \leq S_j - \text{DOST}D_j - \rho_j \tag{3-3}$$

This equation is in a standard linear programming format, since for any point the right hand side of the equation is constant. Such a formulation can ensure that the DO standard specified is maintained. If a large number of such quality points are defined, each with its appropriate constraint equation, the entire river system can be constrained to satisfy minimum desired DO standards.

If it is further necessary to limit the amount of BOD remaining in the stretch of the river, a suitable constraint equation can similarly be developed. The equation becomes

$$B_j = \sum_i b_{ij} L_i \leq \text{BODST}D_j \tag{3-4}$$

where B_j = cumulative BOD at point j due to combined effects of all upstream sources of effluents
b_{ij} = BOD concentration at j due to unit concentration entering at effluent point i
$\text{BODST}D_j$ = maximum possible concentration of BOD at j

Like Eq. (3-3), this function is also in standard LP format.

Normally water quality programming models use only one decay coefficient to characterize the decomposition of each organic effluent: the carbonaceous component which decays rather rapidly. This is because the effects of the slowly decaying nitrogenous component becomes important only at least five days after oxidation starts. Accordingly this portion of the organic effluent can be neglected for rivers having short travel times. The situation, however, is different for the Saint John River which has a long travel time. Hence, for developing the programming model, it was necessary to characterize each waste with two separate components, each having its own unique decay rate. Under such conditions, Eqs. (3-3) and (3-4) can be used to determine the effects of these two components by treating them as two different effluent sources originating from the same point. Figure 3-3 shows the formulation of the removal efficiencies of the various treatment plants and the associated cost functions for the programming model.

One of the most important objectives of any water quality management process is the concept of economic efficiency,[4] but considerable difficulty exists in maximizing the net benefits accruing from water quality improvement processes. This is because it is difficult to evaluate such benefits, especially when many of the benefits are intangible, and thus difficult to quantify.[6] Accordingly it is often simpler to minimize the basin-wide cost of treatment works that can satisfy

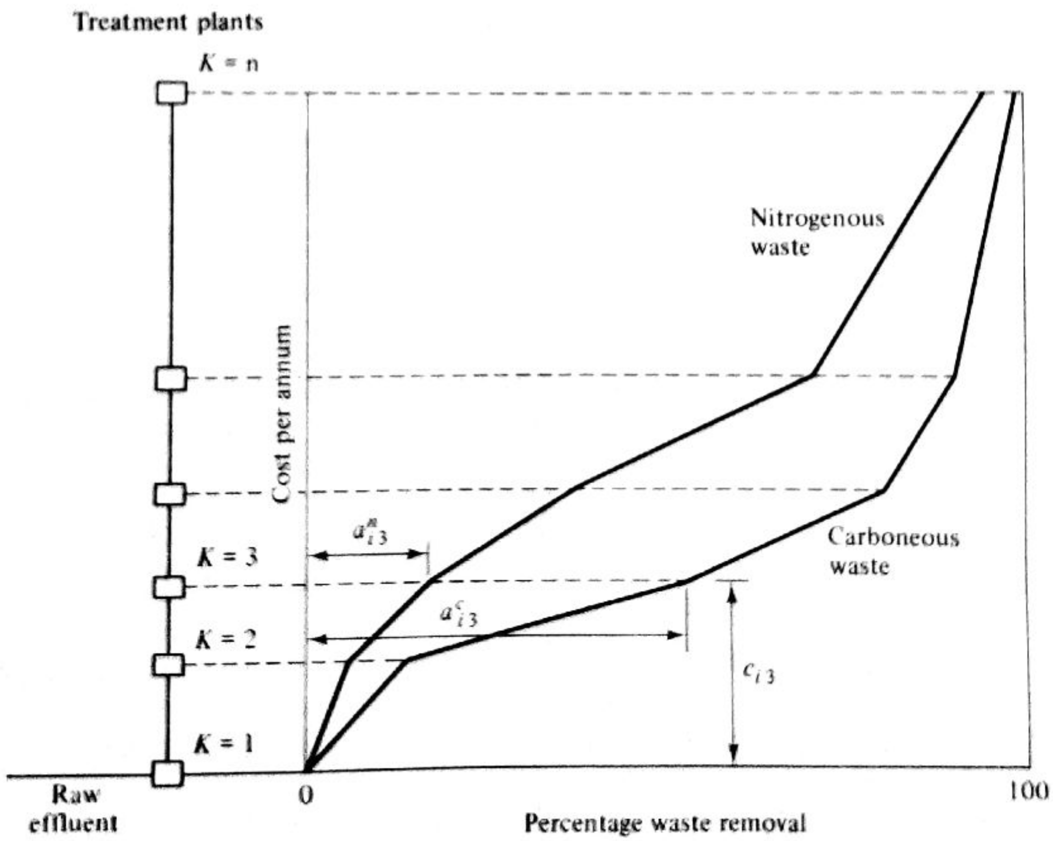

FIGURE 3-3
Relationship between cost and waste removal.

certain specified water quality standards rather than attempting to maximize the benefits. In functional terms, it can be specified as

$$\min \sum_{j} C_i(L_i) \tag{3-5}$$

where C_i = cost function for varying levels of effluent L_i at each waste discharge point i.

Since each effluent had been separated into carbonaceous L_i^c, and nitrogenous, L_i^n, components, the cost function at each effluent point becomes dependent on both components. Hence, the function expressed in Eq. (3-5) had to be reformulated as follows:

$$\min \sum_{i} C_i(L_i^c, L_i^n) \tag{3-6}$$

Generally the cost function C_i, as represented in Eqs. (3-5) and (3-6) is nonlinear. However, this is not a major problem since linearization can be approximated by piece-wise linear representation of each cost curve. Such a modification enables the use of an LP approach.

In addition to the minimum cost objective, the programming model can also maximize water quality for a specified cost, by assuming dissolved oxygen as the surrogate of water quality. This means that the level of dissolved oxygen at a representative point within a given stretch of the river is assumed to be directly proportional to the environmental quality, that is, increase in DO means increase in environmental quality. "Weighting" coefficients can be used for each quality point to reflect the relative social desirability of the different sections of the river, by considering such factors as proximity and accessibility to centres of population, quality of beaches and types of recreation available. The "weighting" coefficients have to be subjectively estimated. Mathematically this can be expressed as

$$\text{Max} \sum_{j} W_j \text{DO}_j \tag{3-7}$$

where W_j = weighting coefficient for the stretch of river represented by point j
DO_j = level of DO at point j

In addition to DO and BOD constraints discussed earlier, the following additional budgetary constraints can be added to reflect different political and institutional considerations:

1 Minimum or maximum levels throughout the basin, or for portions thereof.

2 Total investment for one portion of the basin, or for one general group of polluters, to be less than, or greater than, a certain specified amount.

3 Total cost of treatment for one portion of the basin, or for one general group of polluters, to be equal to less than, or greater than, some factor times the total cost for another portion or group.

3-5 SIMULATION MODEL

In contrast to the steady-state hydrology and constant effluent discharge rates used for the programming model, the simulation model considers more precisely the levels of waste treatment required due to varying hydrology and differing effluent discharge rates. The difference between these two aspects, especially in terms of the eventual use of the model, is important. For example, within the context of the programming model, it is necessary to specify conditions which will produce extreme conditions of water quality. Traditionally this means coupling of low flow conditions with high-effluent discharge rates. The low flow conditions are often assumed to be that value which is not exceeded for seven consecutive days with an expected return period of ten years. Combination of such an extreme hydrologic regime with maximum effluent discharge rates can often lead to an ultraconservative solution. Furthermore, it is neither possible to select a flow regime that provides worst conditions at all points of the river, nor feasible to analyze possible ecological implications without information on the magnitude, frequency and persistence of the violations of various levels of water quality.

A fundamental requirement of any simulation model is to select an appropriate time interval, since water quality conditions vary with time due to changes in flow rates, variations in effluent discharge rates and fluctuations of photosynthetic oxygen production and benthal demand. Changes in flow and effluent discharge rates will evidently affect the concentrations of various pollutants in streams. Just as streamflow rates vary with time, so do industrial effluent discharge rates, which could fluctuate with the season or even the hour. Photosynthetic oxygen production varies throughout the day with changes in the intensity of solar energy, but this daily variation can normally be neglected for large rivers, especially those that are basically free of algae. It could, however, be an important consideration for small streams and lakes. Benthal demand can change due to lack of available oxygen, and by scouring or deposition on the river bed.

Because of these fluctuations, it is important to analyze the effects of changing water quality with time on the aquatic system which can be affected by daily or hourly changes in the DO level, especially when such fluctuations are large and occur near their threshold level. The use of a small time increment in the simulation model will undoubtedly enable one to predict detailed responses in water quality, but such models are more expensive to develop and to operate due to longer running times necessary and more extensive data required. While the use of a larger time interval simplifies the model-building process and lowers the operational costs, it becomes difficult to capture the detailed responses of extreme water quality conditions. Hence, it is necessary to make a compromise between the cost of model development and operation and the results necessary for an adequate appreciation of the responses of the ecological system. For the Saint John River System, it was decided to use a daily time interval, with two-day and three-day time intervals being available as options. Choice of such a time interval allows analyses of variations in pollutant concentrations on a daily basis, but any fluctuations due to diurnal effects, i.e., short-term changes in effluent discharge rates or photosynthetic activity, can only be approximated in an average sense.

The main emphasis of the simulation model was to analyze time-varying BOD–DO relations for the river system, but it should be noted that the model can consider other nonoxygen-consuming pollutants as well. The main difference in approach is the fact that these pollutants are primarily subject to increasing dilution only as they proceed downstream.

The simulation model consists of five distinct segments as shown in Fig. 3-4. The first three segments—synthetic seasonal flow generator, daily (in the generic sense) hydrology inflow generator, and routing component—are basically hydrologic in character. The variations in the pollutant concentrations are analyzed in the fourth segment, and the last segment generates appropriately summarized results.[7]

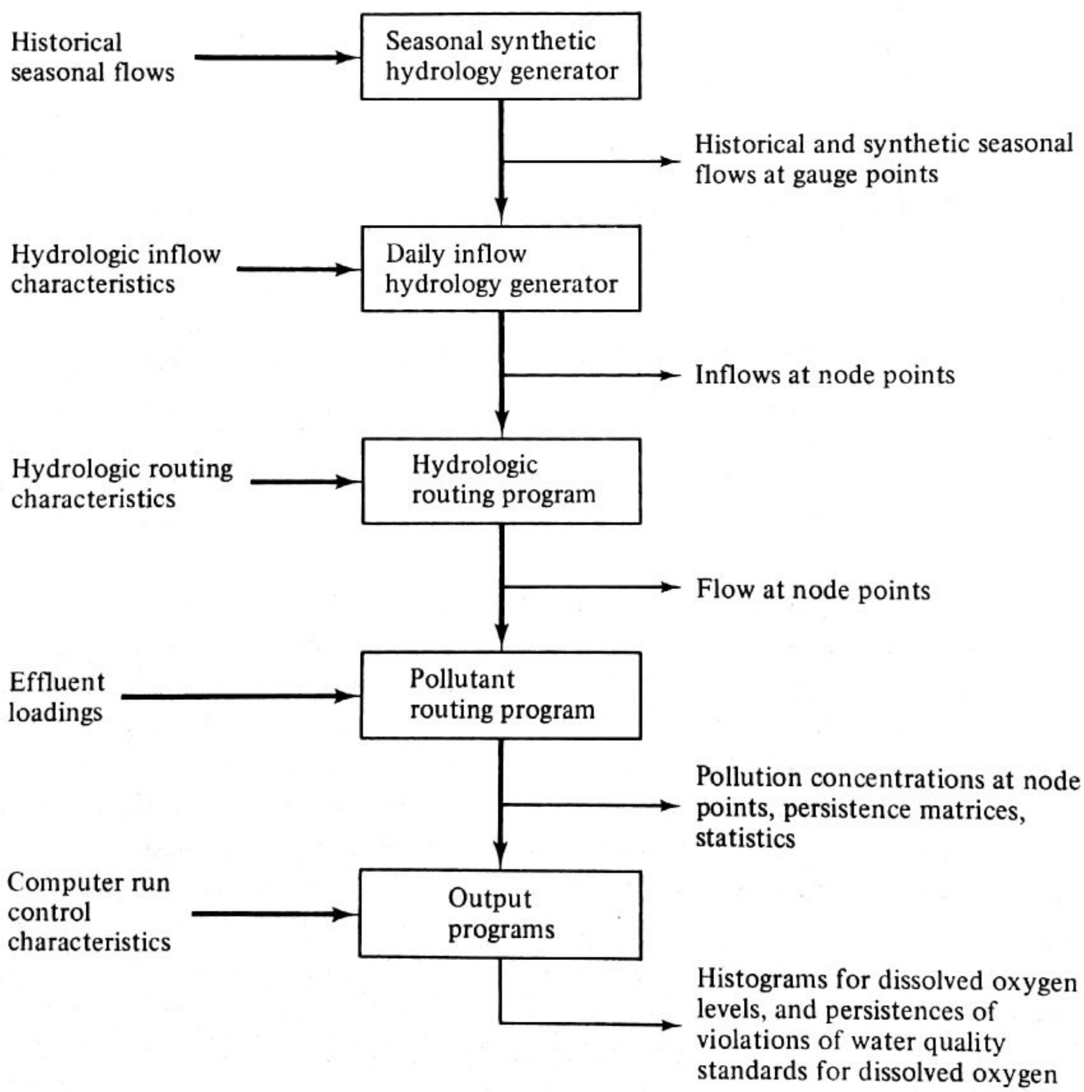

FIGURE 3-4
Structure of simulation model.

3-5.1 Synthetic Seasonal Flow Generator

Streamflow data available for most places are for a relatively short period, and it was no exception in the present study. Accordingly, it was necessary to generate average seasonal flows for the different gauging stations to supplement the existing data available, and thus obtain a greater variety of flow conditions for use in the model. In order to ensure that the significant characteristics of the historical flows were retained in the synthetically generated flows, the following steps were taken:

1 The major statistical parameters of historical flows, including the long-term means, were maintained.
2 The statistical relationships of the following aspects were retained:
  a The flows at different gauges and for each gauge.
  b The flows during adjacent time periods.

Essentially the synthetic seasonal flow generator utilized statistics of historical or actual data to create a river flow record of any duration that is statistically similar to the historical data. Generation of the synthetic sequences provided long-term records of extreme conditions of low and high river flows that do not normally exist in historical records, but are necessary for analyzing critical conditions.

3-5.2 Daily Inflow Hydrology Generator

The Hydrology Generator is used to simulate the flows entering the river channel at different node points. The seasonal flows obtained at different gauging sites by the Synthetic Flow Generator are divided into daily inflows at different node points. An attempt was made to capture the time-varying nature of the overland and groundwater flows entering the river. The gauged instream flow records were examined in detail, and it was decided that a reasonable representation of the inflow can be obtained by considering it to have two parts—a slowly varying base flow that generally reduces with the season, and a storm flow representing the short-term effects of individual storms with their relatively quick peaking character as shown in Fig. 3-5. The characteristics of the historical patterns are maintained in the synthetic generation of daily inflows by identifying several statistically significant relationships. Among these are positive correlation of the base flow to the average seasonal flow, positive correlations of the number of storms to the average seasonal flow, and positive correlations of the inflows at different points from an individual storm. Correct distribution of storm volumes and number of storms were also maintained. An attempt was made to ensure that the total average inflow into the channel over the given season will, after rerouting, reproduce the given average seasonal flow at the various gauging stations.

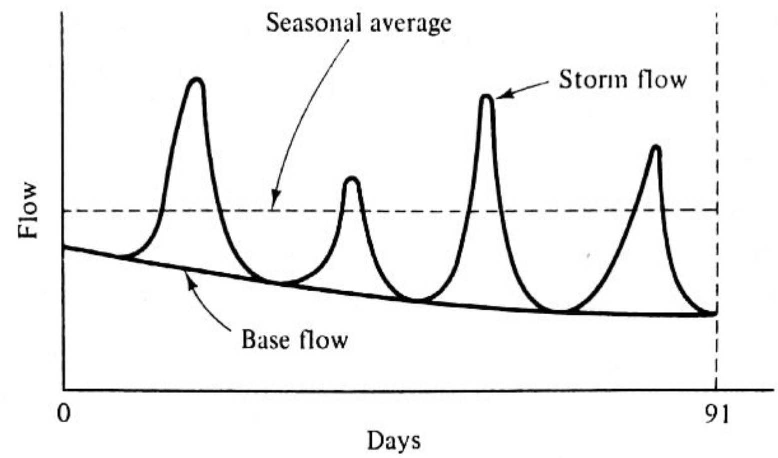

FIGURE 3-5
Daily flow representation for special season.

3-5.3 Hydrologic Routing Program

For routing the flows downstream through the river system, the general approach was to use two separate routing techniques for river reaches and lakes, and to ensure that the effects of changing storage and flow—both in terms of space and time—were accounted for. The river system was separated into a series of short stretches of the order of three to five miles. The Muskingum method was used.

Since the reservoirs on the Saint John river are long and narrow, the flows were considered one-dimensional. The lakes were separated, as in the case of the river, into a series of reaches. The flow at any cross-section was considered to be represented by the mean longitudinal flow at that section. Furthermore, since the reservoirs were primarily developed for hydropower generation, the simulation model was structured in such a way as to reflect the effects of various operating rules on water quality. Specifically, the following conditions were included.[8]

1 Maintaining the reservoir level between specified minimum and maximum water levels.
2 Satisfying a minimum discharge requirement.
3 One of the following four options:
  a Keep elevation as high as possible.
  b Maintain storage at a prescribed fixed value.
  c Increase flow when downstream quality drops below a preset value, but otherwise keep elevation as high as possible.
  d Maintain a prescribed fixed power output (flow times head), but otherwise keep elevation as high as possible.

Daily flows at each point through the system were obtained from this segment of the program. Velocities were computed from flows by using the Manning's formula in the case of the river, and directly by using cross-sectional areas at the location for the lakes.

3-5.4 Pollutant Routing

Pollutant routing differs in one fundamental aspect from the hydrologic routing. Changes in river flow move downstream with a speed equal to wave celerity and at velocities considerably higher than the actual speed of water. In contrast, pollutants are transported downstream by the water itself at the average flow velocity. Accordingly a separate routing program for the pollutants was necessary, whose formulation was based on transport velocities which were computed directly from the flow rates obtained from the hydrologic routing programme. A "back-step" algorithim was used to account for the difference in the flow rates of the river and the pollutant.

In essence, it worked as follows. By knowing the water velocities, it is possible to back-track an element of water in time and space, that is to determine when the same element of water was at the upstream end of the reach. Interpolating water quality conditions for the present and previous time periods at the upstream end of the stretch, it is possible to estimate pollutant and DO concentrations of the element when it passed the upstream end. Knowing the time of travel, changes in BOD and DO can then be calculated by using the modified Streeter–Phelps equation. These concentrations can be appropriately modified if any additional inflow or waste discharge enters the node point, by either diluting or supplementing as necessary. This procedure can be repeated sequentially for the entire river system for each time increment, to obtain the water quality conditions at daily intervals throughout the season.

3-5.5 Output Programs

The main output of the simulation model was a description of the violation of various levels of water quality in terms of their frequency, magnitude and persistence. For selected biological species, especially near their threshold levels, it is important to know the length of time the DO level stays below the specified value.

3-6 RESULTS OF MODEL USE

The programming model was used in the preliminary analysis to identify the possible consequences of certain policy decisions. It was quite evident that extremely poor water quality conditions existed at the Grand Falls Reservoir (Fig. 3-1) due to high effluent discharge of the Edmundston–Madawaska pulp and paper complex (in excess of 800,000 lb/d of BOD), and relatively low assimiliative capacity of the Grand Falls reservoir. The model showed that just to bring the Grand Falls head pond to an aerobic state, the treatment levels necessary to achieve a very high level of effluent removal would far exceed anything previously installed for such plants. Hence, it was considered somewhat unlikely that the DO level in excess of zero ppm could be justified. Consequently this level, zero ppm was maintained in the reservoir, but for the rest of the river it was allowed to vary between 0 and 5 ppm, that is between limiting conditions of becoming septic and the short-term threshold level for the migratory species of cold-water fish. The model further indicated that for the low flows being considered for critical conditions, the DO levels all along the river itself would exceed 5 ppm—provided the Grand Falls head pond was kept aerobic. However, DO levels would still be somewhat lower than 5 ppm in certain parts of the tributaries.

For the steady state conditions of all effluents being discharged at the maximum capacity, and the general condition of seven-day, ten-year low flow throughout, three different DO levels—0, 4 and 5 ppm—were prescribed (excluding the Grand Falls reservoir as noted earlier) to determine the changes necessary for costs of effluent treatments. The objective was to minimize treatment costs throughout the basin for all cases for the following conditions:[7]

1 Reference run—based on assuming no existing treatment and selecting complete freedom of choice in treatment levels to meet the given DO standards.
2 Existing situation—based on assuming that all existing treatment remains in operation and optimally selecting additional treatment to meet the prescribed DO standards.
3 Primary treatment everywhere—based on the policy of having at least primary treatment at each effluent source and optimally selecting additional treatment to meet the prescribed DO standards.
4 Primary treatment everywhere with low flow augmented by 20 percent—as for (3) except that hydrologic flows have been increased everywhere by 20 percent.

Results of these analyses are shown in Table 3-1. The basic cost indicates the expenditure necessary to meet the stated conditions, and then the minimum additional cost is indicated to meet the specified DO levels.

Some interesting conclusions can be drawn from Table 3-1. For example, starting with zero treatment, a DO level of 5 ppm can be maintained at a minimum annual expenditure of $3.266 million. However, for the existing treatment systems, a sum of $2.091 million has already been spent, and another additional $3.156 million is necessary to maintain the DO at the specified 5 ppm level, for a total expenditure of $5.247 million. In other words, the resource allocation process to improve water quality conditions has thus far been less than satisfactory. An additional $2.021 million have to be spent, over the amount necessary to maintain the DO at 5 ppm, compared to starting from zero treatment. One can thus conclude that those who contributed most to water pollution have so far installed little control measure to reduce effluent discharges.

If primary treatment is to be provided everywhere, a condition which is often necessary due to administrative or institutional reasons and health considerations, the increased expenditure necessary over zero treatment is of the order of 30 percent, that is an increase from $3.266 million to $4.163 million. It should be

Table 3-1 SUMMARY OF ANNUAL COSTS FOR PRELIMINARY SCREENING RUNS

		Cost in million $ General standard (ppm)		
Condition	**Basic cost**	**5**	**4**	**0**
Starting with zero treatment	0.0	3.226 3.226	2.948 2.948	2.636 2.636
Starting with existing treatment	2.091	3.156 5.247	2.895 4.986	2.611 4.702
At least primary treatment everywhere	1.712	2.451 4.163	2.333 4.045	2.229 3.941
At least primary treatment everywhere (flow increased by 20 percent)	1.712	2.200 3.912	2.089 3.801	2.089 3.801

noted that even this option is more efficient in terms of cost, when compared with actual expenditures up to the present time.

Finally, the model indicated that the combination of primary treatment everywhere and flow augmentation by 20 percent, would provide a net saving of $251,000, compared to the third option. Such analyses of economic trade-offs are necessary for planning purposes.

Another important conclusion that comes out from Table 3-1 is the fact that the cost of improving water quality from 0 ppm to 5 ppm of DO in every case is reasonable, varying from a minimum of $111,000 in the case of the fourth option to a maximum of $690,000 for the first. If primary treatment is provided everywhere, the additional annual cost is only $222,000. This can be better appreciated from Fig. 3-6, which shows a schematic representation of the DO profile for the entire river basin. The Edmundston–Madawaska pulp and paper complex has a dominating effect on the river water quality due to its large effluent discharge. Hence, if the required treatment facilities can be provided upstream of the Grand Falls reservoir, water quality in the main stem can effectively be satisfied. Changes in costs are reflected by the treatments necessary for municipal wastes and food and starch-processing plants located near the Aroostook and Presquile rivers.

Both the programming and simulation models were used to analyze the water quality effects to maintain a DO level of 5 ppm at all points, except for the Grand Falls reservoir where the minimum DO level was specified to be 0 ppm. It was assumed that all existing treatment plants were operational, and that primary treatment was provided by all polluters. No low flow augmentation alternative was considered.

The programming model was used to determine the optimum size of the waste treatment facilities necessary above and beyond the existing or primary treatment levels. These levels were then defined at each of the effluent sources,

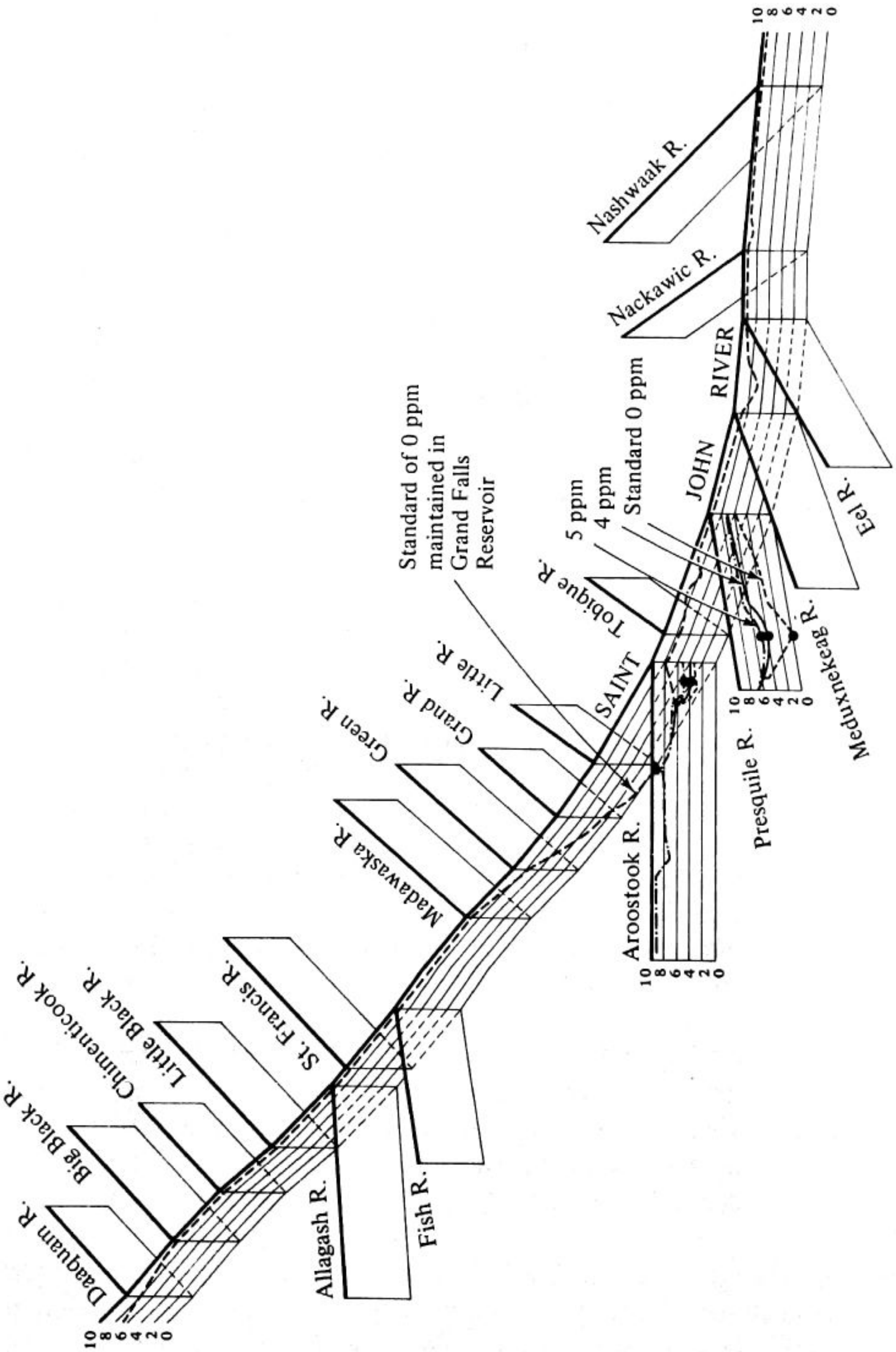

FIGURE 3-6
A schematic representation of the DO profile of the Saint John River Basin.[8]

and were input to the simulation model. The simulation model was run through twenty-five consecutive low flow seasons of synthetic daily flows; this was selected to be the three-month summer to early autumn period, when flows are low and DO levels are reduced due to high effluent loadings from food-processing plants and rapid waste decomposition. The results from both the models are shown in terms of water quality at different node points in Fig. 3-7, by means of a continuous line for the programming model and by a histogram for the simulation model. Since outputs from the simulation model were massive, they have to be summarized in appropriate forms for planning purposes. For example, for the Grand Falls reservoir, DO levels as a function of location and time were displayed as a three-dimensional isometric plot as shown in Fig. 3-8.

It is important here to review the results obtained from both the models since they differ in some important aspects. The use of the simulation model provided the information that the seven-day, ten-year low flow occurs for about only two percent of the time. In other words, if both the models are accurate, the DO levels obtained from the programming model should be exceeded 98 percent of the time in the simulation model. This, however, is not correct, as can be seen from Fig. 3-8. Only near the junction of the Saint John and Grand Rivers, the DO levels from the programming model are lower in value as compared to the simulation model. From the limited data available on the DO levels, it appears that the results from the simulation model are closer to the reality. Table 3-2 shows a summary of the results for five representative points. In this table, the potential range of simulated DO levels between 0 and 9 ppm has been divided into ten categories, and the percentage of time for which the simulated DO level is within each category has been indicated. The DO levels obtained from the programming model are also shown. The last column shows the number of violations of the standard exceeding seven days of duration over twenty-five years. There should not be more than two or three violations if the seven-day, ten-year requirement is satisfied.[8] While there are some expected agreements for the first two locations, the programming model consistently gave much higher DO levels for the rest.

One can then thus conclude, that the use of the steady-state programming model did not capture the extreme low-flow conditions. This can be explained by the fact that for rivers like the Saint John, reduced water quality can occur at higher than low flow levels.

3-7 MODEL LIMITATIONS

It is important at this point to consider the limitations of the models developed and also some of the assumptions that were necessary for their development. It is quite clear that at the present state-of-the-art, comprehensive mathematical models that can answer all pertinent questions cannot be developed for technical, methodological and economic limitations. Lack of adequate data often hinders the model building and validation processes. Furthermore, any decision-making process has political, social, legal and institutional dimensions,[9] which cannot be

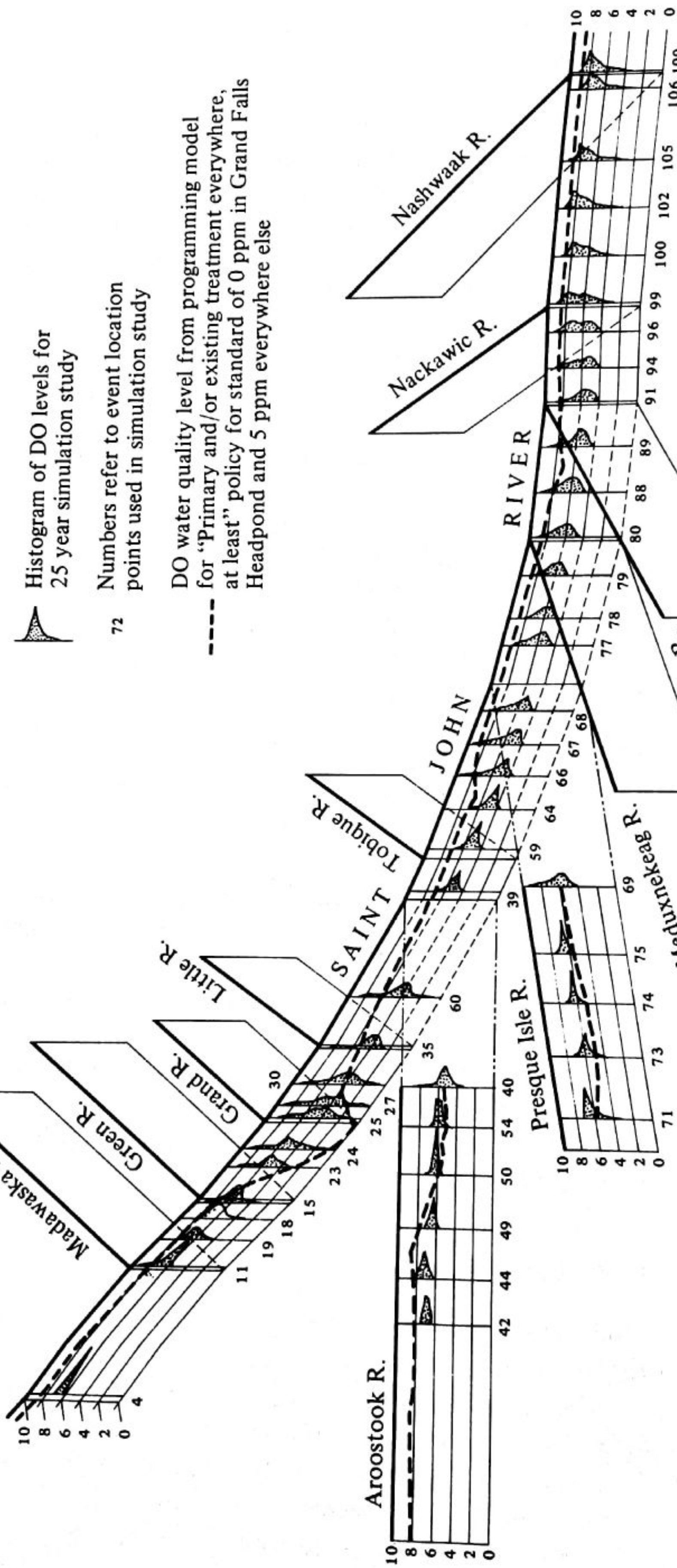

FIGURE 3-7
Water Quality at different node points indicated by programming and simulation models.[8]

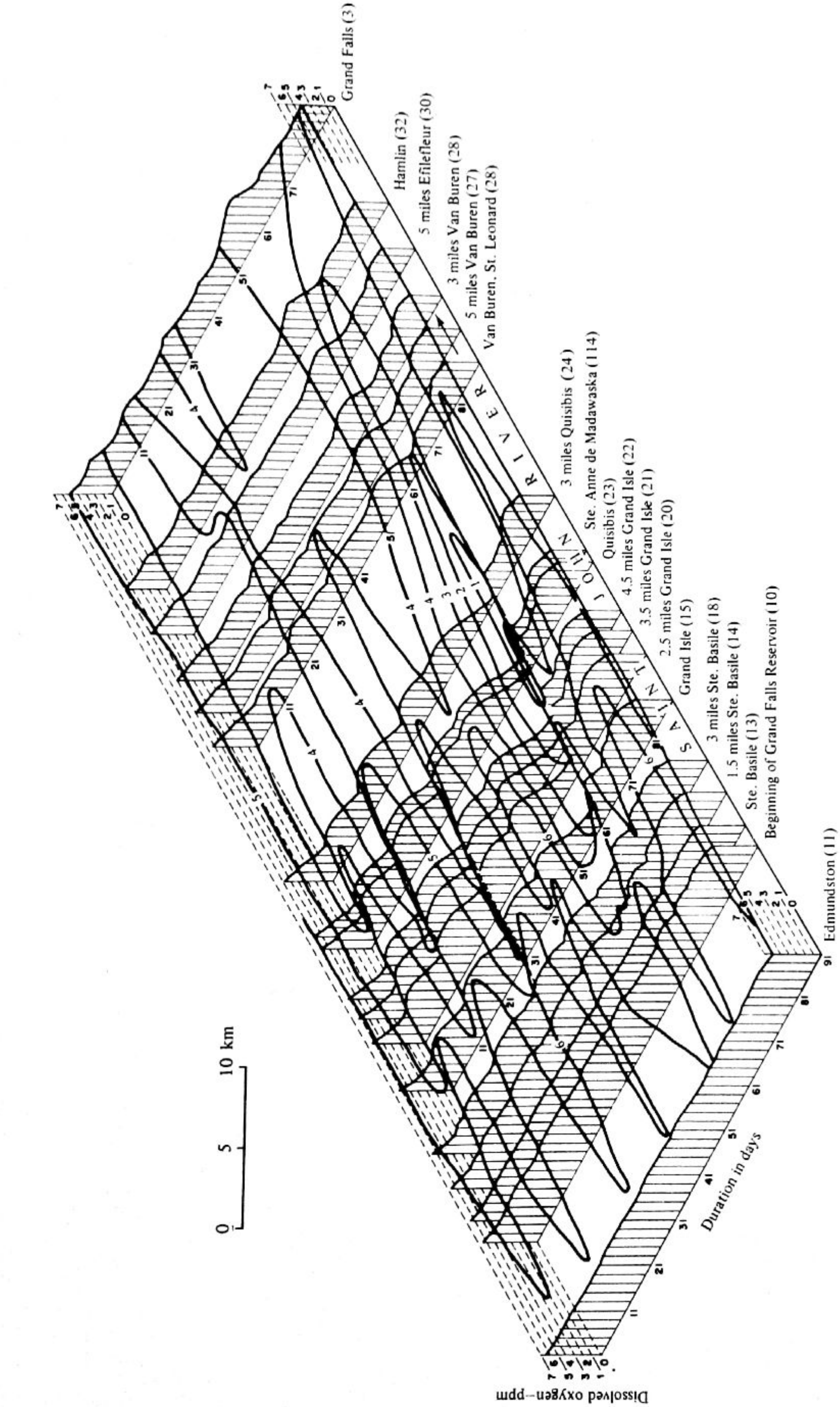

FIGURE 3-8
DO levels as a function of location and time.[8]

Table 3-2 COMPARISON OF DO LEVELS OBTAINED FROM THE TWO MODELS

Location	Range of dissolved oxygen (ppm) 0	0–1	1–2	2–3	3–4	4–5	5–6	6–7	7–8	8–9	DO from LP model	Number of violations of the standard exceeding 7 days duration
	(% of time DO levels lie within range)											
4 miles downstream from Edmundston	0	0.6	0.4	1.4	3.7	8.8	28.1	53.8	1.5	1.6	3.7	None
Quisibis	2.5	2.9	2.5	4.1	7.1	17.8	34.7	25.4	2.0	1.2	0.08	Not calculated
Bellefleur–Grand Falls head pond	0	0.2	2.0	5.4	24.7	36.5	24.2	3.5	2.9	1.7	5.16	None
Beechwood head pond	0.1	0.0	0.1	0.2	0.3	55.9	35.1	3.1	3.7	1.7	8.93	54
Mactaquac head pond	0.1	0.2	0.0	0.1	0.2	3.5	14.9	28.3	37.2	13.9	8.94	11

Results to the left of the heavy line fail to meet the preselected standard.

directly considered by the models. For strictly operational models, the socio–economic–political requirements are not so stringent, and accordingly it is no surprise to find that most of the existing water resources models are used primarily for operational reasons, and not for decision-making purposes. Similarly, for economic considerations, the emphasis of the models developed tends to be on cost minimization (as in the case of the Saint John) rather than benefit maximization. Existing economic techniques available for the quantification of benefits accruing from specific projects leave much to be desired, and for the evaluation of intangible benefits the situation is even worse.[6]

Within this general context it is useful to provide some information on the limitations of the models developed so that they do not mean all things to all men. Some of the important limitations are briefly mentioned herein.

The models developed for the Saint John River do not provide direct monetary benefits associated with achieving a desired level of water quality for the use of the river for recreation, fisheries, water supply or any other use. The linear programming model, however, can be used to assess or determine benefits in the form of treatment plant cost reductions due to different management strategies.

The models consider BOD and DO as surrogates of water quality because decay relationships for oxygen-consuming wastes are the only ones available, and do not treat other toxic pollutants such as pesticides, heavy metals, etc. The simulation model does have the capability to consider conservative pollutants which do not degrade appreciably in an aquatic environment.

The models assume instantaneous dispersion of pollutants from waste sources into the river. Observation of pollution sources indicates that considerable time is often required for complete dispersion to occur depending on flow conditions, river, geometry, and other factors.

Only conventional municipal and industrial pollution abatement methods are considered in the models. Transfer of wastes by piping, instream aeration, and flow augmentation by storage reservoirs are other possibilities for treatment that must be considered outside the models.

Greater improvements are required in developing the cost curves for pollution removal at each treatment site. In some cases relatively complex curves had to be derived from only three or four treatment points, and thus their accuracies have to be questioned.

On the Saint John River one large pulp and paper mill upstream is the major polluter. Because this waste source effectively "drives" the model, all the results are particularly sensitive to small changes in waste load or treatment costs at this point.

There are also some difficulties associated with obtaining direct links to the models for aquatic ecology (in the form of "fish kills" at low DO levels), and regional economics (in the form of employment and income effects of high river water quality standards near the large industrial polluters).

The estuary of the Saint John River from Oromocto to Saint John is not catered for by the models since it is extremely difficult, if not impossible, to adequately define the effect of tidal fluctuations on water quality.

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4

MODEL OF THE SAINT JOHN RIVER, UNITED STATES*

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4-1 SOME ISSUES ASSOCIATED WITH THE PLANNING PROCESS AND MODEL DEVELOPMENT

In the process of planning investments in water pollution control for a river basin, a great deal of analysis is needed to examine many pertinent facets of the problem. There are interacting physical, biological, economic, social and political subsystems in a basin. Each subsystem can be complex and involve time-consuming study. Models are often used to ensure and facilitate consistent analysis. However, there is no one prescribed approach for model building.

A model can be comprehensive enough to include all subsystems. A model of this type in general will be coarse and simplified for computational convenience and, therefore, will seldom provide sufficient details of the system's response. To remedy the shortcoming, other models are used to generate refined information. Some of these models will be subsystem specific while others will include more than one subsystem. They can be designed to be used conjunctively in that the output of some models may be the input to others. For example, a highly aggregated optimization model generates plans which can be further analyzed by

* This chapter is drawn largely from deLucia and Chi,[1] deLucia, et al.,[2] and deLucia et al.[3]

a detailed simulation model. In addition, models can be designed for feedback iteration. The suitability of a single model or models, and how they should be structured depends upon the analyst's experience, preference, and the particular system under consideration. In practice, one would start with relatively simple but robust model(s) and refine toward more reliable model(s).

In the broadest sense, the development of a water quality model is dependent on the characteristics of the problem being examined. This chapter will not provide an exhaustive list of the issues which arise in this process, but will present four areas of concern which influence the development actively. These are:

1 The relationship of planning activity to the problems of variable specification.
2 The nature of the institutional/client setting.
3 The type and quality of data available to those developing the model.
4 The availability of resources for model development and implementation.

In the first area one is faced primarily with the issue of the specificity with which the model's endogenous variables must be defined. For example, although a river basin may have an important nonpoint waste load, this pollution input can be accounted for in the model as an equivalent point source. Another example might be a case in which nontreatment alternatives such as flow augmentation or bypass piping and regional treatment are under consideration, but they need not be included in the model specificiation. Rather, such alternatives are dealt with by running the model under an alternative flow regime (to account for flow augmentation) or by developing treatment cost functions which reflect transport costs (to account for piping and regional treatment). There are other issues, of course, having to do with the nature of the pollution problem (e.g., the importance of nutrients) and the nature of the preference function (e.g., the problem of equity versus efficiency).

The second area of concern is associated with the institutional or client setting of the planning problem. Here the issues influence decisions related to the structure of the mathematical model and its level of documentation. Is the model for general use, or is it to be employed in the analysis of a specific problem? Is the planning problem strictly a water quality management problem, or is the model to be part of a larger planning effort and will it have to be made compatible with other quantitative models?

The more general the model use will be, the stronger the argument for detailed documentation and the inclusion of a variable structure that explicitly deals with many of the issues suggested in the area discussed above. On the other hand, if the model is being developed for a very specific planning problem or is strictly a water quality management model as opposed to a broader water resources management model, then both the structure and ultimate documentation of the model can be limited. Strict water quality management problems argue for less complex models. The availability of complementary models suggests that the model's variable structure may be limited and these variables or characteristics may be considered in a second or complementary model.

The last two areas (availability of information and resources) jointly influence both the level of complexity of model variable structure and ultimately its implementing computer code. The more complex the model in general, the more the data, computing and professional resources necessary. In some cases significant prior information allows the use of a more complex model than would be warranted by the study budget. Consider, for example, the situation in which there is adequate existing field data to allow extensive calibration of parameters. Additional data from the present study could then be developed specifically for the ultimate verification exercise.

The development of the Saint John River model, described in subsequent sections, provides an example in which many of the issues in the above four areas arise. As the model is presented below, we will highlight the issues we considered most important and discuss the important features of the discussion which led to the development of the model in its present form.

4-1.1 Institutional and Physical Setting

The model discussed in this chapter was developed for initial use in the Saint John River Basin. However, the characteristics of the model reflect not just the particulars of the specific Saint John planning problem for which it is being used; in addition, its characteristics reflect the planning requirements and the constraints implicit in the 1972 Amendments to the Federal Water Pollution Control Act (Public Law 92–500). The model was developed and was documented such that it would be useful in examining basin or area-wide planning efforts required or suggested by various sections of the Amendments.

The model was developed for use in a planning study conducted by a team of consultants, under a U.S. Environmental Protection Agency sponsored contract with the Northern Maine Regional Planning Commission in a joint venture of E. C. Jordan Co. Inc., and Meta Systems Inc.[4]

The Northern Maine Regional Planning Commission in cooperation with the state agency is undertaking aspects of water quality planning mandated by the Amendments. The initial model users were members of the Meta Systems' staff who have been involved in the development of this and other models. However, the model development was funded by the Environmental Protection Agency and has been documented at a level sufficient to allow ready application by other users.

The planning effort which sponsored the development of the present model(s) focused on the Maine portions of the basin and international boundary waters between New Brunswick and Maine. Another planning study was carried out by a Canadian team,[5] and a series of both formal and informal exchanges of information occurred. These exchanges included the sharing of unpublished water quality sampling data. In addition, the United States planning study and the development of the Saint John model benefited significantly from the results of an earlier methodological study sponsored by the Department of the Environment of Canada.[6] This earlier study used the Saint John River Basin as a case study and included the development of two models: first, a prescriptive model that identified optimal treatment configurations within a steady-state context, and second, a time-variable simulation model. Both of these models from this earlier study of the Saint John River Basin were available for use.

Prior to discussing particular characteristics of the model used in the U.S. Environmental Protection Agency funded study, it is useful to describe some of the particulars of the 1972 Amendments under whose mandate the planning is being done, the nature of the basin for which the planning was done, and complementary planning efforts that have been done.

The 1972 Amendments call for the attainment of "secondary" treatment or better by 1977 for publicly owned treatment works and best-practicable control technology for industries provided that these levels of treatment/control will result in meeting water quality standards. In those cases where these levels of technology are insufficient to meet the standards, more stringent technology would be called for. The Amendments further state that when it is necessary to move beyond secondary and best-practicable, the targeted reductions should reflect economic considerations.

The Amendments call for more stringent 1983 goals, namely the application of best-practicable by publicly owned treatment works and best-available technology economically achievable by industries.

The two time-period treatment targets explicit in the Amendments are incorporated into this model which considers both capacity expansion and the level of treatment.

As for the nature of the basin for which the planning was done, the Saint John River Basin is international in nature; it is located in northern Maine and the adjacent areas of Quebec and New Brunswick between the watersheds of the Saint Lawrence River to the north and the Penobscot River to the south. It extends from a point on the Maine–Quebec international boundary, about 70 miles south-east of Quebec City to the Bay of Fundy (see Fig. 4-1). The total drainage area is 21,300 square miles of which 36 percent or 7,600 square miles lies in northern Maine, 51 percent or 10,950 square miles in New Brunswick and the remaining 13 percent or 2,750 square miles in Quebec.

The river originates in Little Saint John Lake, located in the southwestern corner of the basin on the international boundary between Quebec and Maine at an elevation of 1,580 ft above mean sea level. It flows in a northeasterly direction from Little Saint John Lake until it reaches the vicinity of Edmundston, where it gradually changes to a southeasterly course to Grand Falls, New Brunswick, then southerly to the Bay of Fundy. The mainstem of the river is approximately 435 miles long. The boundary between Canada and the United States follows the mainstem of the river for about 30 miles from its point of origin in Little Saint John Lake, and for approximately 75 miles from Saint Francis, Maine, to a point about 7.5 miles upstream of Grand Falls, New Brunswick. The Saint Francis River, a tributary of the Saint John, forms the international boundary for an

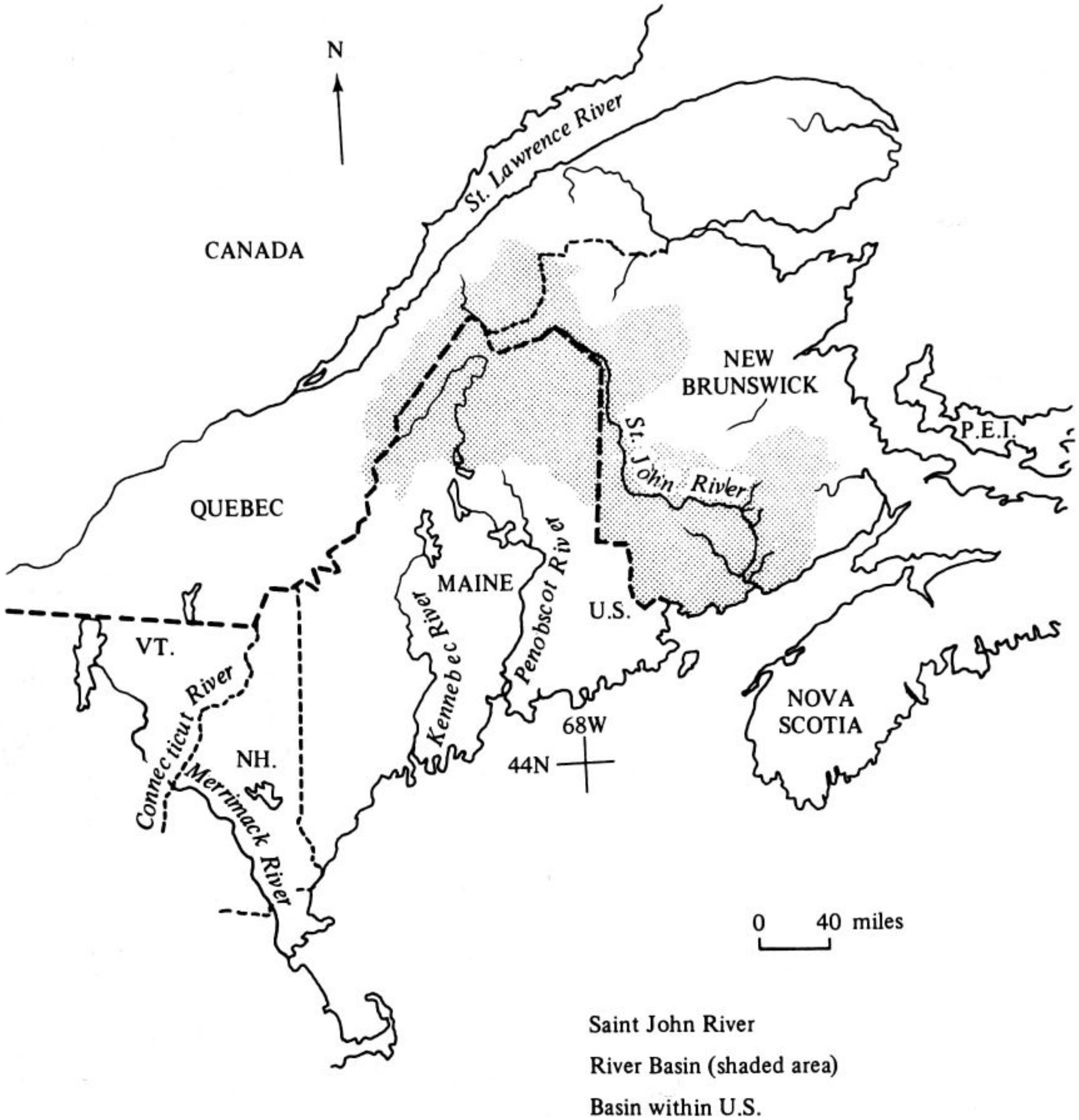

FIGURE 4-1
Saint John River Basin location map.

additional length of about 27 miles north-west of Saint Francis, Maine. The river slopes gradually, decreasing from about 8 ft/mile near the head waters to 3 ft/mile in the vicinity of Grand Falls and 2 ft/mile in the Canadian reach above Fredericton. Within the United States the major tributaries of the Saint John River are the Allagash, the Fish, the Aroostook, the Meduxnekeag and the Prestile Stream.

The watershed is sparsely populated with large areas of timberland and agricultural land both of which contribute significant nonpoint pollution loads. There are, however, a number of large volumes of either untreated or partially treated industrial (pulp and paper, potato processing) and domestic effluents which have created severe water quality problems.

Tables 4-1 and 4-2 provide a tabular comparison between the point source and the non-point source inputs to river regions for the seven-day ten-year design low flow condition and for a typical winter flow. Only BOD, nitrogen and phosphorus values are summarized in the tables. The values presented suggest quite strongly the important role of the nonpoint sources with respect to nitrogen and phosphorus.

Table 4-1 TABULAR COMPARISON OF POINT AND NONPOINT SOURCES FOR 7-DAY AND 10 YEAR LOW FLOW REGIME

	BOD		Total N		Total P	
Region	Point Source (lb/d)	Nonpoint Source (lb/d)	Point Source (lb/d)	Nonpoint Source (lb/d)	Point Source (lb/d)	Nonpoint Source (lb/d)
Grand Falls	430,967	22,932	17,192	2,622	399	591
Aroostook	156,220	2,809	2,362	752	604	44
Prestile	551	445	95	75	33	4
Meduxnekeag	15,636	176	284	99	103	55

Table 4-2 TABULAR COMPARISON OF POINT AND NONPOINT SOURCES FOR JANUARY-MARCH FLOW REGIME

	BOD		Total N		Total P	
Region	Point Source (lb/d)	Nonpoint Source (lb/d)	Point Source (lb/d)	Nonpoint Source (lb/d)	Point Source (lb/d)	Nonpoint Source (lb/d)
Grand Falls	430,967	124,415	17,192	3,547	399	681
Aroostook	156,220	5,010	2,362	6,866	604	421
Prestile	551	1,583	95	675	33	40
Meduxnekeag	15,636	4,006	284	701	103	201

4-2 THE MODEL

4-2.1 Model(s) Overview

The model(s) developed actually comprise a set of complementary models. As has been described elsewhere[7] there is often no unique mathematical model for a particular problem. Rather a set of linked, sometimes hierarchical models, is most useful. In this study we have developed both a descriptive and a prescriptive stream/river model and a descriptive lake model. Only the stream/river models are discussed in this chapter.

A descriptive model relates pollutant loads with stream quality indicators. This model can be run independently as a steady-state, quality simulation model. If requested by appropriate input data statements, it also can produce the matrix of descriptive relationships which become part of a prescriptive nonlinear (separable) programming model which minimizes the present value of costs over a multiple period time horizon. Capital costs are distinguished from operating and maintenance costs and appropriate relative multipliers may be introduced when useful. The impact of reimbursement funds (both federal and state) and cost sharing of joint municipal industrial treatment works may be included in this manner. Provision is made in the model for inclusion of various equity and budgetary constraints.

The physical systems relationships are based on *stream* hydraulics, and kinetics with some modification for reservoirs and lakes which may be integral to the system. The dissolved oxygen/biochemical oxygen demand (DO–BOD) regime is modeled using a modification of the empirical Streeter–Phelps relationship.[8] The carbonaceous and nitrogenous BOD loads are handled individually (the latter including an appropriate lag effect). The combination of long travel time and pulp and paper industry loads present in the Saint John River, with their resultant significant nitrogenous loads, argued for this inclusion. Point and nonurban nonpoint pollution sources are significant. They are explicitly included with the options of including cost of controls for both point and nonpoint sources, since source control options differ between sources and between point and nonurban sources. The kinetic characteristics of the major pollutants (pulp and paper waste, potato processing waste and municipal wastes) can vary considerably. The inclusion of polluter-specific kinetics allows explicit consideration of different control levels for different polluters. The nutrient regime is examined considering both nitrogen and phosphorus as total species and therefore essentially as conservative elements. The analysis of this model is based on the assumption of steady-state regimes (although variation of the hydrologic regime is easily accomplished with alternative model runs).

The structuring of a prescriptive water quality management model which incorporates relationships between pollution loads and stream quality based on "steady-state" solutions of stream and/or estuary equations is not new (e.g., Thomann;[9] Loucks, et al;[10] and Sigvaldason, et al.[11]). The uniqueness of the prescriptive model presented herein lies in: (1) its structure reflecting planning problems arising from the 1972 Amendments; (2) the use of a synthetic pollutant to express the cost removal function; and (3) a series of heuristics adopted to overcome possible nonconvex difficulties.

The first of these will be obvious from the structure presented below. The second and third are somewhat interconnected.

4-2.2 The Model

The Basin is divided into activity points or reaches labeled j, where $j = 1, \ldots, J$, and quality points or reaches i, where $i = 1, \ldots, I$. For particular i and j, the reach or point may be identical.

The descriptive approach entails the development of a set of transfer functions, T_{ji}, relating the quality at point i with the pollution at point or reach j. T_{ji} is a vector transfer coefficient in that this relationship is developed for each of the l pollutants and quality indicator couples we are examining (DO, BOD, N, P). Letting T_{jil} represent the scalar element for pollutant l in the vector T_{ji}, we note that T_{jil} is a function of geometry, flow, kinetics, etc. T_{jil} is a function of system physics.

The descriptive river model uses stream hydraulics—i.e., overwhelmingly advective, little or no dispersion, and based on steady-state analysis. The pollutants (instream) that are discussed below will be then treated with zero or first-order kinetics dependent on their conservative or nonconservative nature.

The calculations of the transfer coefficient for the nonconservative organic BOD and DO interrelationships is programmed using variations of the Streeter–Phelps[8] equations as follows:

Extensions to the Streeter–Phelps given by Camp[12] and Dobbins[13] are for a BOD level L,

$$\frac{dL}{dT} = -(k_1 + k_3)L \tag{4-1}$$

where the terms k_1 and k_3 represent constants respectively for deoxygenation and sedimentation, and

$$\frac{dD}{dt} = k_1 L - k_2 D - A \tag{4-2}$$

where k_2 is the oxygen reaearation rate, D is the oxygen deficit, or saturation concentration less actual concentration and A is the photosynthetic production rate. Integrating Eq. (4-1) for L_t, the concentration at a point corresponding to time t downstream from an initial concentration L_0 is

$$L_t = L_o e^{-(k_1 + k_3)t} \tag{4-3}$$

Utilizing Eq. (4-3) it is possible to integrate Eq. (4-2) for D_t the oxygen deficit at any time t downstream from an initial deficit D_0, thus

$$D_t = \frac{k_1}{k_2 - (k_1 + k_3)} L_0 - [e^{-(k_1+k_3)t} - e^{-k_2 t}] - \frac{k_1}{k_2}(1 - e^{-k_2 t}) + D_0 e^{-k_2 t} \tag{4-4}$$

Equation (4-4) thus provides the means for determining transfer coefficients for BOD and dissolved oxygen in the models.

If F_{ji} denotes the flow between upstream activity point j and downstream point i, then stream velocity and depth are functions of flow—$V(F_{ji})$, $H(F_{ji})$. Velocity and depth in turn are determinants of the reaeration.

Using the Thackston and Krenkel[14] formulation, the reaeration coefficient may be written as

$$k_2 = \partial\left[1 + \left(\frac{V(F_{ji})}{\sqrt{gH(F_{ji})}}\right)^{1/2}\right]\sqrt{\frac{S}{H(F_{ji})}} \tag{4-5}$$

Where ∂ is a unit conversion factor, g is acceleration due to gravity and S is the channel friction slope. Using the above expression in the river segments and the O'Connor and Dobbins[15] formulation in impoundments, it is now possible to define a set of transfer functions or coefficients which express the change in oxygen concentration (Q_i) at quality point i in response to a unit load of BOD carbonaceous ($L_j = 1$) introduced into the stream at waste point j (point source)

$$T_{ji}(F_{ji}) = \frac{dQ_i}{dL_j} \tag{4-6}$$

The coefficient T_{ji} describes the interrelation between streamflow and waste input for a particular flow. If M_{ji} is the distance between points j and i, then the time of travel is $M_{ji}/V(F_{ji})$: thus

$$T_{ji}(F_{ji}) = \frac{k_1}{k_2(F_{ji}) - (k_1 + k_3)} [e^{-(k_1 + k_3)M_{ji}/V(F_{ji})} - e^{-k_2(F_{ji})M_{ji}/V(F_{ji})}] \tag{4-7}$$

Since oxygen concentration is also coupled with nitrogenous BOD, a similar transfer coefficient is developed representing the nitrogenous activity.

Additional transfer coefficients can be developed for the other constituents. If nitrogen and phosphorus are treated as total species or conservative substances, a number k of pollutants to quality indicator couples, l_i, $i = 1, k$, are obtained for BOD carbonaceous/DO, BOD nitrogenous/DO, DO/DO nitrogen/nitrogen, and phosphorus/phosphorus. DO is not a pollutant, but a transfer coefficient must be developed for this couple since upstream DO influences downstream DO as indicated in Eq. (4-4).

The T_{ji} describe the transfer of a pollutant concentration lumped at point j to point i downstream. We have also included in the model provision for pollutant loadings arising by runoff from the area drained by each reach. By assuming a constant increment of flow/length and unit uniform loading along the reach, we can derive a transfer coefficient for the propagation downstream to reach i of a pollutant load linearly distributed along reach j. In contradistinction to the situation discussed above, the flow in such a reach is not constant, i.e., F_{ji} does not exist; rather, one has an F_j and an F_i such that

$$F_i = F_j + F_j^1 M_{ji} \tag{4-8}$$

where F is the increment of flow/length in reach j. Then

$$\tau_{ji} = \frac{k_1 F_j^1 V(F_1, F_j)}{k_2(k_1 + k_3)F_i} [1 - e^{-k_2 M_{ji}/V(F_i, F_j)}]$$
$$+ \frac{k_1 F_j^1 V(F_i, F_j)}{(k_1 + k_3)[k_2 - (k_1 + k_3)]F_i} [e^{-k_2 M_{ji}/V(F_i, F_j)} - e^{-(k_1 + k_3)M_{ji}/V(F_i, F_j)}] \tag{4-9}$$

To a good approximation the velocity can be regarded as constant

$$V(F_i, F_j) \simeq V(F_j) = V(F_{ji}) \tag{4-10}$$

Clearly, in reaches downstream of j, the pollutant so introduced propagates simply according to the τ_{ji}.

The transfer coefficients then constitute the necessary relationships for descriptive modeling. The downstream quality can be calculated by multiplying the upstream load times the transfer coefficient and summing over the upstream loads and appropriate couples. For example, if Q_i is the DO at point i, LN_j the nitrogenous BOD load at j, LC_j the carbonaceous BOD load at j, then

$$Q_i = \sum_{j \in J} LN_j T_{jil1} + \sum_{j \in J} LC_j T_{jil2} + Q_{j*} T_{jil3} \tag{4-11}$$

J is the set of pollutant points upstream from i, j^* the uppermost point in the system, and l_1 indicates the BOD nitrogenous/DO couple, l_2 the BOD carbonaceous/DO couple and l_3 the DO/DO couple. The prescriptive model will have constraints of the form $Q_i \leq \bar{Q}_i$ where $\bar{Q}_i$ is the quality target (standards).

In addition to the pollutants and quality indicators mentioned above, the model utilizes an additional parameter, biomass potential, that indicates the extent to which substances in the waste stream distort the biological activity of streams beyond natural levels. The biomass potential concept was developed as a part of another study.[7] Certain pollutants (or certain levels and intensities of pollutant loadings) cause irreversible ecosystem disruptions; we do not deal with such regimes. The pollutants in which we are interested are those which have reversible effects at pertinent loadings. These may cause overfeeding and overpopulation, or malnutrition and loss of species diversity; they are responsible for excess productivity which causes only such perturbations to the natural system from which recovery is possible through simple physical and biological activities. For given hydraulic conditions, a measure of the relative extent of excess productivity in a water body is the biomass potential, a parameter that may be estimated from conventional water quality criteria.

(Biomass Potential (BP) $= 1.47\ \mathrm{BOD}_5 + 4.57\ \mathrm{TRN} + \gamma P$, where $\gamma \simeq 30$)

When this parameter is combined with stream size, dilution ratio, and detention time parameters, a measure of the degree of reversible distortion of an aquatic ecosystem is obtained. Biomass potential is an appropriate water quality criterion to be used in the post-1977 period. At that time, standards will be met and DO criteria will no longer be as relevant. As written above, the biomass potential is expressed in terms of an equivalent biochemical oxygen demand. The first two terms are the familiar stoichiometric estimate of ultimate BOD ($NH_4^+ + 2O_2 + H_2O \rightarrow NO_3^- + 2H_3O^+$; $2 \times MW_{O_2}/1 \times MW_N = 64/14 = 4.57$) that follows from the oxidation of nitrogen in the reduced stage (as in ammonia NH_3 to nitrate NO_3^-). The basis of the estimate $\gamma \simeq 30$ is discussed in the Meta Systems report. A predecessor to the concept of biomass potential (or equivalent) as a water quality criterion was contained in Standards of the Royal Commission on Sewage Disposal in Great Britain (Fifth and Eighth Reports of the 1898 Commission).[16] The basis of the standards, which were developed from extensive surveys of small streams draining densely populated areas in the United Kingdom, was that the 65°F, 5-day BOD of the receiving water should be less than 4.0 mg/l during the low-flow warm weather season. This was perhaps the first example of a true BOD or biomass standard for streams; similar standards were adopted subsequently in the United States. Most stream standards, however, related to dissolved oxygen concentration; BOD concentration in streams was regarded as significant only with regard to the effect on dissolved oxygen. Dissolved oxygen reserves are directly important only in connection with safety factors pertaining to maintenance of aerobic stream conditions. That dissolved oxygen per se has little direct correlation with water quality for recreation and aesthetic uses is attested to by the fact that stream standards in many regions of the United States today (e.g., New York and the New England states) do not use the oxygen criterion to distinguish between the good quality waters—Classes A, B, and C.

In addition to serving as a useful water quality criterion especially for the post-1977 period, biomass potential proves a useful synthetic pollutant to utilize in costs of pollutant control expressions. This is discussed in detail below.

4-2.3 The Prescriptive Model

Most early water quality management models utilized the BOD carbonaceous/DO relationship only. Many were single period annual cost models.[17] Cost functions were often expressed as total annual cost as a function of BOD_5 removal efficiency.

Complete treatment plants consist of a series of processes and/or mechanisms acting simultaneously or in series. Although often one or more of the processes may be present primarily for the removal of a particular pollutant (i.e., biological treatment processes for the removal of BOD) treatment plants are in fact "joint product" plants removing BOD, suspended solids, nitrogen, etc., subject to all the difficulties associated with allocating the costs to the joint products. Thus, expressing total costs in terms of a single constituent such as BOD was an artificial mechanism but one that proved useful in models that focused primarily on the BOD_5/DO link.

Programming models which incorporated the nitrogenous BOD in the quality relationships (constraints) while expressing costs in terms of BOD carbonaceous often result in convexity difficulties. The problems stem from the fact that in the range of BOD_C removals for which the cost function is convex—that is, the marginal costs rise with higher degrees of removal—the same is not true for one of the other "joint products," typically BOD_N.

Planning models examining capacity expansion problems face nonconvex issues even when dealing only with the BOD_C/DO couple. These follow directly from the existence of economies of scale in treatment plant costs.

In the model, costs are expressed as a function of biomass potential removal and capacity. The specific planning requirements explicit in the 1972 Amendments allow the use of certain heuristics to examine both degree of treatment and capacity expansion questions in a computationally efficient manner. A convex separable programming algorithm is utilized allowing the use of readily available computer codes.[18]

The model is formulated with two decision periods, one to reflect the decisions necessary to meet the 1977 water quality goals of the 1972 Amendments, the other reflecting the 1983–1985 goals.

The following paragraphs present the cost structure utilized in the model, generalized for the nth point source polluter. From the discussion above we suggest that there exist cost functions of the form shown in Fig. 4-2. For the purposes of discussion, capital cost functions only are shown, and the curves are schematic. Annual operation and maintenance costs are assumed to have the same general shape.

The model requires that, in the neighborhood of a *particular* design capacity flow $X, D_1 \le X \le D_2$ for a specific waste source, the cost function can be approximated in the following form:

$$\text{Capital Costs} = K = \alpha X^{\beta} + f(r) \qquad D_1 \le X \le D_2 \qquad r_1 \le r \tag{4-12}$$

where X is the design capacity, and r the design removal efficiency, and where r_1, D_1 and D_2 are the bounds over which Eq. (4-12) is valid. Note that in general $f(r)$ is convex, and αX^{β} obviously concave for $X > 0$, $\alpha > 0$, and $0 < \beta < 1$.

The rationale for this approach is to select a particular design load (capacity) X, and approximate the concave function with a linear term. This linearization is schematically depicted in Fig. 4-2. The approximation is a good one because the solution, X, must be in the neighborhood of D since the 1972 Amendments prescribe at least secondary treatment for all flows. The size of the "neighborhood of D" in which the solution might fall, and hence the adequacy of

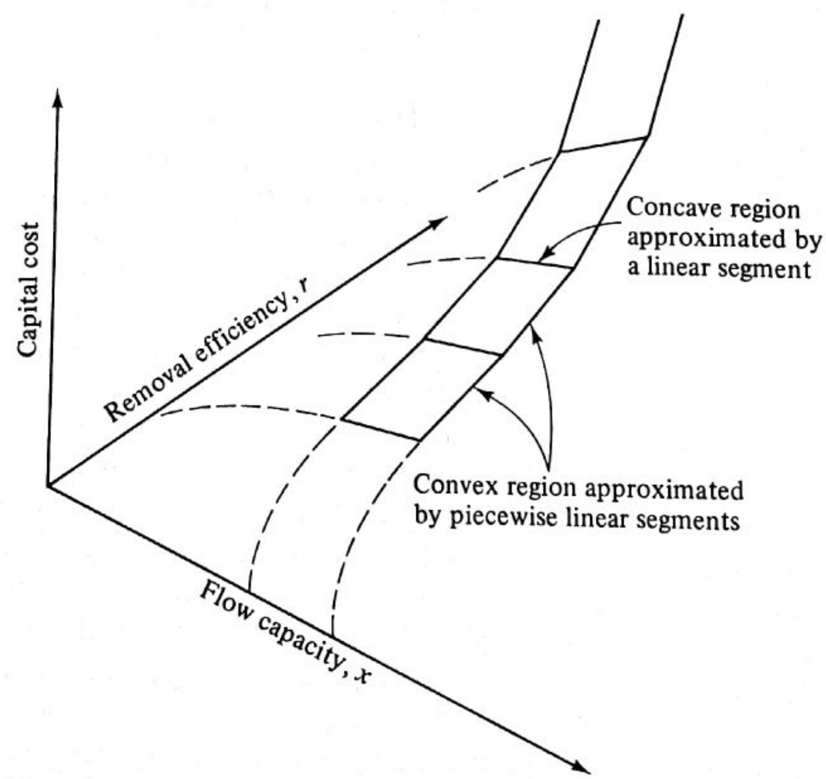

FIGURE 4-2
Linearization of concave segment of design flow cost function.

the approximation, is a function primarily of pollution load growth rates. To consider this method in greater detail, examine a two-period capacity expansion problem. Let D_1 and D_2 represent the design loads for the wastewater flow in periods 1 and 2. They are a function of the flow growth rates for the particular point sources and hence a function of population growth. The smaller the growth rate, the smaller will be the difference between D_1 and D_2, and hence the smaller the neighborhood of approximation.

The decision variables include:

X_1 = total capacity to be built by start of the first time period
X_2 = incremental capacity to be built by start of the second time period
r_1 = percent removal in first period
r_2 = percent removal in second period

The parameters include:

r_0 = percent removal in existing facility
D_1 = design flow for first period
D_2 = design flow for second period
$\Delta = D_2 - D_1$
r_{m1} = minimum percent removal for first period (secondary) (assume r_{m1} is greater than or equal to r_0)
r_{m2} = minimum percent removal for second period (best-practicable)
T_1 = length of first period (years)
T_2 = length of second period (years)
($T_1 + T_2$ is the length of the planning horizon)

To develop expressions for the cost functions it is necessary to introduce various constraints.

4-2.4 Treatment Facility Cost Constraints

The following constraints apply to the definition of the costs of capacity and removal efficiencies for each wastewater treatment plant:

$D_1 \leq X_1 \leq D_2$ (in the case of no existing plant) (4-13)
$Q_0 \leq X_1 \leq D_2$ (where Q_0 is an existing capacity) (4-14)
$0 \leq X_2$ (the nonnegativity condition) (4-15)
$r_1 \geq r_{m1}$ (minimum level of treatment for the first period) (4-16)
$r_2 \geq r_{m2}$ (minimum level of treatment of the second period) (4-17)

The explicit assumption made above is that the cost function, for a *particular* concentration and flow, may be written over a limited range of capacity and removal efficiencies as a separable function of capacity and removal.

A mechanism for expressing the cost functions for each period remains to be developed. If K_1 represents the capital costs prior to the first period, and K_2 the capital costs during the first period, assuming X_0 to be zero (for purposes of simplicity of presentation, X_0 will be assumed to be zero henceforth; for situations where it is nonzero, simply substitute $X_1 - X_0$ in the applicable cost functions and budget constraints), then the expenditure prior to the first period must involve three separate terms. The first term is the capital cost associated with a design flow of D_1 and a removal efficiency, r_0. The second term reflects the change in cost resulting from the difference between X_1, at the beginning of period 1, and the design flow D_1, at the beginning of period 1. As explained above, for small differences $X_1 - D_1$, the change in cost can be approximated as a linear function, $a(X_1 - D_1)$. The coefficient "a" is the slope between the capital costs associated with increasing the removal efficiency from r_0 to r_1, assuming that the flow of the treatment is D_1. The removal cost is a convex function of the form depicted in Fig. 4-3. The capital costs to be incurred by the start of the first time period can therefore be written as

$$k_1(X_1, r_1) = K_1 + a(X_1 - D_1) + f_1(r_1) \tag{4-18}$$

where K_1 is defined by Eq. (4-12) when $X_1 = D_1$ and $r_1 = r_0$ and $f_1(\;)$ denotes the cost function for time period 1 to increase the treatment efficiency of the flow X_1 from r_0 to r_1.

The capital costs incurred during the first period T_1, depend on whether incremental capacity X_2, will be constructed during this period, i.e., whether

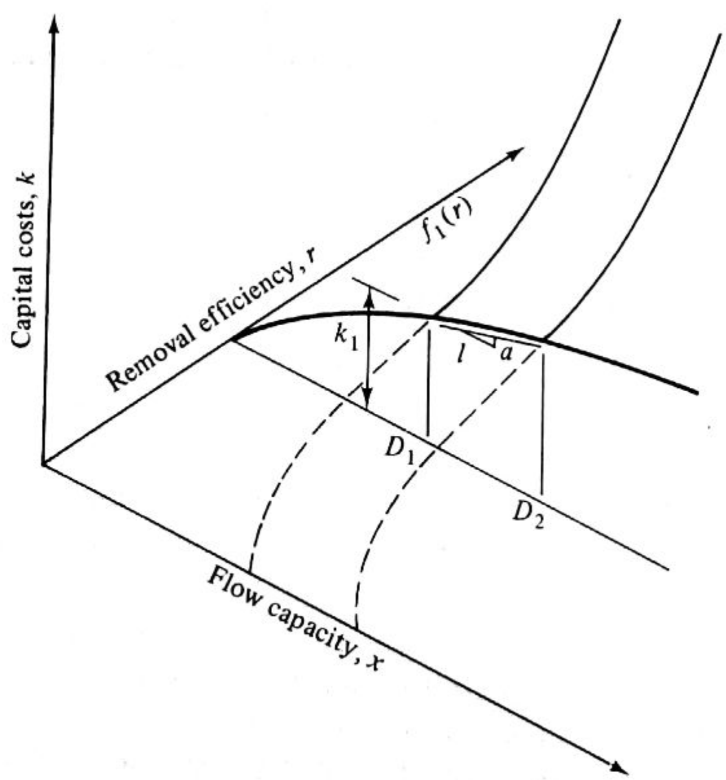

FIGURE 4-3
Capital costs incurred by the start of the first time period as a separable function of capacity, x and removal, r.

$X_2 = 0$ or $X_2 > 0$. If $X_2 = 0$, then the capital costs of the first period equal only the costs of increasing the removal efficiency from r_1 to r_2,

$$k_2(X_2, r_2) = f_1(r_2) - f_1(r_1) \qquad \text{if } X_2 = 0 \tag{4-19}$$

as shown in Fig. 4-4. If $X_2 > 0$, the capital cost includes the cost of constructing additional flow capacity X_2, and increasing the removal efficiency of the total capacity flow, $X_1 + X_2$, to r_2. The cost of additional removal efficiency, i.e., from r_1 at the beginning of period 1 to r_2 during period 1, for a flow capacity of X_1, is defined in Eq. (4-19). The unit cost of removing a fraction of the waste r_2, from an additional flow X_2 can be estimated by a linear approximation, b, of the initial portion of the cost function for capacity X given r_1, $k(X|r_1)$, as illustrated by Fig. 4-5.

$$b = \frac{k(D_2 - D_1|r_1) - k(0|x_1)}{D_2 - D_1} \tag{4-20}$$

The removal efficiency costs for X_2 are approximated by the function $f_2(r_2)$ which denotes the cost function for attaining removal efficiency r_2 in the increased capacity X_2. The capital costs may now be written as

$$k_2(X_2, r_2) = bX_2 + f_2(r_2) + f_1(r_2) - f_1(r_1) \qquad \text{if } X_2 \geq 0 \tag{4-21}$$

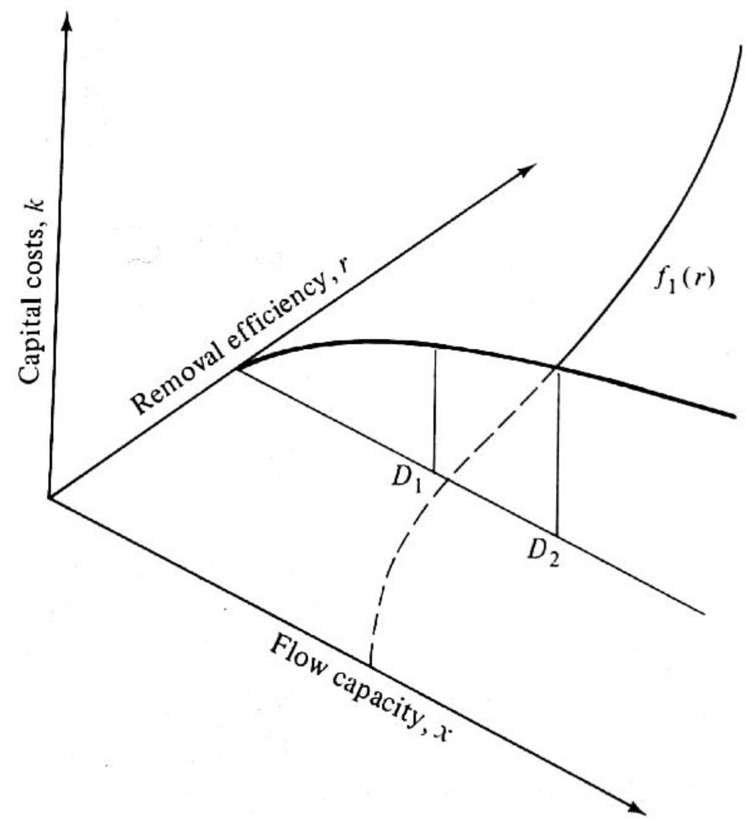

FIGURE 4-4
Capital costs incurred during the first time period when $x = 0$.

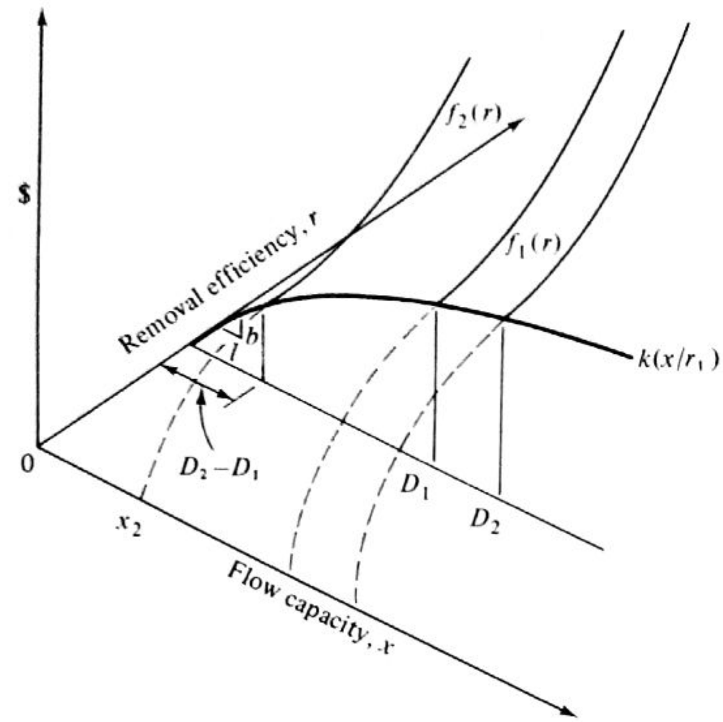

FIGURE 4-5
Capital costs incurred during the first time period when $x > 0$.

This equation is only an approximation of the actual costs associated with X_2 and r_2, based on an assumed D_1 and D_2. If the values obtained for X_2 significantly differ from either 0 or $D_2 - D_1$, then the b and $f_2(r_2)$ should be changed to more accurately estimate the total capital cost before resolving the model.

There are at least two approaches to handling the expression $k_2(X_2, r_2)$: (1) a mixed integer formulation; or (2) a heuristic utilizing a "loss function on X_2." Both have been investigated in the Saint John Study.

The first approach suggests that I_1 be a dichotomous variable and

$$0 \le X_2 \le \Delta I_1 \tag{4-22}$$

then

$$k_2(X_2, r_2) = bX_2 + I_1 f_2(r_2) + f_1(r_2) - f_1(r_1) \tag{4-23}$$

The second approach suggests a linear loss function in X_2 as a replacement for $f_2(r_2)$. It should be noted in general that $f_2(r_2)$ will not be a major term of $k_2(X_2, r_2)$ in the cases under consideration because the capacity expansions during the second time period will be small.

Consider the graph shown in Fig. 4.6.

Let $\bar{r}_2$ be the maximum expected removal for the second period and $f_2^* = f_2(\bar{r}_2) - f(r_{m1})$; then let

$$f_2(r_2) \simeq (X_2 - X_1)\frac{f_2^*}{\Delta} \tag{4-24}$$

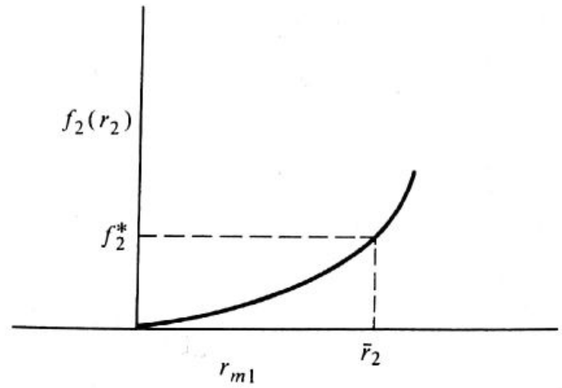

FIGURE 4-6
Cost and percentage removal relationship.

hence

$$k_2(X_2, r_2) = b(X_2 - X_1) + (X_2 - X_1)\frac{f_2^*}{\Delta} + f_1(r_2) - f_1(r_1 - r_0) \quad (4\text{-}25)$$

Since the second approach avoids the need for integer programming and its concomitant problems of possible "cycling" and of extensive computer expenditure, where the load growths during the second time period are expected to be fairly small, the second approach is the better option. Collecting terms, the mathematical expressions for k_{i1} and k_{i2} can now be written as

$$k_1(X_1, r_1) = a_1 X_1 + f_1(r_1) + \text{constant terms} \quad (4\text{-}25a)$$

$$k_2(X_2, r_2) = a_2 X_2 - a_3 X_1 + f_1(r_2) - f_1(r_1) + \text{constant terms} \quad (4\text{-}25b)$$

where a_1, a_2, a_3 are positive coefficients and f_1 is a convex function. A similar argument leads to the following for operation and maintenance costs:

$$0_1(X_1, r_1) = b_1 X_1 + g_1(r_1) \quad (4\text{-}26a)$$

$$0_2(X_2, r_2) = b_2 X_2 - b_3 X_3 + g_1(r_2) - g_1(r_1) \quad (4\text{-}26b)$$

where g_1 is a convex function and b_1, b_2, and b_3 positive coefficients.

In the objective function both the capital and operating cost functions will be multiplied by discount factors β_1, β_2, and θ_1, θ_2.

The objective function then is

$$\begin{aligned}\min Z = &\sum_{j\in N} \{\beta_1[a_{1j}X_{1j} + f_{1j}(r_{1j})] + \theta_1[b_{1j}X_{1j} + g_{1j}(r_{1j})]\} \\ &+ \sum_{j\in N} \{\beta_2[a_{2j}X_{2j} - a_{3j}X_{3j} + f_{1j}(r_{2j}) - f_{1j}(r_{1j})] \\ &+ \theta_2[b_{2j}X_{2j} - b_{3j}X_{3j} + g_{1j}(r_{2j}) - g_{1j}(r_{2j})]\}\end{aligned} \quad (4\text{-}27)$$

where the second subscript j indicates a particular pollution source, of which there are N. The N pollution source points j are a subset of the J activity points defined earlier.

There are four types of constraint:

1 Local constraints
2 Removal and load relationships
3 Instream quality constraints
4 Budgetary and equity constraints

The first of these are Eqs. (4-13) through (4-17) associated with each of the cost functions. The second set of constraints relates the removals and loads of the various pollutants to that of biomass potential. To facilitate this, we define the following sets of variables and parameters.

Parameters

C_{jpt} = the concentration (#/mgd) at waste point j before treatment, of the pth pollutant in the tth period. $p = 1, \dots N_p$. $p = 1$ for biomass potential

L_{jpt} = the daily load (#/day) . . .
$= C_{jpt} D_t$, where D_t is the design flow defined above.

Variables

r_{jpt} = removal percentage of the pth pollutant at the jth waste point in period t

$$r_{jpt} = \lambda_{jpt} r_{j1t} \tag{4-28}$$

relates the removal of the other pollutants to that of biomass potential. Within the range of removals considered, the linear relationship is adequate in most cases.

$\hat{L}_{jpt}$ = the daily load (#/day) at waste point i after removal of pollutant p after treatment in the tth period

$$\hat{L}_{jpt} = L_{jpt} \lambda_{j1t} r_{j1t} \qquad \lambda_{j1t} = 1 \tag{4-29}$$

The third class of constraints are instream quality/waste load relationships and the limits (or targets) placed on instream quality. We let Q_{int} be the nth quality measure at the ith stream point in period t. Recalling the structure of Eq. (4-11) the instream quality/waste load relationships are of the following form

$$Q_{\text{int}} = \sum_{j \in J} \sum_{l \in K} T_{jil_k} L_{jpkt} + Q_{j*} T_{jil} \tag{4-30}$$

and the quality targets are of the following form

$$Q_{\text{int}} \leq \bar{Q}_{\text{int}} \qquad \begin{array}{l} t = 1, 2 \\ i = 1 \dots \end{array} \tag{4-31}$$

where the transfer coefficient T_{ijl_k} is based on an appropriate low flow regime, usually the ten-year seven-day low flow.

Here J is the set of pollutant points upstream from i, and j^* is the uppermost point in the system. K is the set of pollutant/quality measure couples appropriate and l is the appropriate quality measure/quality measure couple (e.g., BOD nitrogenous/DO couple).

The last set of constraints that may be of interest are budgetary and equity constraints. These constraints may take the form of limitations on capital and/or operating expenditures for a particular polluter or for sets of polluters. Let $K_{tj}(r_{tj})$ and $O_{tj}(r_{tj})$ represent the cost functions of Eq. (4-25) and (4-26) for time period t and pollution source j; then such constraints might be of the form

$$K_{tj}(r_{tj}) \leq \bar{K}_{tj} \tag{4-32}$$

$$O_{tj}(r_{tj}) \leq \bar{O}_{tj} \qquad \text{for some or all of } j, t \tag{4-33}$$

where the righthand sides of Eqs. (4-32) and (4-33) reflect capital and operation and maintenance budget limitations. Possible equity constraints might take the form

$$K_{tj^1}(r_{tj^1}) \leq \alpha_j \sum_{j \neq j^1} K_{tj}(r_{tj}) \qquad \alpha_j < 1 \tag{4-34}$$

reflecting the fact that irrespective of marginal costs, one polluter should not bear more than some percentage of the basin costs. There are a number of other equity or political constraints that may be introduced. In addition, as discussed in deLucia, et al.,[2] in order to facilitate solution, constraints are introduced as necessary and useful when problem data require them. For example, it has been necessary to introduce piecewise linear constraints on biomass potential and constraints relating to removal efficiency with particular treatment processes.

In summary, the model selects treatment levels and capacity expansion sizes and timing based on a criterion of minimization of weighted present value of capital and operating costs. The weight can be varied to reflect either federal resource cost criteria or local/regional (after subsidy) costs or other alternative weights.

The selection is constrained by (1) minimum treatment and capacity constraints which reflect federal legislation (these are augmented with local constraints to facilitate model solution); (2) waste removal and load relationship; (3) instream quality constraints which reflect stream standards and/or targets; and (4) various budgetary and equity constraints.

4-3 MODEL VALIDATION AND EXAMPLE SOLUTION

4-3.1 The Need to Calibrate/Verify the Descriptive Relationships

This section discusses an indispensable step in water quality modeling, namely validation. In specifying the functional (or structural) relationship among variables in the previous sections, we have made assumptions, many of which are for convenience while others are founded on the observational data of similar systems. For example, we have postulated that the Saint John River behaves like the Ohio River and some laboratory systems in that the BOD degradation is proportional to the BOD remaining. We also assume that the rate of degradation in our systems may be different from that of the Ohio River* and is to be determined by the pertinent data. Therefore, it is important to realize that a water quality model is, at best, an approximation of the actual system, and that results obtained are only as reliable as the input information and the accuracy of the operations performed on the data.

Recognizing that a model is only an approximation of the real system, it appears logical to visualize that water quality modeling is merely curve fitting in a river system. This line of recognition motivates one to apply fruitfully the available curve fitting techniques in water quality modeling. For instance, statistical theory of estimation and hypothesis testing are all useful in model building, and we have utilized such approaches in other cases.† On the other hand, failure to recognize the quality modeling as curve fitting has, in part, contributed to the making of water quality modeling a field of ambiguity and mystery. Too often the calibration/verification procedure is described as a distinct and creative step in water quality modeling. However, it is merely, in fact, an ad hoc procedure to fill partially the role of estimation and hypothesis testing.

Admittedly, on many river basins rigorous estimation and hypothesis testing cannot be carried out due to a paucity of data, although some less precise and rigorous procedures must be adopted to justify the model before its use. But whatever the procedure used, it must be clearly described together with any adjustment to a parameter's values. In the following we present the procedure used to justify the Saint John River Basin model.

Although in the full Saint John study, calibration/verification procedures were undertaken for all of the sub-basins, the discussion below will focus on only a portion of one sub-basin, namely the Grand Falls headpond region. This region is highlighted since it contains interesting hydraulic characteristics and is also the region receiving the largest pollution inputs. This segment of the basin is shown schematically in Fig. 4-7.

Equation (4-11) shows that the quality indicator concentration instream at a point i is a complicated summation of products of the upstream loadings and the appropriate transfer coefficients. Within the reliability of sampling theory, the quality indicator level is known from the instream measurements. The problem is then one of examining the system with respect to those unknown values of the loads and transfer coefficients. The large number of unknown parameters (e.g., loadings, instream decay and sedimentation rates, etc.) necessitates numerous initial assumptions which in turn affect the computed profile. To create a better comparison between measured and computed profiles, the art of calibration becomes important in determining which characteristics of the computed profile

* The Ohio is the river that the "Streeter–Phelps"[8] formulation utilized.

† See, for example, deLucia and Chi.[1]

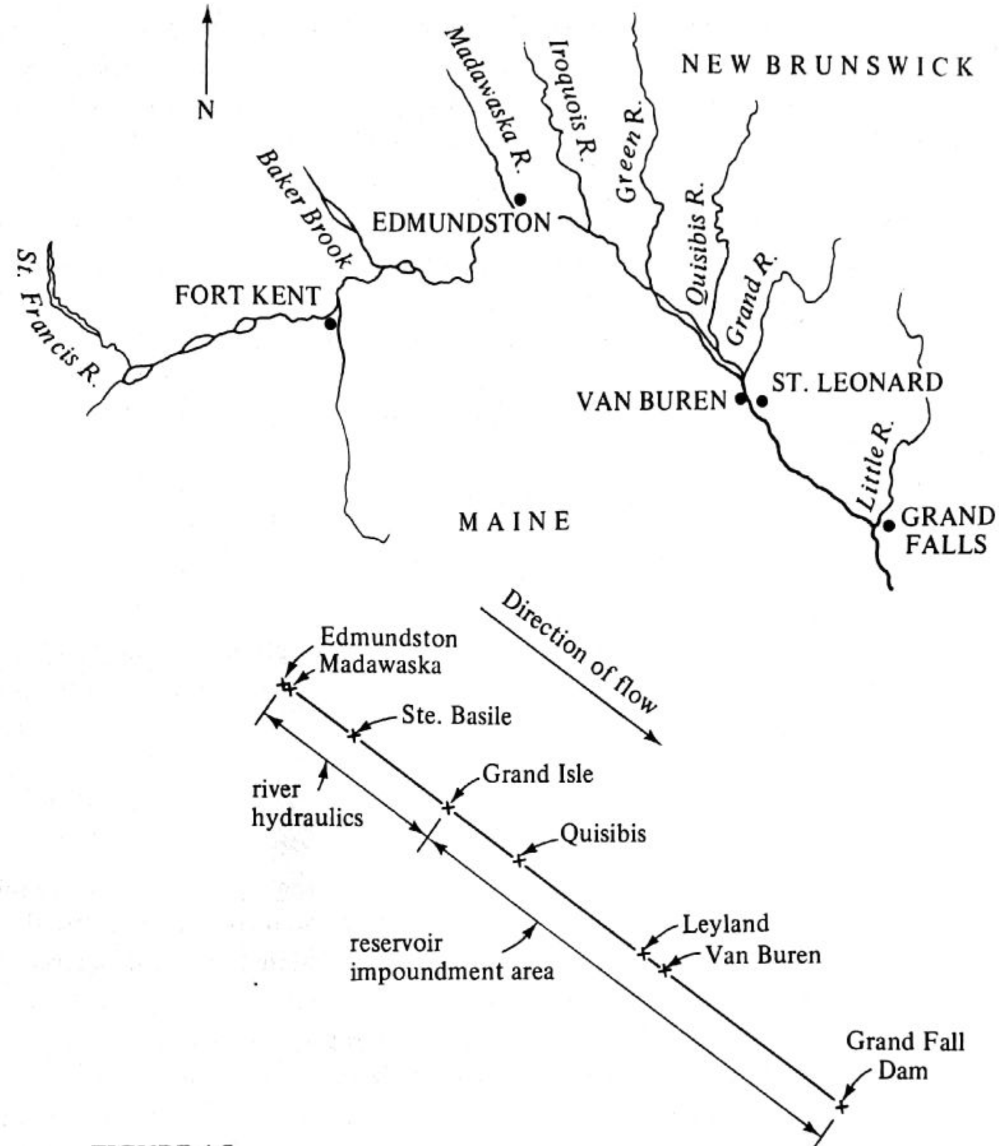

FIGURE 4-7
Spatial location of point sources in example application.

are incorrectly being modeled or are in need of incremental adjustment. During the calibration exercise it is essential to maintain an understanding of the physical system being modeled, since apparent calibration can be obtained by incorrect adjustment of parameters; however, this would create incorrect forecasts of the water quality levels if the model were subsequently used in a predictive or verification-type mode.

In this example we discuss only the calibration exercises associated with the non-conservative pollutants since their transfer coefficients are more complex.

A series of line schematics identifying the spatial location of sampling stations along the river sections provides a medium of comparison between the measured and computed water quality profiles. On these schematics, the DO–BOD stream sample information collected during a particular sampling program is plotted. The ranges of the sample measurements are denoted by the wide vertical bar at each of the sampling stations, the length of the vertical bar indicating the spread of the measurements. The short cross-bar on the vertical bar denotes the mean value of the measurements. It is important to point out that the plotted values for BOD profiles represent the ultimate total BOD values. The need for considerations of ultimate (as opposed to five-day) BOD values arises because of the extensive time of travel for transit through the river system. The transformation of the instream samples of five-day BOD is complicated because the ratios of ultimate carbonaceous (BOD_C) and nitrogenous (BOD_N) values to the five-day values are dependent on the nature of the source. A method for computing the transformation was developed by utilizing the best estimate of the transfer coefficient and the pollution release levels to create a weighted coefficient for the ratio of five-day to ultimate BOD conditions for the instream, sampled BOD_5 levels.

Manipulations of the sampling results, in accordance with procedures such as those indicated above, provide the appropriate data to compare with the mathematically computed results. Prior to the mathematical computation phase, however, there were many considerations that received attention, the more important being:

1 Checking the one-dimensionality assumptions—i.e., the assumption of cross-sectional uniformity implicit in the mathematical model—is an important approximation that must be critically examined particularly for regions of considerable river width, such as found in impoundments like the Grand Falls headpond. Experimental evidence of the cross-sectional homogeneity was provided by sampling at intervals across the cross-section. The results identified no strong channeling effects, indicating the one-dimensional assumption to be appropriate.

2 Selection of reaeration levels—the reaeration coefficient, k_2, is one of the rate constants incorporated within the mathematical model computation. Since the rate constant cannot be measured accurately in a polluted stream, and is a function of the characteristics of the stream (e.g., velocity, bottom slope, etc.), as a first approximation, the appropriate reaeration levels must be derived from literature equations. Efforts to match sampled and computed profiles were complicated because, as is so often the situation in sampling programs, a considerable instability of the flow regime occurred during the sampling periods. A sensitivity analysis of reaeration coefficient levels was undertaken for both the river regime and the headpond to determine the magnitude of the changes in the velocity and reaeration that probably occurred during the sampling program. This information was useful in suggesting the level of uncertainty and therefore the difference between the experimental and calculated profiles that might be attributed to the selection of the reaeration coefficient level.

3 Since the Grand Falls headpond creates an impoundment for a distance of about thirty miles upstream of the dam, it was important to establish the consequent effects on the pollutant transport processes (e.g., whether the water stratifies). Use of a technique based on a reservoir Froude number[19] indicated that the reservoir does not stratify. This finding was later backed up with experimental evidence. The absence of stratification, however, did not preclude a considerable decrease in velocity in the wide portions of the reservoir and a consequent difficult task of establishing levels for the sedimentation coefficient.

The calibration computations are necessarily an iterative process, each trial refining appropriate parameters and hopefully improving the comparison between measured and calculated water quality profiles (subject to the proviso that no parameter be adjusted without valid reason). Table 4-3 documents a synthesis of the calibration exercises on the Basin. The information has been tabulated by (1) calibration run number, (2) general comments, (3) flow regime, (4) point source, (5) background contribution, and (6) rate coefficients. The second heading describes the general emphasis of the calibration run. The last four columns denote general sub-sectors of the problem and entries in the columns describe the specific refinements made in each run. The successive runs of the calibration generally indicate activity in sectors or columns from left to right (e.g., it is necessary to establish the hydrologic regime prior to refinement of background loading contributions).

Several general comments will clarify elements on Table 4-3:

1 The flow regime of the computer program is developed by summing successive contributions along the river system. Therefore, in order to match the hydrologic regime of the sampling program, one must iterate, trying flow additions and then modifying, if necessary, to create a match of hydrology at many points of the system.

2 The literature estimates of background pollutant contributions are necessarily given as a range of values. Successive trials were necessary to establish the general area within the literature range which was appropriate for the Saint John River. (This range factor was particularly important for the conservative pollutants since the principal N and P contributions develop from background sources.)

3 As illustrated in the example results presented in Fig. 4-8, the series of minor adjustments eventually provided good comparisons to the instream measured values upstream of Edmundston. Successive adjustments to parameters for the region downstream of Edmundston, however, failed to create the comparison with the instream measured parameters that was expected. This divergence indicated that an important contribution or activity was being omitted. Adjustments to coefficients and loadings produced an improvement in measured and computed profiles, but the profiles still demonstrated a wide divergence in behavior. Subsequent bottom sampling indicated a significant benthal activity in the area. This finding represented an important affirmation of the value of the calibration exercise.

Table 4-3 DESCRIPTION OF SEQUENTIAL CALIBRATION EXERCISES

Run	General	Flow regime	Point source	Background contribution	Reaction rates
1	Testing for configuration of data deck, e.g., connectivity of node points	First trial at flow contributions to create hydrologic regime	First trial at loading contributions	First trial at background contributions	First trial at decay rates
	Results—connectivity is satisfactory	Requires adjustment			
2	Evaluating flow contributions	Refinement of contributions			Refinement of travel times in Grand Falls headpond
3	Attention focused on point source contributions	Minor adjustments	Modification of levels to reflect new data, new BOD_5/BOD_u ratios		
	Results—point source contributions much too high to agree with instream measurements	Hydrologic regime in agreement with values from sampling set #1	Reduction in levels needed—checking of assumed conditions		
4	Attention focused on point source contributions		Decrease in point source contributions		
	Results—movement in correct direction but insufficient—background loadings too high		Checking of assumed conditions		
5	Attention focused on background levels			Lowering of values in accordance with some literature values	Adjustment of k_1 level for Edmundston Fraser's to reflect revised estimates
	Results—concentration levels still too high in computed levels			Contributions too high	

6	Attention focused on background levels throughout river basin Results—large disagreement still prevalant in Grand Falls head-pond		Lowering of background contributions in particular areas	
7	Attention focused on Grand Falls headpond Results—agreement better between computed and measured but still significantly different	Adjustment of area *vs* depth relationships to test sensitivity in Grand Falls area Computed profiles not very sensitive to adjustments		Adjustment of sedimentation rates in Grand Falls headpond
8	Attention focused on Grand Falls headpond Results—as in (7)			Reduction in reaeration rate rate G. F. headpond; upward adjustment of rate
9	Attention focused on Grand Falls headpond Results—same as in (7) and (8)—must be factor causing divergence in addition			Similar to (8) $k_2 \simeq 0.001$ $k_2 \simeq 0.8$
10	Attention focused on Grand Falls headpond Results—general agreement between computed and measured profiles but further adjustment of benthic and decay rates required		Addition of estimated benthic demand	

Table 4-3 (continued)

Run	General	Flow regime	Point source	Background contribution	Reaction rates
11	Attention focused on Grand Falls headpond Results—improvement in agreement but sedimentation and therefore BOD rate of change is too high in headpond	Incremental adjustments in travel times between points in headpond	Decrease in Edmundston loading		
12	Attention focused on Grand Falls headpond Results—sedimentation rate still too high, magnitude of load must still be too great		Decrease in Edmundston loading		Adjustments in k_2 to reflect O'Connor formulation; adjustment of k_3 from 0.8 to 0.6 in headpond
13	Attention focused on Grand Falls headpond Results—agreement very good between experimental and computed profiles		Decrease in Edmundston loading Edmundston contribution apparently significantly less than originally thought		Adjustments of k_2 to 0.2 in headpond

Table 4-4 CHARACTERISTICS OF THE POINT SOURCES

		Design characteristics—period 1				Design characteristics—period 2			
Number	Location name	Design flow (mgd)	BOD (lb/d)	Total N (lb/d)	Total P (lb/d)	Design flow (mgd)	BOD (lb/d)	Total N (lb/d)	Total P (lb/d)
1	Fort Kent Municipal	0.2	655.0	135.0	45.0	0.22	720.0	150.0	50.0
2	Stein Hall	0.6	1,340.0	60.0	20.0	0.6	1,340.0	60.0	20.0
3	Frenchville Municipal	0.1	330.0	40.0	15.0	0.13	430.0	50.0	20.0
4	Frenchville Starch	0.65	28,630.0	235.0	80.0	0.65	28,630.0	235.0	80.0
5	Edmundston	1.25	4,380.0	375.0	125.0	1.28	4,485.0	385.0	130.0
6	Fraser Pulp	55.0	423,900.0	16,000.0	10.0	55.0	423,900.0	16,000.0	10.0
7	Madawaska	0.6	1,490.0	165.0	55.0	0.63	1,565.0	175.0	60.0
8	Fraser Paper	13.0	21,175.0	0.0	0.0	13.0	21,175.0	0.0	0.0
9	St. Basile	0.31	1,055.0	95.0	30.0	0.33	1,125.0	100.0	32.0
10	St. Anne	0.13	415.0	40.0	12.0	0.13	415.0	40.0	12.0
11	Van Buren	0.56	1,105.0	120.0	40.0	0.57	1,125.0	125.0	41.0
12	St. Leonard	0.20	560.0	45.0	15.0	0.21	590.0	47.0	16.0

The comments included on Figs. 4-8 through 4-11 indicate, in addition to Table 4-3, the sequence of trials taken prior to a reasonable calibration. Too frequently in discussion of water quality models, it is only the equivalent of Fig. 4-10 and a brief discussion which is presented.

4-3.2 Example of Prescriptive Model Solution

The model was applied in examining a series of pollution control options after it had been validated. An application of the prescriptive model to the heavily-stressed region mentioned above is presented here. The fifty-mile long segment illustrated above in Fig. 4-7 receives large quantities of pollutants from both American and Canadian industrial and municipal sources. The industrial sources include potato processing wastes, and most important, pulp and paper water-borne residuals. The downstream portion of the stretch of river is a relatively shallow impoundment that, because of both the extent of the pollutant sources

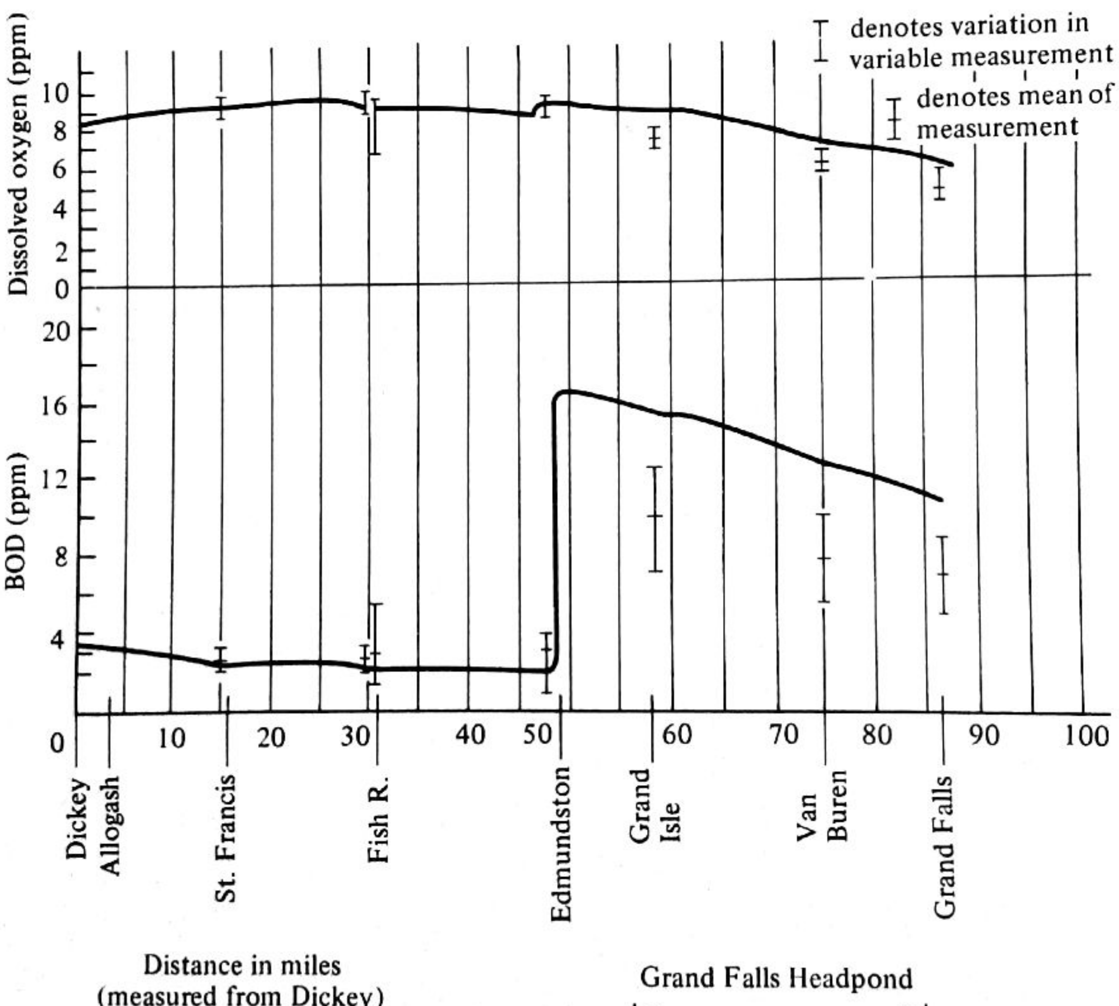

FIGURE 4-8
DO 8 BOD Profiles, mainstream Saint John River verification runs.

and the decreased assimilative capacity due to the presence of the impoundment, incurs frequent anaerobic water quality levels.

Figure 4-12 illustrates the location of the point sources and Table 4-4 documents the relevant characteristics of the sources. Table 4-5 lists the water quality characteristics of the present concern. The cost curve information and the pollutant removal efficiencies for the stream of unit treatment processes considered as alternatives in the model are documented in Table 4-6.

Since the presentation of the case study is principally to demonstrate the mathematical model described earlier, discussion of the implications of the results as they impinge on the Saint John River will be very limited. Simply stated, assuming the existence of a constraint stipulating at least secondary treatment to be adopted everywhere, no budget or equity constraints, and a translation of the above data into the objective function and constraint set outlined in earlier sections, a linear programming problem with 1,600 variables and 900 constraints results. The computer time required for solution of the problem was 7.6 min on

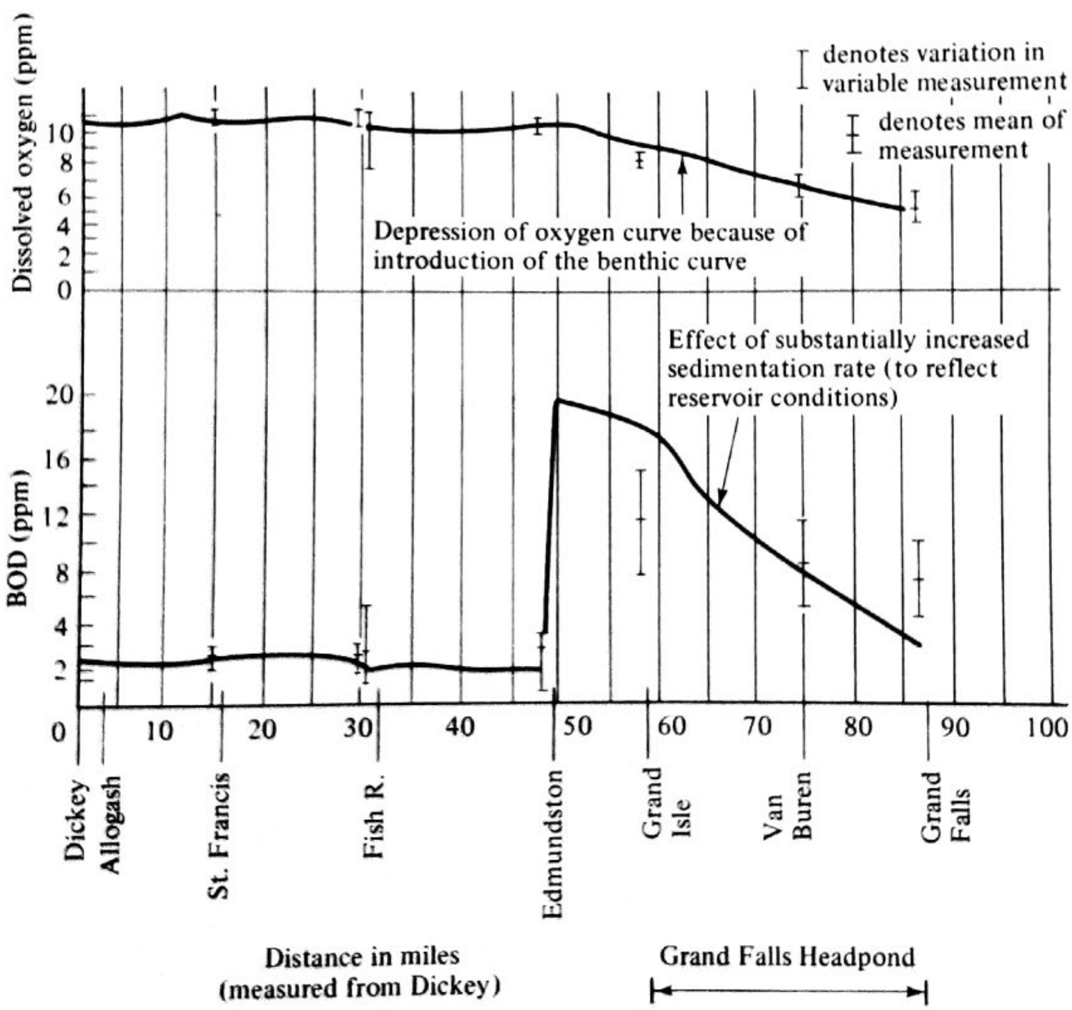

FIGURE 4-9
DO 8 BOD Profiles, mainstream Saint John River verification runs.

an IBM 370/75. The resulting set of decision variables, which indicate which of the treatment processes were adopted, is listed in Table 4-7. As well, at all locations, the capacity of the treatment installation was constructed in accord with the second time period design flow, indicating that savings in deferment of capital expenditure do not outweigh the advantage of capturing the economies of scale obtained from construction prior to the first time period.

Obviously, the problem as specified above is only one of many possible scenarios that must be examined in a planning study. In fact, the results obtained are not surprising in that, for the low growth rates typical of the Saint John area, deferment of capital expenditure for a six- to eight-year period would not be warranted. However, an important capability of the model is, for example, the ability to establish the impact of the imposition of budget constraints on some of the point sources, or, alternatively the impact on treatment plant construction of the process of upgrading the water quality (e.g., the impact of adjusting the total P constraint from 1.5 mg/l to 1.0 mg/l, effective during time period 2, is

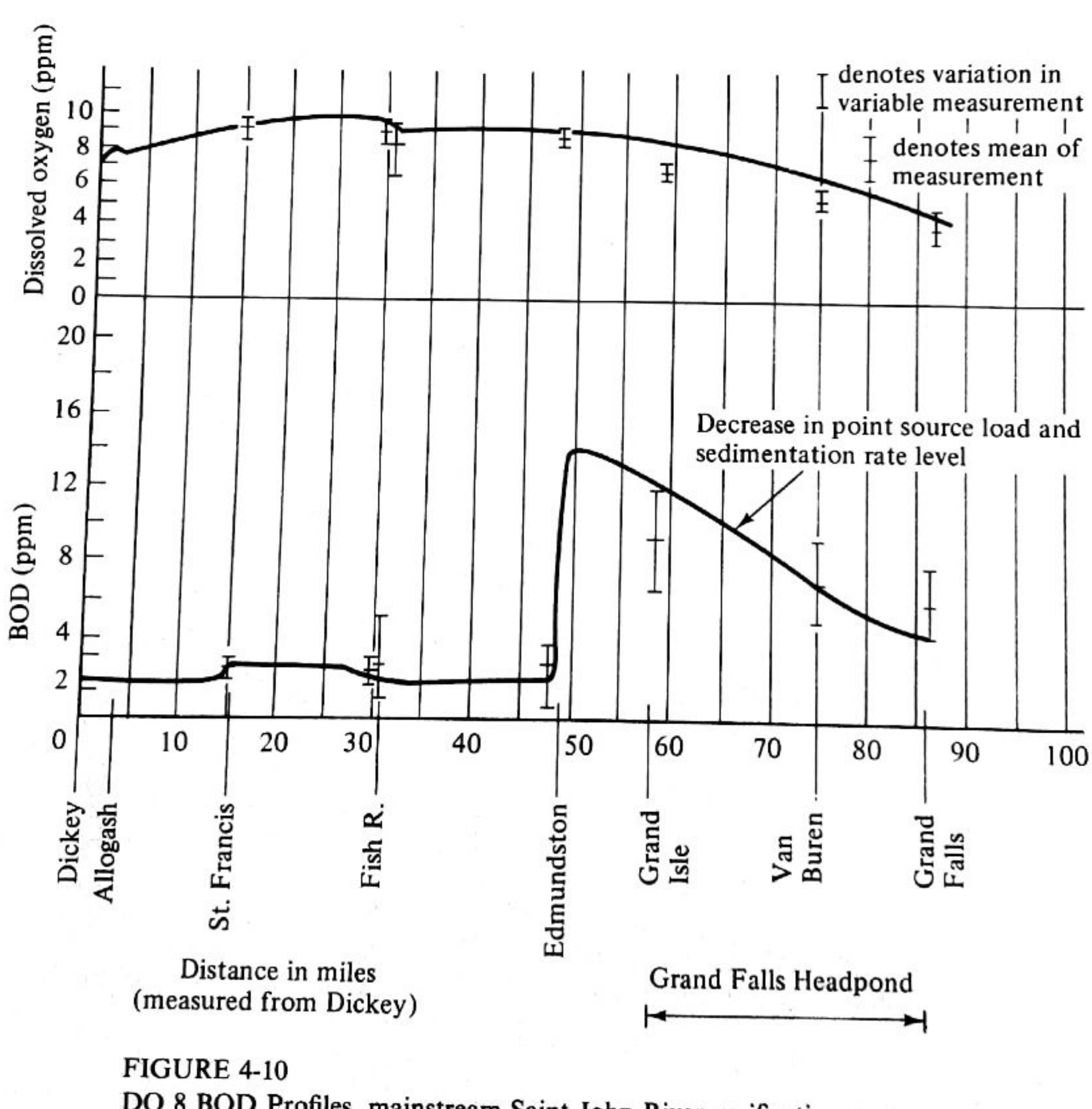

FIGURE 4-10
DO 8 BOD Profiles, mainstream Saint John River verification runs.

documented in Table 4-8, and makes the treatment considerably more sophisticated.

Table 4-5 WATER QUALITY CONCERNS

Parameter	Magnitude
Instream flow (at upstream end)	
Water quality standards:	850 ft^3/sec
Dissolved oxygen	
Biochemical oxygen demand	5 mg/l
Total nitrogen	3.5 mg/l
Total phosphorus	5.0 mg/l
Water temperature	1.5 mg/l
	22 C

In addition to cost and treatment process information, the results of the model bring to bear all of the valuable attributes of linear programming. For example, the dual variables suggest the marginal costs of adjustment of the righthand sides in the constraint set—the result is an awareness of, for example,

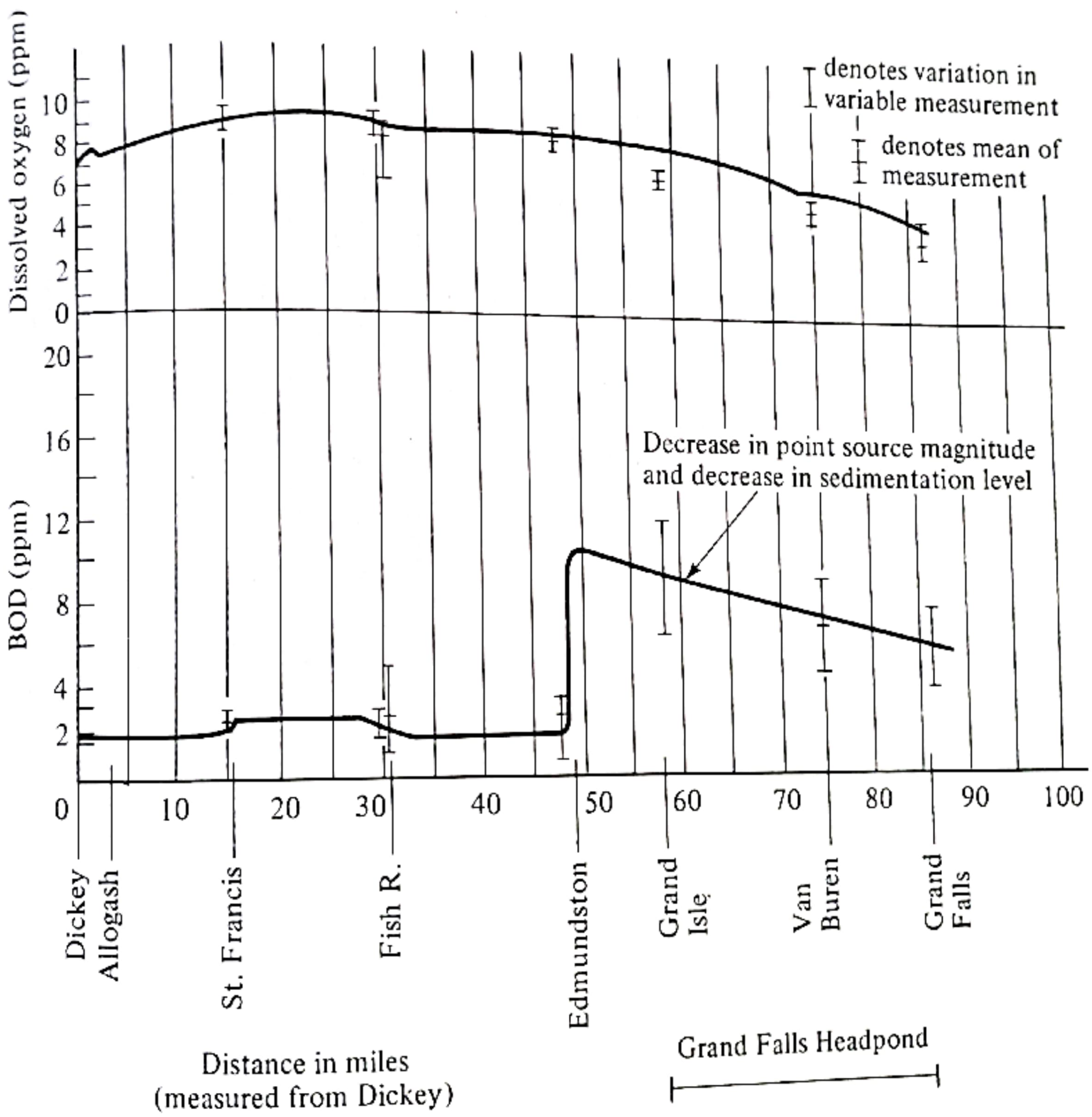

FIGURE 4-11
DO 8 BOD Profiles, mainstream Saint John River verification runs.

Table 4-6 POLLUTANT REMOVAL ALTERNATIVES CONSIDERED

Process number	Unit process name	Capital construction costs ($\times 10^6$ \$) α	β	Operation and maintenance costs (c/1,000 gal.) η	ξ	Total removal efficiencies BOD	Total N	Total P
1	Extended aeration	1.2	0.68	8,400.0	−3.42	87.5	20	10
2	First stage P removal	0.064	0.40	8.5	−0.23	90	20	70
3	Ammonia removal —nitrification	0.38	0.73	20.5	−0.73	95	20	80
4	Carbon adsorption and filtration systems	0.99	0.66	38.0	−0.41	99	35	90
5	Tertiary P removal	0.051	0.64	1.05	−0.07	99	35	99
6	Denitrification	0.111	0.96	12.0	−0.60	99	91	99

where

$$\text{Capital costs} = \alpha(\text{design flow})^{\beta}$$
$$\text{Operation and maintenance costs} = \eta(\text{design flow})^{\xi}$$

Table 4-7 DECISION VARIABLE SET

Number	Location name	Removal processes adopted Time period 1	Time period 2
1	Fort Kent Municipal	1, 2, 3, 4, 5	1, 2, 3, 4, 5
2	Stein Hall	1, 2, 3, 4, 5	1, 2, 3, 4, 5
3	Frenchville Municipal	1, 2	1, 2
4	Frenchville Starch	1, 2, 3	1, 2, 3
5	Edmundston	1	1
6	Fraser Pulp	1, 2, 3, 4, 5	1, 2, 3, 4, 5
7	Madawaska	1	1
8	Fraser Paper	1	1
9	St. Basile	1, 2	1, 2, 3, 4, 5
10	St. Anne	1, 2	1, 2
11	Van Buren	1, 2, 3, 4, 5	1, 2, 3, 4, 5
12	St. Leonard	1	1

how much treatment cost could be foregone by relaxing the water quality standard by a unit value.

The above discussion is only one very small set of exercises through which the model may be put. The value of the model is that it makes it easy to examine the impact of the many different kinds of questions that arise in a water quality management study that in essence are "what happens if I do this?" The computer model used in the study accepts the general description of the river, pollution sources, etc., and translates the information into input for the MPS linear programming package. As a result, in order to examine different management questions using the model, it is normally a simple matter of adjusting a few, select, data cards and rerunning the computer program.

Table 4-8 EFFECT OF UPGRADING WATER QUALITY WITH RESPECT TO TOTAL PHOSPHORUS

Number	Location name	Removal processes adopted Time period 1	Time period 2
1	Fort Kent Municipal	1, 2, 3, 4, 5	1, 2, 3, 4, 5
2	Stein Hall	1, 2, 3, 4, 5	1, 2, 3, 4, 5
3	Frenchville Municipal	1, 2	1, 2, 3, 4, 5
4	Frenchville Starch	1, 2, 3	1, 2, 3, 4, 5
5	Edmundston	1	1, 2, 3, 4, 5
6	Fraser Pulp	1, 2, 3, 4, 5	1, 2, 3, 4, 5
7	Madawaska	1	1, 2, 3, 4, 5
8	Fraser Paper	1	1, 2, 3, 4, 5
9	St. Basile	1, 2	1, 2, 3, 4, 5
10	St. Anne	1, 2	1, 2
11	Van Buren	1, 2, 3, 4, 5	1, 2, 3, 4, 5
12	St. Leonard	1	1

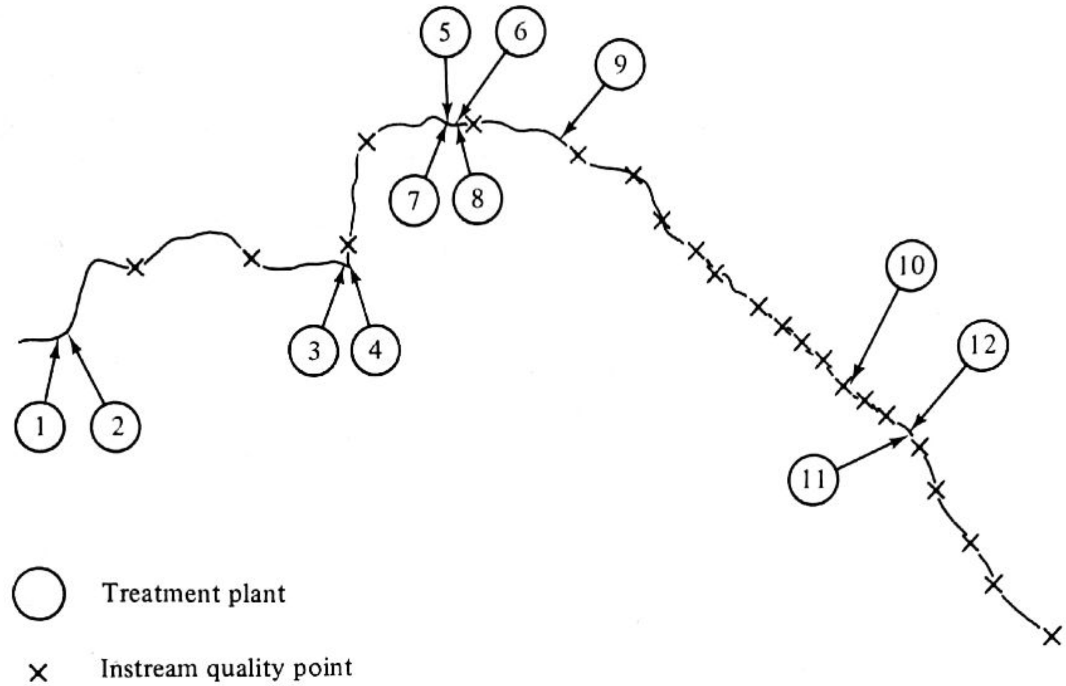

FIGURE 4-12
Location of point sources.

4-4 CONCLUDING COMMENTS

The Saint John River Basin planning study which sponsored the development of these model(s) was recently completed; hence some observations can be made regarding the usefulness of the model(s). As has been the case in the past, the model for prescriptive purposes was of more limited use than the model for descriptive purposes. There are two reasons for this. First, although knowledge of least cost solutions of control alternatives of both treatment level and capacity expansion are important in the planning process, the number of polluters in this case is small and an ad hoc analysis with a descriptive model is sufficient to investigate a reasonable range of options. In some measure this was foreseen early in the study and it was the potential for wider application and the availability of EPA sponsorship that lead to the prescriptive model development.

Second, there is the need to examine control under a variety of criteria in addition to least cost. For example, these criteria are associated with a concern for such things as: (1) cash flow implications for municipalities; (2) energy requirements of alternatives; and (3) administrative requirements of control alternatives, both during the construction phase and continuing operation. All of these are of some concern in the Saint John River Basin and the investigation is amenable to the use of the descriptive model with some complementary ad hoc prescriptive investigations, rather than the prescriptive formulation which is limited in its criteria.

Extending the comments beyond the Saint John study, we believe that there are two types of problem associated with the use of water quality planning models. The first problem is one of acceptance of a descriptive or prescriptive mathematical model for use in an actual water quality planning study. The second is associated with the implementation of a water quality plan which is based at least partially on a study in which a mathematical model has been employed.

Currently there is little impediment to the use of descriptive models in water quality planning in the United States. Model building as a water quality management analysis procedure has been gaining popularity. Advancement in the modeling skill has improved the ability of a model to account for relevant elements of the problem examined. As a consequence, the image of modeling has been enhanced. However, one of the most important stimulants to the application of a model for analysis may be attributed to a shift in the federal legal procedures in dealing with pollution control in the United States.

In a traditional procedure of constitutional governments, an individual awaiting trial in a court is regarded as innocent until he (or she) is proven guilty. The burden of proof that he (or she) is guilty lies with the plaintiff. A similar procedure has been applied in the litigation of pollution control in the U.S. The court (or environmental regulatory agency) is required to assume the burden of proof of damage due to a waste discharge. If the evidence is insufficient to prove damage, then waste producers (private or public) are free of the charge. Pollution control efforts following this procedure have not experienced a great deal of success in improving environmental quality. Since the burden of proof is on the regulatory agencies, information is valuable to them but not to waste dischargers. However, data collection and analysis efforts assumed by the agencies are inadequate to cope with the problem. Moreover, little cooperation can be expected from waste dischargers, since lack of information acts in their favor. The universal shortage of waste discharge information throughout the U.S. (and an unwillingness to accept results obtained on the basis of a model) may be partially attributable to the reluctance of waste producers to cooperate. The apparent measure to remedy the situation is to reverse the burden of proof; namely, a waste discharger would be required to provide evidence to show his innocence; otherwise any discharge is regarded as harmful to the environment.

The National Environmental Policy Act requires that for any new construction larger than a certain size an assessment must be carried out to evaluate the environmental impact. The National Pollution Discharge Elimination System (NPDES) specifies more precisely that a waste producer must obtain a permit for waste discharge. Public Law 92-500 also states that any polluter must use a minimum treatment prior to discharge and if the water quality standard is still not met, the level of treatment must be increased to reduce the amount of discharge. In these pieces of legislation, legal procedure appears to shift the burden of proving non-damage to the waste producers. Because of this change, data collection and analysis are essential for polluters. Many large waste producers have built up personnel capable of using some sort of analytical techniques to perform pertinent studies. There is evidence that these staffs are using model building in their continuous efforts to ensure consistent analysis.

The application or use of prescriptive models has not been nearly as widespread. We believe that this results from the inability of the models to produce acceptable control plans to inform those involved in the decision-making process. As stated above, the fault lies with the criteria functions employed. The functions are based on cost minimization or benefit maximization. The former criterion is often inadequate because it does not encompass such issues as those associated with distribution or equity. The latter criterion, while suffering from some of these same shortcomings, is of little use in guiding the selection of acceptable plans because water quality benefits are usually not measurable.

On the other hand, there are cases, where the situation is complex and in which it is useful to have the solution alternatives generated by prescriptive models as the basis for comparison, where one finds such models implemented. The Saint John is such a case. In this we sought an implementation of the model but not an implementation of the plan based on the solutions of the prescriptive model. The eventual plan implementation will be based on a number of issues integrated into the plan formulation process, only a few of which stem from the results of the model analysis.

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