RR #37

MODELING OF WATER QUANTITY AND QUALITY FOR RIVER-BASIN PLANNING

By

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TECHNICAL COMPLETION REPORT

Project Number: A-059-NH

Annual Allotment Agreement No: 14-34-0001-2131

The research on which this report is based was financed in part by the United States Department of the Interior, as authorized by the Water Research and Development Act of 1978 (P.L. 95-467).

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University of New Hampshire
Durham, New Hampshire

April 1983

ABSTRACT

The objective of this study was to develop and test a river-basin planning model incorporating the effects of point and non-point sources of pollution on water quality. The model simulates the quantity and quality relations in a river network under critical low-flow conditions, and incorporates precise and hydrologically sound definitions of demand and supply. Flow and water quality are modeled at use sites (municipalities or industrial locations) by simple equations that allow planners to evaluate the effects of a complete set of structural and non-structural alternatives for meeting future water demands. The model was used to compute critical dissolved-oxygen concentrations for the Merrimack River Basin in New Hampshire, and showed encouraging correspondence with measured values. Phosphorus is also an important water-quality constituent, and extensive field studies of the behavior of that element were carried out in southeastern New Hampshire to elucidate its behavior in stream networks. It was found that a given stream reach acts as a source of phosphorus under some conditions and a sink under others. It appeared that a chemical-kinetic effect and the erosion and deposition of fine sediments largely controlled this behavior. The study has established the need for water-resource planning models, described the specific elements of such models, demonstrated the feasibility of developing and applying such models in New England, and identified specific areas where research will contribute to the improvement of planning models.

ACKNOWLEDGMENTS

The work discussed in this report was supported by the Water Resources Research Center at the University of New Hampshire (UNH) as project A-059-NH. I am grateful to the Center's Director, Gordon L. Byers, for his guidance, encouragement, and support on this and preceding projects. Bruce E. Lindsay of the Department of Resource Economics and Community Development at UNH was a collaborator on an earlier project on this general topic, and discussions with him helped to clarify many of the ideas discussed here.

Much of what is presented herein is the result of the dedicated hard work and high standards of Richard M. Green and Wendy M. Larson, graduate students in the Hydrology Program of the Institute of Natural and Environmental Resources (INER) at UNH. Valuable guidance was provided by members of their advisory committees: Francis R. Hall (INER-Hydrology), Gordon L. Byers (INER-Hydrology), and Alan L. Baker (Botany).

I thank Ken McKenna and Frank Blackey of the U.S. Geological Survey in Concord, New Hampshire, for assisting in obtaining and interpreting discharge data for Dudley Brook. The Concord office of the Survey and the New Hampshire Water Supply and Pollution Control Commission provided data and guidance for the modeling of the Merrimack River Basin.

This report was completed while I was on sabbatical leave at the Institute of Hydrology, an establishment of the National Environment Research Council of the United Kingdom. I am grateful to the Institute's Director, Dr. J.S.G. McCulloch, for making the Institute's facilities available to me and to the members of its typing pool for preparing a draft manuscript. My wife, Jane, provided invaluable editorial guidance that has improved the consistency and readability of this report.

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I. INTRODUCTION

This report describes the results of a three-year study which had the objective of developing and testing a model for incorporating the effects of point and non-point sources of pollution on water quality. This model is designed for river-basin planning, with particular application in New England. Field work and testing were carried out on river basins in New Hampshire.

Following this introductory section, the report contains four main sections. The first of these discusses the objectives of river-basin planning models, the general approach to modeling that meets those objectives, and short-comings of previous attemps at river-basin planning. In general, this section establishes the need for planning models and describes the requirements that such models should fulfill.

Section III looks in more detail at those requirements and develops a conceptual planning model. The elements of this conceptual model are described in considerable detail, including explicit definitions of water demand and water supply, a comprehensive list of planning alternatives whose impacts on water quantity and quality can be evaluated via the model, and the ways in which streamflow, dissolved solids, and dissolved oxygen are modeled through the river network of a particular basin. The selection of particular water-quality indicators appropriate to New Hampshire is also discussed here.

The implementation of the conceptual model is described in Section IV. The model was applied to the Merrimack River Basin in New Hampshire, with dissolved oxygen as the only water-quality indicator. This actual modeling effort includes most, but not all, of the elements of the previously described conceptual model. The model is tested by comparing predicted critical dissolved-oxygen concentrations with measured values, and it is concluded that the model gives satisfactory results given the precision of existing data and inherent uncertainties in predicting the behavior of dissolved oxygen in rivers. However, this precision is not great, and it is suggested that further thought be given to the design of data-collection programs.

Section V describes detailed studies of dissolved-solids (chloride and several forms of phosphorus) transport in a small river basin in southeastern New Hampshire. The field data are used in models developed in Section III to elucidate the role of the stream bed as a source and sink of phosphorus.

The final sections of the report summarize the results and indicate the directions of future research needed to increase capabilities for modeling as a basis for sound river-basin planning.

II. PLANNING MODELS

Modeling Objectives and Approach

In general, water-resource models can be classified as: 1) planning models; 2) design models; and 3) operational models (Whitehead *et al.*, 1981). Planning models, such as the one developed herein, are intended to allow planners and resource managers to evaluate a large number of investment programs and to select those which appear most promising for further more detailed analysis. Because of the necessity of evaluating many alternatives, such models must generally be designed to represent the water-resource system as simply as possible while simulating the essential aspects of the system's behavior. To be useful at the planning stage, such models must also provide information about the statistics (*i.e.*, temporal variability) of the important characteristics of the system.

Dingman and Lindsay (1981) and Dingman (1981b) described the basic components and relationships of a model intended for planning in river basins like those in New England. That model retains the essential aspects of the stream network within a river basin. It is a steady-state model in which streamflows are represented by the flow that is exceeded on 95% of the days, and water use is represented by average-annual values. This provides a "picture" of the basin under conditions that are usually most critical for water supply and water quality. In using the model, the planner can alter this picture by: 1) simulating the effects of population and industrial activity, water-supply alternatives, water re-use and conservation activities, and water-treatment alternatives on water supply and quality; and 2) simulating the effects of flow-augmentation strategies on water supply and quality.

Problems with Previous Models

There appears to be a wide gap between the models developed in the literature for guiding water-resource decisions and the actual decision-making process at the local, regional, and state levels. In many cases, the problem is more than one of simply educating the decision-makers about sophisticated models, although that problem is far from simple itself. Rather, the models are commonly conceptually inappropriate, or they require data and parameter estimates in which one can have little confidence.

It is widely recognized that water-resource decisions, like most public-policy decisions, are made in the political arena in a "bottom-up" process more like the "Bow River" model of Dorfman and Jacoby (1970) than like the "top-down" process prescribed by the Principles and Standards (U.S. Water Resources Council, 1979). Not only is the "bottom-up" process multi-objective (which often implies one decision-maker with many objectives), it is multi-objective and multi-decision-maker, with possibilities issues not directly related to water resources will influence decisions.

In spite of this, single-objective optimization models are common in the water-resource literature. While it is recognized that such models represent a high degree of abstraction and only approximate the relations that are of concern to decision-makers, in some cases the problem is formulated such that the model misses the point entirely. One example of this is the classical water-source sequencing problem, as described for example by Butcher *et al*. (1969). The problem is that of identifying the sequence in which a set of *n* reservoirs should be built so as to minimize the present value of the cost while meeting the future demands for water.

However, it seems clear that no future decision-maker will be bound by a sequence dictated by a past analysis, and with good reason: the decision-maker wants only information about what to do now, and the classical sequencing problem has little relevance to this (Dingman, 1981a).

There is another aspect common to single-objective optimization approaches that must be viewed with caution. Increasing computer capacities have made it feasible to find optima among a very large number of alternative configurations, which can be helpful. However, objectives other than the one expressed in the objective function will in general be important to the decision-maker. Thus, it would be of value for one to know which alternatives were second-, third-, or fourth-best, and by what magnitude (generally, dollars) these differ from the optimum solution. This information cannot usually be obtained from these models, and this consideration seems so important to the actual decision-making process as to make one wonder whether this approach is ever appropriate.

In spite of the above considerations, the common multi-objective approaches, and even single-objective models, can inform and contribute constructively to the process of water-resource planning. However, the most valuable role of the water-resource professional may be that of developing <u>simulation models</u> that are understandable to decision-makers and which can be used interactively with them and by them as they work toward a decision. This was essentially the role of formal modeling in the Dorfman and Jacoby (1970) "case study", and it seems a useful paradigm for contributing to public-policy decisions in a democratic context.

A second set of problems with previous integrated planning models is the use of inappropriate levels of temporal and spatial aggregation. For example, two well known previous attemps at using integrated basin models as tools for planning and policy assessment were those of Wollman and Bonem (1971) and the Second National Assessment of the U.S. Water Resources Council (1978). In both these models, annual flows were used, and water supplies and demands were aggregated over large water-resource regions.

Use of long-term mean flows is generally inappropriate for water-resource planning, as means typically have very low exceedance probabilities (25 to 35% of the days in New England), and the distributions of annual flows are not very meaningful except in estimating safe yields of very large reservoirs. Daily flows provide a more realistic picture of shortages. Since it is usually possible to obtain information on the frequency distribution of daily streamflows, and to develop means of estimating low-flow statistics such as the 95% exceedance flow for arbitrary reaches (e.g. Dingman, 1978; 1981a), it makes sense to use daily flows for the estimates of supply.

Both the Wollman and Bonem study and the Second Assessment aggregated supplies and demands conceptually at the outlets of the water-resource regions. This high degree of spatial aggregation is unrealistic from many points of view, and can give a misleading picture of the nature of the water-supply problem and of the alternatives available for solving it. As noted by Rickert et al. (1976, p-M4),

> Rivers and their basins are dynamic, and the processes within them result from the interaction of complex natural factors with man's activities and alterations. This interaction creates unique local problems which, once adequately assessed, often have unique local solutions.

To be hydrologically sound, it is important to retain the basic characteristics of the stream network and the distribution of water-use points, reservoirs, aquifers, and of the influx of point and non-point sources of pollutants. This is important because neither supplies nor demands are additive, and only consumptive use can be summed over the basin. Furthermore, relations between reservoir size and yield and the effects of reservoir storage on downstream flows depend on the relative locations of the reservoirs in the network (Dingman, 1981a). In other words, it is not very meaningful to simply add storage capacities within a basin, as was done for example by Wollman and Bonem. As a final point, there are downstream changes in concentrations of both conservative and non-conservative pollutants that can only be realistically accounted for by retaining the stream-network configuration. The recent work of Gianessi *et al*. (1981) and Gianessi and Peskin (1981) describes a water-quality model that attempts to account for this configuration.

A third major problem with previous planning models is that, while they are centrally concerned with "demand" and "supply", these terms are often not carefully nor comprehensively applied. They must be defined such that they are hydrologically realistic, and such that they include considerations of water-quality and minimum-flow requirements for instream uses. Detailed definitions of these terms are developed later in this report.

III. THE CONCEPTUAL PLANNING MODEL

General Description

Figure 1 illustrates a typical model configuration. It consists of links, representing stream reaches, which are separated by nodes. The nodes are of two types, representing either stream junctions or water-use sites, and are characterized by the mixing of flows, either from the joining of tributaries or from the entrance of water from point sources of waste water.

The model is steady state. Stream flows are represented by design flows, taken here as the average daily discharge exceeded on 95% of the days (Q_{95}). These flows increase downstream at a rate appropriate to the region. Water supplies withdrawn from aquifers, stream channels, or reservoirs can be represented by annual or seasonal averages projected for the appropriate planning horizon.

Water quality is accounted for by first selecting one or more appropriate critical water-quality constituents; separate quality computations have to be made for each constituent. Simple mixing models are used to compute concentrations where two flows join at nodes or within use sites. Downstream changes in concentrations are also modeled by equations that account for influxes of non-point sources of contaminants and changes that occur within the channel due to physical, chemical, and biological processes.

Variables within the model fall into three categories: 1) forecast variables, the values of which are predicted to represent levels of population and industrial activity in future time periods of interest; 2) decision variables, the values of which are determined by the planner

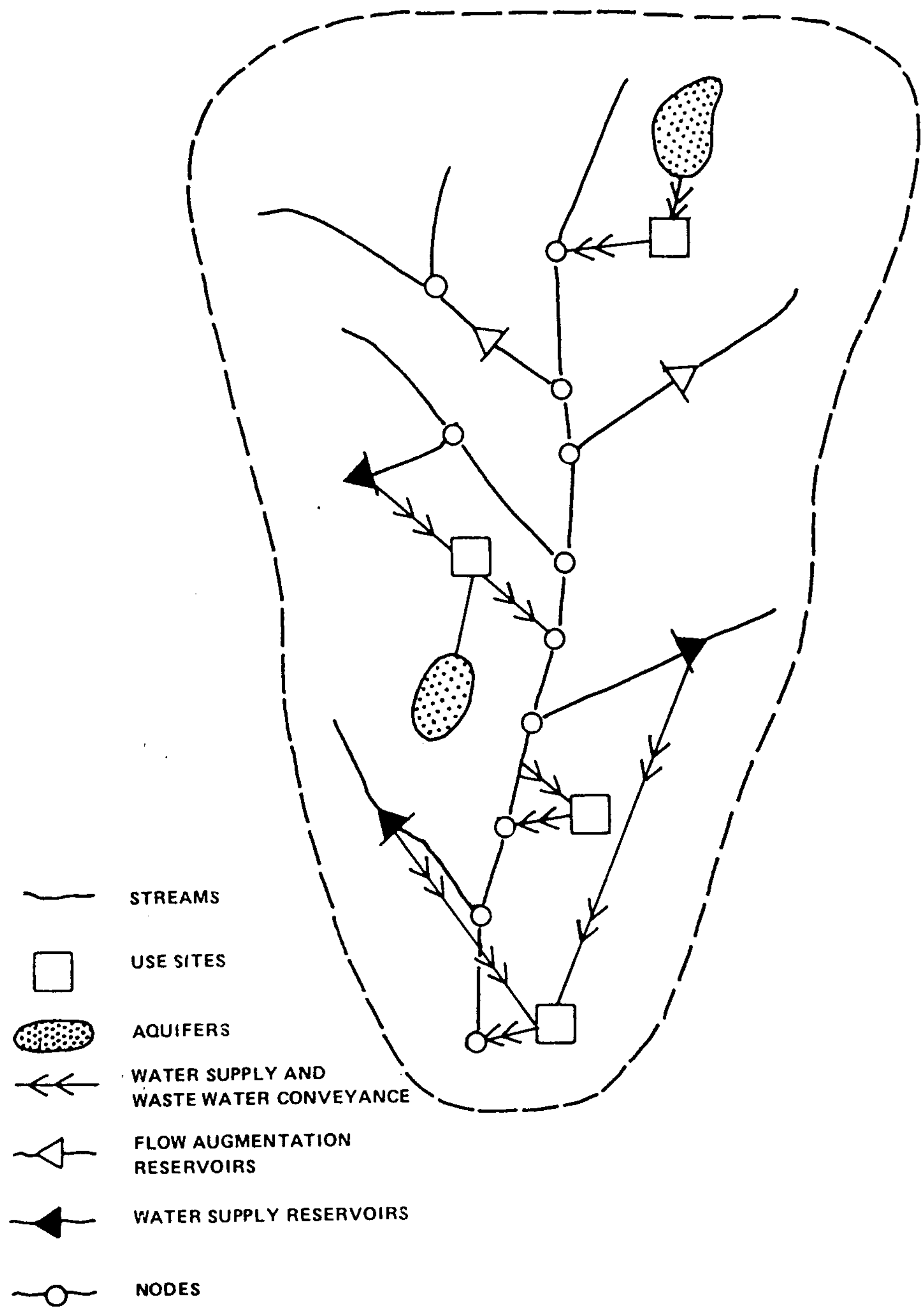


Figure 1: Typical river-basin configuration to be represented by a planning model.

reflect alternative strategies whose effects are to be investigated; and 3) computed variables, the values of which are determined by computations within the model for each set of forecast and decision variables. Appendix 1 lists all the variables within each category.

Definitions of Demand and Supply

Demand (Water Requirements) - It is most logical to begin with a consideration of "demand", as our definition of it will determine the most appropriate definition of "supply". Russell *et al.* (1970) have made the most insightful approach to defining demand for municipal systems. First, they distinguished between short-term demand (determined by maximum daily demand) and longer-term trends (determined by, say, annual averages of demand). They also point out that past records of consumption cannot be assumed to equal demand at current price levels, because use restrictions or the capacity of the distribution-treatment system may have prevented consumption from equalling demand.

Wollman and Bonem (1971) provided a careful definition of "demand" or "requirement", again considering only long-term averages. They used a high degree of spatial aggregation, and defined demand as the sum of the instream flow required to maintain water quality plus consumptive use, plus the discharge of fresh water into the ocean. Other instream uses were assumed satisfied if flow was sufficient to satisfy the water-quality requirement. The Second National Assessment (U.S. Water Resources Council, 1978) aggregated supply and demand data for major river basins, as did Wollman and Bonem (1971). Demand was computed as the sum of total consumptive use plus net evaporation and net exports of water. While both these studies provided useful insights, the definitions of demand are not totally satisfactory because they do not allow a realistic evaluation of the efficacy of a number of available management strategies.

Russell et al. (1970) defined municipal, or withdrawal, demand on the basis of long-term trends rather than short-term fluctuations that are constrained by the capacities of distribution and treatment systems. This definition is retained herein. However, it is important also to separate consumptive and non-consumptive components of withdrawal demand, as doing so allows consideration of re-use as a supply strategy, and can make a profound difference in demand forecasts. This is particularly true with respect to industrial usage, a large component of which is cooling or other process water that can be readily recycled.

As the two large-scale assessments (Wollman and Bonem, 1971; U.S. Water Resources Council, 1978) recognized, a concept of demand that is useful for river-basin planning and assessment must include consideration of instream as well as withdrawal uses. The present report distinguishes between the two classes of instream demand: 1) flows required to maintain water quality at acceptable levels; and 2) flows required to satisfy other instream requirements, such as habitat, navigation, esthetics, and hydropower generation. All instream uses are considered to be non-consumptive.

Russell et al. (1970) and Wollman and Bonem (1971) assumed that any changes in the real price of water would not have a significant impact on demand over the time horizons considered. However, the National Assessment (U.S. Water Resources Council, 1978) did project that ground-water overdrafts would disappear because "excessive pumping becomes uneconomical as water levels continue to decline." Ideally, it should be possible to include the effect of price on demand, particularly for industrial uses, so that its use as a management strategy can be evaluated. As will be shown, this presents no conceptual problem, though sound estimates of price elasticity for various uses may be difficult to come by.

Figure 2 summarizes the classification of demands used herein. Annual average values are used for withdrawal uses (although the average flow for the highest month or other indicator of more extreme values could be as readily used), the lowest prescribed daily flow is used for minimum flow, and the flow needed to achieve a prescribed water-quality standard on 95% of the days is used for the water-quality flow.

The overall water demand, or requirement, is determined by the flow rates needed to satisfy: 1) instream uses for quality maintenance; 2) instream minimum flow requirements; 3) withdrawal requirements; and 4) consumptive-use requirements. However, as noted above, the total requirement cannot be found by simple addition of these four components. In order to show how these requirements are computed, we must first develop a model of water use and water quality at a use site, which could be a municipality, single industry or group of industries.

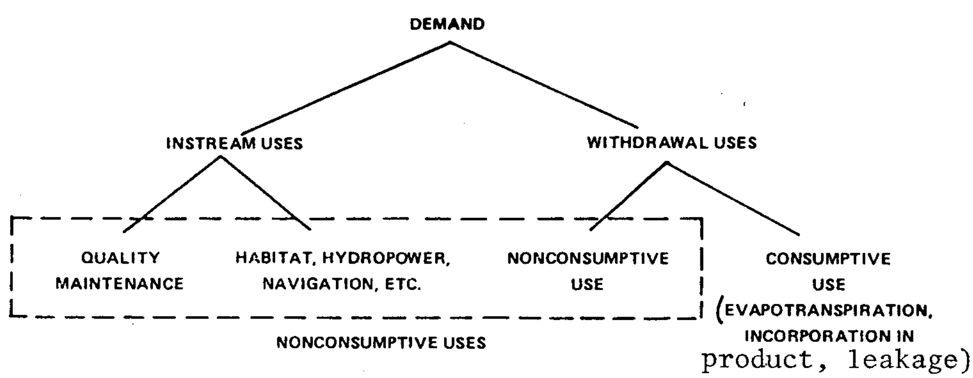


Figure 2: Classification of water demands.

Figure 3 shows a water-use location which obtains its water supply from an adjacent stream and/or from some other source (aquifer, reservoir, or desalting plant). The letters indicate mean flow rates in each segment of the system:

E = rate of supply from off-stream source;

W = rate of withdrawal from stream;

C = rate of consumptive use (includes leakage and other losses);

U = rate of use of water;

J = rate at which waste water is treated;

R = rate of recycling;

D = rate of discharge;

We also define:

k_c = C/U, "consumptive-use factor";

k_r = R/U, "recirculation factor";

k_d = D/U, "discharge factor".

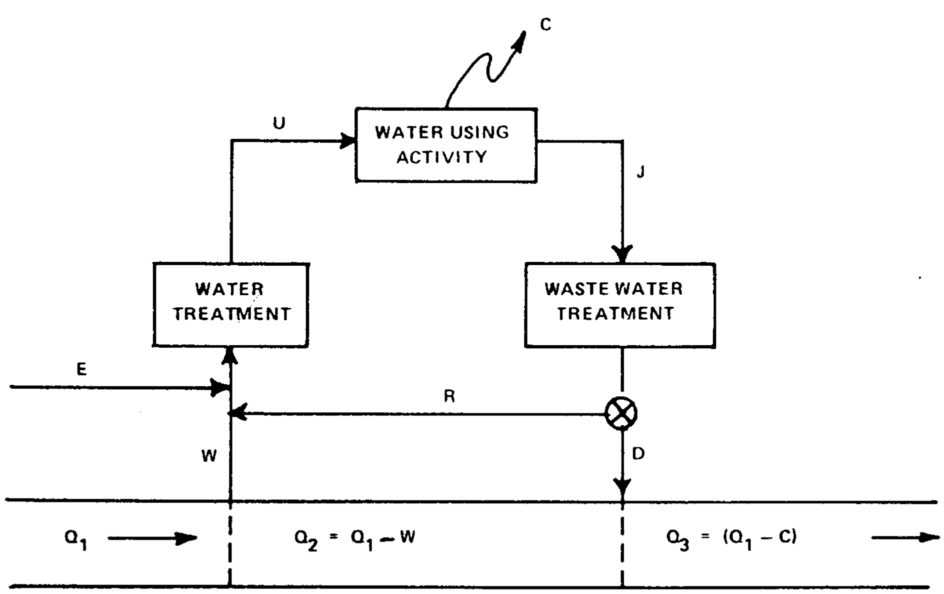


Figure 3: Definition of Water Quantity Terms at Use Sites.

The values k_c and U are forecast, depending on the mix of projected uses. E and k_r are decision variables. With these four variables fixed, the others are readily computed by the following relations:

$$C = k_c U; \tag{1}$$

$$D = k_d U; \tag{2}$$

$$W = U - E - R = (1 - k_r)\, U - E; \tag{3}$$

$$J = U - C; \tag{4}$$

$$D = J - R = W + E - C = (1 - k_r - k_c)\, U. \tag{5}$$

In considering water quality, we are concerned with one contaminant at a time, and concentrations are computed by a steady-state mixing models. Figure 4 shows the mass flow rates, expressed as a concentration (small "c" with a subscript) times a volume flow rate (capital letters as defined above for water flows). In addition, the following terms are defined:

F = mass rate of removal of contaminant in water-treatment process;

A = mass rate of addition of contaminant during use;

G = mass rate of removal of contaminant in sewage-treatment process;

t_f = fractional rate of removal of contaminant in water-treatment process;

t_g = fractional rate of removal of contaminant in sewage-treatment process;

c_1 = concentration of contaminant in stream under design-flow conditions.

A and c_1 are forecast variables, A depending on the type and level of water-using activity projected for the particular use site and time period, and c_1 depending on the hydrologic and land-use conditions projected to exist upstream. t_f, t_g, and c_e are decision variables, the first two depending on the treatment-process alternatives specified by the planner, and c_e depending on the nature of the off-stream source.

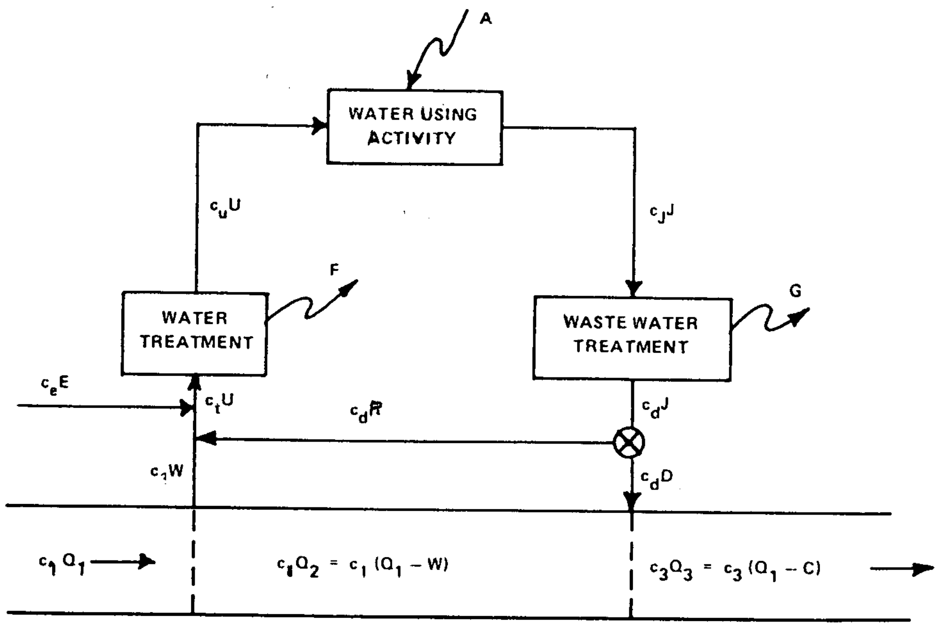


Figure 4: Definitions of Water Quality Terms at Use Sites.

Using mass-balance equations, one can formulate and solve six equations for the six unknown variables. The solution can be expressed as:

$$c_j = \frac{(1 - t_f)\ (c_eE + c_1W) + A}{J - (1-t_f)(1-t_g)R} \quad ; \qquad (6)$$

$$G = t_g\ c_j\ J \quad ; \qquad (7)$$

$$c_d = c_j - \frac{G}{J} \quad ; \qquad (8)$$

$$c_t = \frac{c_dR + c_eE + c_1W}{U} \quad ; \qquad (9)$$

$$F = t_f\ c_t\ U \quad ; \qquad (10)$$

$$c_u = c_t - \frac{F}{U} \quad . \qquad (11)$$

We are now in a position to compute water requirements at use sites, accounting for withdrawal needs, minimum-flow requirements and water-quality requirements. Comparison of requirements with the available flows identifies the existence of surpluses or deficits in any reach. A later section of the report indicates how strategies for dealing with projected deficits can be evaluated via the model.

With reference to the use-site shown in Figure 2, it is required that sufficient flow be available in the stream to satisfy the needs for withdrawal, minimum flows, and water-quality maintenance on 95% of the days. The design flow projected to be available from the basin upstream from the site of withdrawal at some future time is designated "Q_1". The design flow required to satisfy all needs is designated "Q_R", and if $Q_R > Q_1$, a deficit is projected for that reach at that time. We use "Q_{min}" to designate the minimum flows required for habitat, navigation, and power and "Q_q" to designate the flow required for water-quality maintenance. "W" is the flow required for withdrawals and "C" is the rate of consumptive use, as defined earlier. It can be shown that there are four possible cases, and the value of Q_R can be determined for each as follows:

$$\frac{\text{if} \quad (Q_{min} + W) > Q_{min} > Q_q \text{ (case a) then}}{Q_R = Q_{min} + W \,;} \tag{12a}$$

$$\frac{\text{if} \quad (Q_{min} + W) > Q_q > Q_{min} \text{ and } (Q_{min} + W - C) > Q_q \text{ (case b1) then}}{Q_R = Q_{min} + W \,;} \tag{12b-1}$$

$$\frac{\text{if} \quad (Q_{min} + W) > Q_q > Q_{min} \text{ and } (Q_{min} + W - C) < Q_q \text{ (case b2) then}}{Q_R = Q_q + C \,;} \tag{12b-2}$$

$$\frac{\text{if} \quad Q_q > (Q_{min} + W) > Q_{min} \text{ (case c) then}}{Q_R = Q_q + C \,.} \tag{12c}$$

Thus Q_R can be determined if W, C, Q_{min}, and Q_q are known. W is determined by the projected water use (U), the amount of water taken from other sources (E), and the projected amount of recycling (R). C is determined by the projected type and rate of water use (k_c and U), and Q_{min} is projected as a legislative requirement or less formal goal. The value of Q_q is computed for the critical water-quality constituents by first computing the concentration of constituent downstream from the outfall, c_3:

$$c_3 = \frac{C_1\ (Q-W) + c_d\ D}{Q_1 - C} \quad , \tag{13}$$

where c_d is found from Equation 8 and D from Equation 5, then Q_q is given by

$$Q_q = \frac{c^+ C - c_1\ W + c_d\ D}{c^+} \quad , \tag{14}$$

where c^+ is the maximum allowable concentration of the constituent.

In the dissolved-oxygen model developed in the present study (Larson, 1982), c_3 is computed for successively larger values of Q_1 via Equation 13. The value of Q_1 that first gives an acceptable value of c_3 is then taken as the value of the water-quality requirement, Q_q.

Supply - In Wollman and Bonem (1971), supply was aggregated over each region and defined as the annual flow leaving the region that is exceeded in 90%, 95% or 98% of the years. In the Second Assessment, the U.S. Water Resources Council (1978) defined supply as the sum of stream inflow, imports, ground-water overdraft, and runoff generated within each region; long-term average values were used for each component.

Russell *et al.* (1970) used the "safe yield" as the definition of supply for individual municipalities. For run-of-river supplies and reservoirs, this was considered to be the flow available 95% of the time. Apparently they meant "the mean annual flow available in 95% of the years".

By implication, their definition applied to ground-water as well as surface-water sources.

For river-basin planning at the scale proposed herein, it is most practical to separate surface-water sources from ground-water sources. For surface-water sources, which can be either river out-takes or reservoirs, the supply is taken as the mean daily flow exceeded on 95% of the days. This is a more conservative figure than the annual flow values used by Wollman and Bonem (1971) and one that is more meaningful for decision-making.

For ground-water sources, supply is defined in one of two ways, which are appropriate at least for New England. For sources developed in isolated aquifers, the supply is equal to the nominal safe yield estimated from pump tests or estimates of direct recharge. Where pumping the aquifer will induce recharge from an adjacent stream, the supply is the same as defined above for an out-take in the same reach.

Evaluation of Alternatives

Table 1 lists alternative strategies for reducing projected water deficits. The impacts of these strategies on water supply and quality can be evaluated via the proposed model. Complete evaluation would, of course, have to include estimates of the economic costs of each strategy and consideration of environmental impacts and social-institutional implications.

On the supply side, the principal objective is to estimate the effects of alternatives on the design flow (Q_{95}). Approaches to this are simply mentioned herein; more detailed discussion can be found in Dingman and Lindsay (1981). Simple planning-level methods for estimating safe yields of reservoirs and the effect of regulation on downstream low flows can be developed (Dingman, 1981a). Yield of aquifers can also

TABLE 1

Alternatives for Solving Water Resource Problems

To Increase Yield (supply)	To Decrease Requirements (demand)
Reservoir construction	Waste water treatment
Ground water extraction	Recirculation
Conjunctive use of ground water and surface water	Reduction of losses
Water importation	Water pricing
Desalination	Water use regulation/conservation
Watershed management	Growth control
(Weather modification)	

generally be estimated for planning purposes (e.g., U.S. Army Corps of Engineers, 1976; Hall, 1979; Dingman and Lindsay, 1981). Methods like that of Jenkins and Taylor (1974) can be applied to estimate river recharge of aquifers and effects of pumping on stream flows where this is important.

Water-importation schemes can be evaluated by applying the reservoir or ground-water yield estimates at the water source, and desalination can be straightforwardly evaluated based on the design yield of the plant. The supply (Q_{95}) increases due to watershed management may be difficult to assess due to lack of data. Most studies of yield increases due to vegetation manipulation appear to be concerned largely with increases in annual or seasonal means. However, a few studies are available that explore effects on flow-duration curves (e.g., Hornbeck and Federer, 1975). Where watershed management is viewed as an important component of the strategy, it may be necessary to conduct studies using watershed models that estimate daily streamflows, like the BROOK model of Federer and Lash (1978).

Weather modification is listed in Table 1 largely for completeness. It is likely that the legal problems and the effect on climatic and hydrologic data records will be so pernicious as to rule it out as a water-management strategy. Again, watershed models can be used to estimate the changes in impacts produced by projected changes in precipitation or snowmelt regime if this is to be seriously considered.

From the discussion of water requirements, we see that any strategies that reduce E, W, C, Q_{min} or Q_q below some originally projected value will reduce projected water requirements. (E + W) and C are directly related to the projected population and type and level of industrial use. Thus any strategies that control growth will affect those quantities. In addition, (E + W) are directly affected by the degree of recycling and the consumptive-use factor. C can be controlled by changing the mix of projected water uses and controlling leakage. Q_q is controlled largely by the projected level and type of water use, the degrees of water and waste-water treatment, the degree of recycling, and the nature of the flows from up-basin. These effects are all built in to the model as described above.

Clearly, any projected effects of regulation or conservation on water use can be accounted for by simply adjusting the projected values of U. Price effects can be evaluated by simple models of the form:

$$U = \delta P^{-\pi} \quad , \tag{15}$$

where P is price per unit of water, δ is an empirical constant, and π is the demand elasticity of water use appropriate for the projected uses.

Selection of Water-Quality Indicators

In an earlier phase of this study, Dingman and Lindsay (1981, App. B) determined that dissolved oxygen, phosphorus, and suspended sediment are the water-quality constituents most critical for water-resource planning in New Hampshire and in New England generally. Chloride and total dissolved solids, while not usually of direct water-quality concern themselves, are generally present in amounts proportional to other undesirable substances, and are thus often useful indicators of water quality. The modeling effort in this project has concentrated on dissolved oxygen and phosphorus, as described in detail by Larson (1982) and Green (1982) and discussed later in the present report. Sediment was not included for the reasons discussed below.

The concentrations of many species of dissolved solids in rivers are typically inversely related to streamflow rate. This means that critical concentrations tend to occur at low flows, and thus that concentrations of chloride or total dissolved solids associated with the Q_{95} flow can be considered good indicators of water-quality conditions. The use of the water-quality equations developed earlier in this report (Equations 6-14) is straightforward when constituents of this type are to be modeled.

Those equations may also be readily used to model any of the various forms of dissolved phosphorus. However, the inverse relation between phosphorus concentration and streamflow rate usually exists only in a stream reach immediately downstream from a significant point source of phosphorus, such as untreated or partially treated sewage. However, if phosphorus is contributed by non-point sources such as agricultural or urban runoff, concentration may tend to increase as stream discharge increases. Furthermore, physical, chemical, and biological processes within the stream channel may alter phosphorus concentrations in complex ways. Phosphorus concentrations therefore often show little relation to streamflow rate, as was found in the detailed field studies undertaken during the present project (Green 1982). Thus while Equations 6-14 are perfectly applicable to phosphorus, the concentrations associated with the steady-state Q_{95} flows in the model may not be the most critical from a planning viewpoint. This suggests that further modeling studies of phosphorus may be needed to supplement the approach adopted here.

In modeling dissolved oxygen, a biochemical oxygen demand (BOD) should be used as the constituent in Equations 6-14. As explained later in this report and in the detailed study by Larson (1982), the BOD concentration in the river channel is related to a dissolved-oxygen concentration, and it is dissolved oxygen that is modeled through the river network. For dissolved oxygen, which is positively related to water quality, low concentrations typically occur at low streamflows, because low flows mean low velocities and hence low rates of reaeration. In addition, low flows usually occur in summer, when temperature is high and the water can hold less oxygen, or in winter, when ice cover restricts reaeration. Thus a steady-state low-flow model such as the one developed here is well adapted for use with dissolved oxygen as a critical water-quality constituent.

Suspended sediment concentration usually increases markedly with streamflow rate, so a steady-state low-flow model is not suitable for identifying critical concentrations of this constituent. Any attempt to model suspended sediment must await extensive research on sediment yield in the region.

Streamflow Modeling

As noted above, the model represents a stream network as links and nodes (Figure 1). Nodes represent either tributary junctions or locations where wastewater discharges into a river. (In the operational version of the model (Larson, 1982), a node can also represent a location where there is a marked change in stream width, depth, or velocity, but no addition of flow). Thus nodes generally represent point sources of water, and the model must contain an algorithm for increasing the design flow at such nodes.

In the present version of the model (Larson, 1982), nodes representing tributary junctions are assigned two design-flow (Q_{95}) values, one of which is for the mouth of the tributary and the other of which is for the main stream immediately above the junction. These values are entered as input data, and are based on gaging-station records (Dingman and Capsis, 1981). The two values are added to give the Q_{95} just below the junction.

While this procedure is not strictly correct statistically (Warn and Brew, 1980), it is most practical for manipulation of flow data and does not introduce serious error at least in New Hampshire. This is indicated by the following empirical relationship which was found for the 21 unregulated gaged streams in the state:

$$Q_{95} = 0.055\ A_D^{1.25} \tag{16}$$

(r^2 = 0.83; std. error = 0.4297 natural log units),

where Q_{95} is in ft^3/s and A_D is drainage area in mi^2. Statistical tests indicate that the exponent in this relation is not different from 1.00 at the 5% significance level. Thus, for practical purposes, Q_{95} is very nearly proportional to drainage area and the process of adding Q_{95} values at tributary junctions does not seriously distort the actual relations. In any case, input values can be readily adjusted to maintain the correct values through the river network.

The computation of flows at use-site nodes is the same as at tributary nodes, except that the Q_{95} just downstream of the point of discharge is the sum of the Q_{95} just upstream of the discharge and the rate of discharge, D, for the use site. This procedure is statistically appropriate if D is in fact constant, as the program assumes. If D varies widely and is of significant magnitude relative to Q_{95}, this procedure will produce distorted estimates of Q_{95} below the node.

In the present version of the model (Larson, 1982), the Q_{95} computed as described above for a tributary or use-site node is considered to apply throughout the reach below each node. Thus there is in effect no "non-point source" contribution of water in the present model. However, some aspects of the downstream increases of Q_{95} were addressed as part of the research. Under the assumption that Q_{95} increases with drainage area according to Equation 16, the preliminary empirical study described in Appendix 2 yields the following relation between Q_{95} and distance downstream of a node, x:

$$Q_{95}(x) = Q_{95}(0) + 0.085\ x^{1.3} \quad , \qquad (17)$$

where $Q_{95}(0)$ is the design flow just below the upstream node, x is down-valley distance below the node (mi), and the discharges are in ft^3/s.

Dissolved-Solids Modeling

The present operational version of the model (Larson, 1982) does not include modeling of dissolved solids, but the problem has been examined theoretically and in the extensive field study of Green (1982).

Elementary mass-balance considerations produce the following relations for discharge and dissolved solids at a node (Figure 5):

$$Q_C = Q_A + Q_B \qquad (18)$$

$$c_C = \frac{c_A\ Q_A + c_B Q_B}{Q_C} \quad , \qquad (19)$$

where c's represent concentrations, Q's represent discharges, and the subscripts refer to the locations indicated in Figure 5. Equation 18 is the relation for combining flows at a node, discussed in the preceding section.

While Equation 18 and 19 are based on fundamental physical relations that are true at any instant, difficulties may arise when they are incorporated in a design-flow model: Warn and Brew (1980) showed that these equations are not in general true if the discharges represent statistically-defined design flows such as Q_{95}. However, it was shown in the preceding section that Equation 18 is approximately true for Q_{95} in unregulated streams in New Hampshire (Equation 16). Thus Equation 19 appears to be

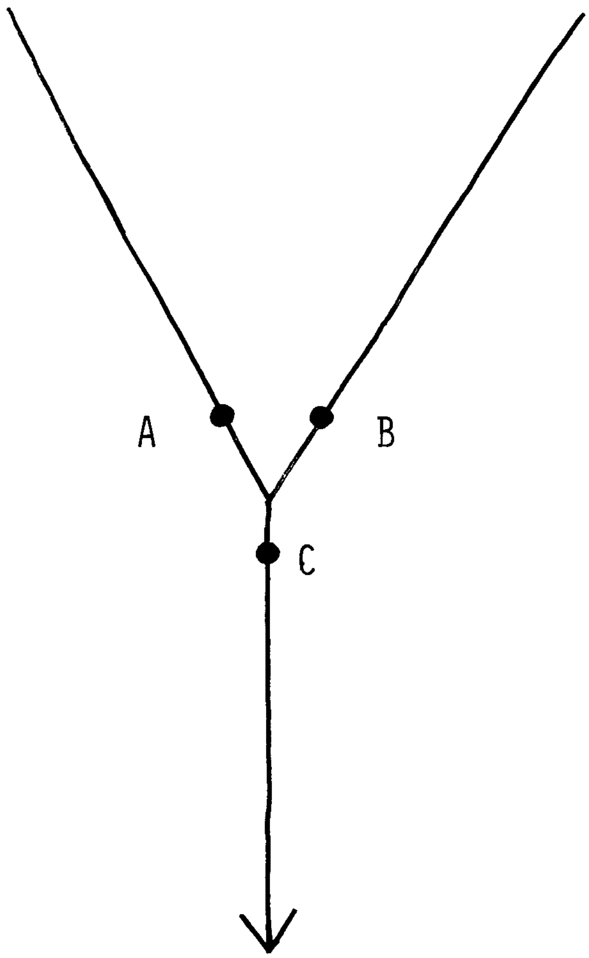


Figure 5: Definition of locations for mixing equations (Equations 18-19).

an acceptable basis for modeling design concentrations at nodes; in any case it is difficult to conceive of an alternative approach that would be more appropriate for a model of the type developed here.

In general, non-point sources of streamflow and physical, chemical, and biological processes in the stream itself will cause the concentration of a dissolved-solid constitutent to change in the downstream direction. Thus the concentration in the design flow arriving at a node will be different from that leaving the node next upstream from it. To account for this, equations are derived in Appendix 3 for the case when the design flow increases downstream (the usual situation):

$$c_D = c_b + \frac{Sr_s x + Q_o(c_o - c_b)}{Q_D} \quad , \quad Q_o \leq Q_D \quad ; \tag{20}$$

and for the case where it decreases downstream:

$$c_D = c_o + \frac{Sr_s x}{Q_D} \quad , \qquad Q_o \geq Q_D \quad . \tag{21}$$

In Equations 20 and 21, c_D is the concentration in the flow arriving at the downstream node, c_b, is the concentration in the non-point-source flow entering the stream, r_s is the rate at which the constituent is added to the flow as a result of physical, chemical, and biological processes in the stream (mass per unit distance of channel per unit time), x is down-valley distance, S is stream sinuosity, Q_o is stream discharge leaving the upstream node, and c_o is the concentration leaving the upstream node.

Green's (1982) approach to representing downstream changes in concentration is slightly different from Equation 20:

$$c_D = \frac{Sr_s x + c_o Q_o + A_b L_b}{Q_D} \quad , \tag{22}$$

where A_b is the area of watershed contributing streamflow between the upstream and downstream nodes and L_b is a non-point-source loading factor for that area (mass per unit area per unit time). The similarity between the two approaches can be seen by rearranging Equation 20 to:

$$c_D = \frac{Sr_s x + c_o Q_o + C_b(Q_D - Q_o)}{Q_D} \quad ; \qquad (23)$$

from which it is seen tha $A_b\ L_b = c_b(Q_D - Q_o)$.

Dissolved-Qxygen Modeling

Larson (1982) reviewed approaches to dissolved-oxygen modeling appropriate to the objectives of this study. Most models of dissolved oxygen in rivers are based on the Streeter-Phelps (1925) equations, and that approach has been used here. However, other steady-state methods, such as the WIRQAS model developed by the U.S. Geological Survey (McKenzie et al., 1979), are equally suitable for a planning model.

As noted earlier, when dissolved oxygen (DO) is the water-quality constituent of concern, biochemical oxygen demand (BOD) is the contaminant in the use-site equations (Equations 6-11). There is no general means for relating BOD concentration to DO concentration, so unless other information is available, one can conservatively assume a zero DO concentration in effluent from a use site. When this effluent is introduced into the stream, the concentrations of BOD and DO immediately downstream from the node are computed via the standard mixing equation (Equation 19). These values become the initial values of BOD and DO for computing DO concentrations downstream of the node (L_o and c_o, respectively, in Equations 23, 25, and 26 below).

The traditional Streeter-Phelps approach assumes that DO in a river reach below a source of biodegradable waste depends on the balance between the rate at which carbonaceous waste is oxidized and the rate at which new oxygen is dissolved in the water. (Other oxygen-regulating processes can be accounted for in the equations if appropriate). By assuming that both deoxygenation and reaeration are first-order rate processes, an equation relating DO concentration and travel time downstream from the node can be derived:

$$c_T = \frac{K_1 L_o}{K_2 - K_1} [\exp(-K_1 T) - \exp(-K_2 T)] + c_o \exp(-K_2 T) \quad , \qquad (23)$$

where c_T is DO concentration at travel time T, k_1 is the deoxygenation rate coefficient, k_2 is the reaeration rate coefficient, and L_o and c_o are the mixed initial concentrations of BOD and DO just below the upstream node, respectively.

Equation 23 typically defines an "oxygen-sag curve" of the form shown in Figure 6. Note that travel time in that equation is readily translated into downstream distance, X, by assuming a constant flow velocity.

$$X = v_{95} T \quad , \qquad (24)$$

where v_{95} is the flow velocity at the design discharge, Q_{95}. The location of the point of minimum DO concentration ("sag point") can be found by computing the travel time at which this point occurs, T_c, from

$$T_c = \frac{1}{K_2 - K_1} \ell n \left[\frac{K_2}{K_1} \left(1 - \frac{c_o (K_2 - K_1)}{L_o K_1} \right) \right] , \qquad (25)$$

and using Equation 24. If the sag point occurs upstream of the next node downstream, the DO concentration at the sag point is given by

$$c_c = \frac{K_1 L_o}{K_2} \exp(-K_1 T_c) \quad . \qquad (26)$$

(Derivation of Equations 23, 25, and 26 is given by Larson (1982).)

If the location of the sag point computed from Equation 25 is

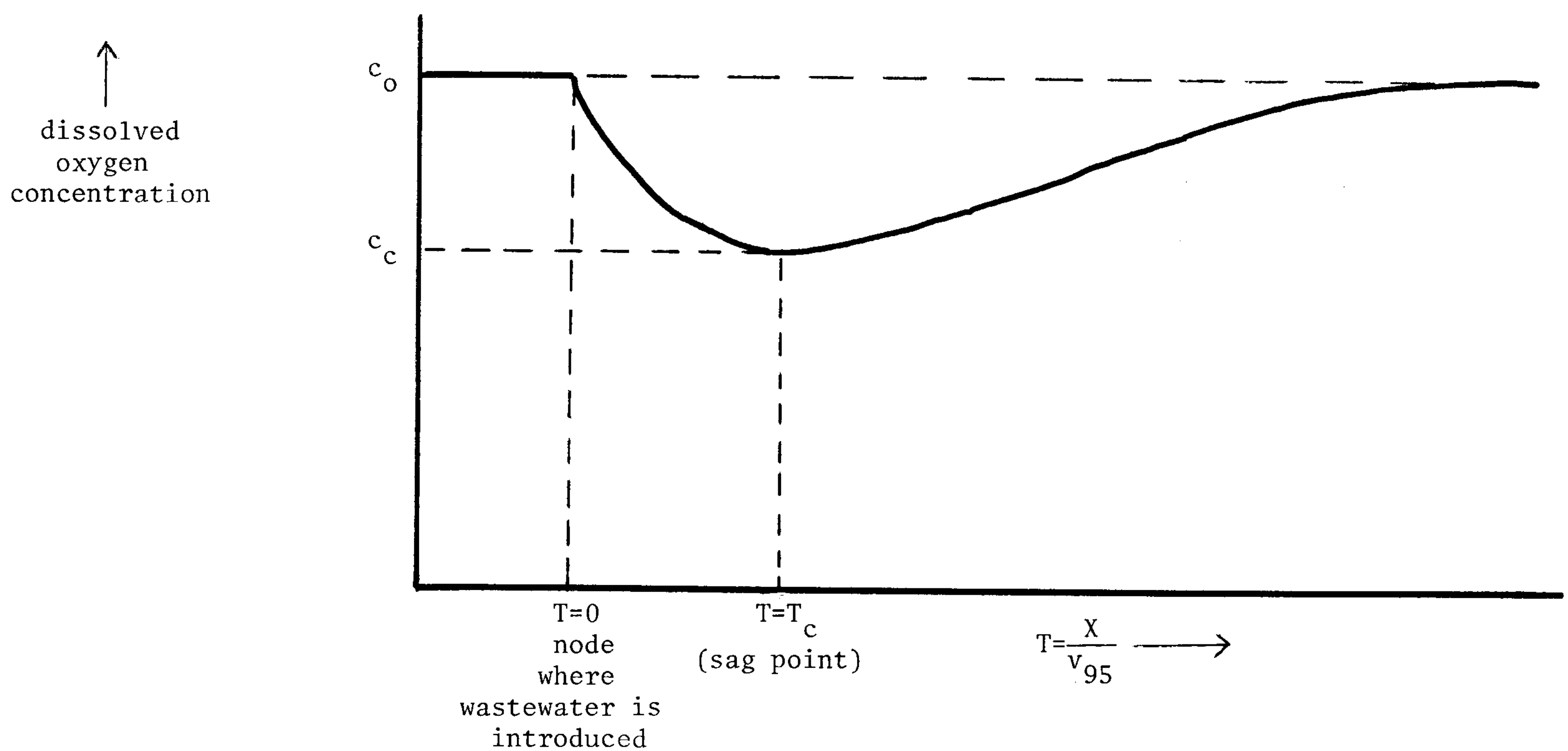


Figure 6: Typical oxygen-sag curve.

downstream of the next downstream node, the lowest DO concentration in the reach occurs immediately above that node. The magnitude of that concentration is computed as

$$c_c = \frac{K_1 L_o}{K_2 - K_1}\left[\exp\left(-K_1 \frac{X}{v_{95}}\right) - \exp\left(-K_2 \frac{X}{v_{95}}\right)\right] + c_o \exp\left(-K_2 \frac{X}{v_{95}}\right), \quad (27)$$

where X is the distance between the two nodes. The value of c_c, whether computed by Equation 26 or 27, is analogous to the value of c_3 computed for dissolved solids via Equation 13. However, for dissolved oxygen, a water-quality requirement exists (_i.e._, $Q_q > 0$) when c_c is _less_ than a desired value (c^+) rather than greater than that value.

In order for the computations of DO concentration to proceed downstream, the DO and BOD concentrations at the end of each reach must be calculated. For DO, this is done via Equation 27, while "Phelps' Law" is applied for BOD:

$$L_X = L_o \exp\left(-K_1 \frac{X}{v_{95}}\right) \quad , \quad (28)$$

where L_X is the concentration just above the downstream node.

The report of Larson (1982) describes in detail how the above computations are incorporated in the logic of a computer program. In that program, it is assumed that no BOD is contributed by non-point sources of streamflow; in fact, it is assumed that Q_{95} is constant between nodes, with an appropriate step increase at each node proceeding downstream. Other practical aspects of this model, including estimation of K_1 and K_2, are also discussed by Larson (1982).

IV. IMPLEMENTATION OF THE MODEL: DISSOLVED-OXYGEN MODELING OF THE MERRIMACK RIVER BASIN, N. H.

Introduction

A major portion of the present research project has involved the implementation of the conceptual planning model developed in Section III. A complete description of this effort can be found in Larson (1982). The present section gives an overview of her work, with particular attention to those aspects that relate to the general problems addressed in Sections II and III.

Overview of Modeling Study

Larson's (1982) model was written to simulate summertime low-flow (Q_{95}) dissolved-oxygen conditions in the Merrimack River Basin above Manchester, N. H. (Figure 7). The modeled river network includes the mainstem Merrimack River and two orders of tributaries, and contains 39 use-site nodes and six tributary nodes. Only point sources of BOD are included, as non-point sources are thought to be insignificant under low-flow conditions. The model, called OXYGEN.PAS, retains most of the features of the conceptual model described in Section III, and was developed on the University of New Hampshire DEC-10 system using the computer language PASCAL.

The output of the model is a list of values of the dissolved-oxygen concentrations at each node, representing the Q_{95} design-flow and summer temperature conditions for a set of water-use, treatment, and design-flow conditions provided by the planner for each node. The specific input data are the Q_{95} values for each node and, for each use-site node, either: (1) the appropriate values for the forecast and decision variables

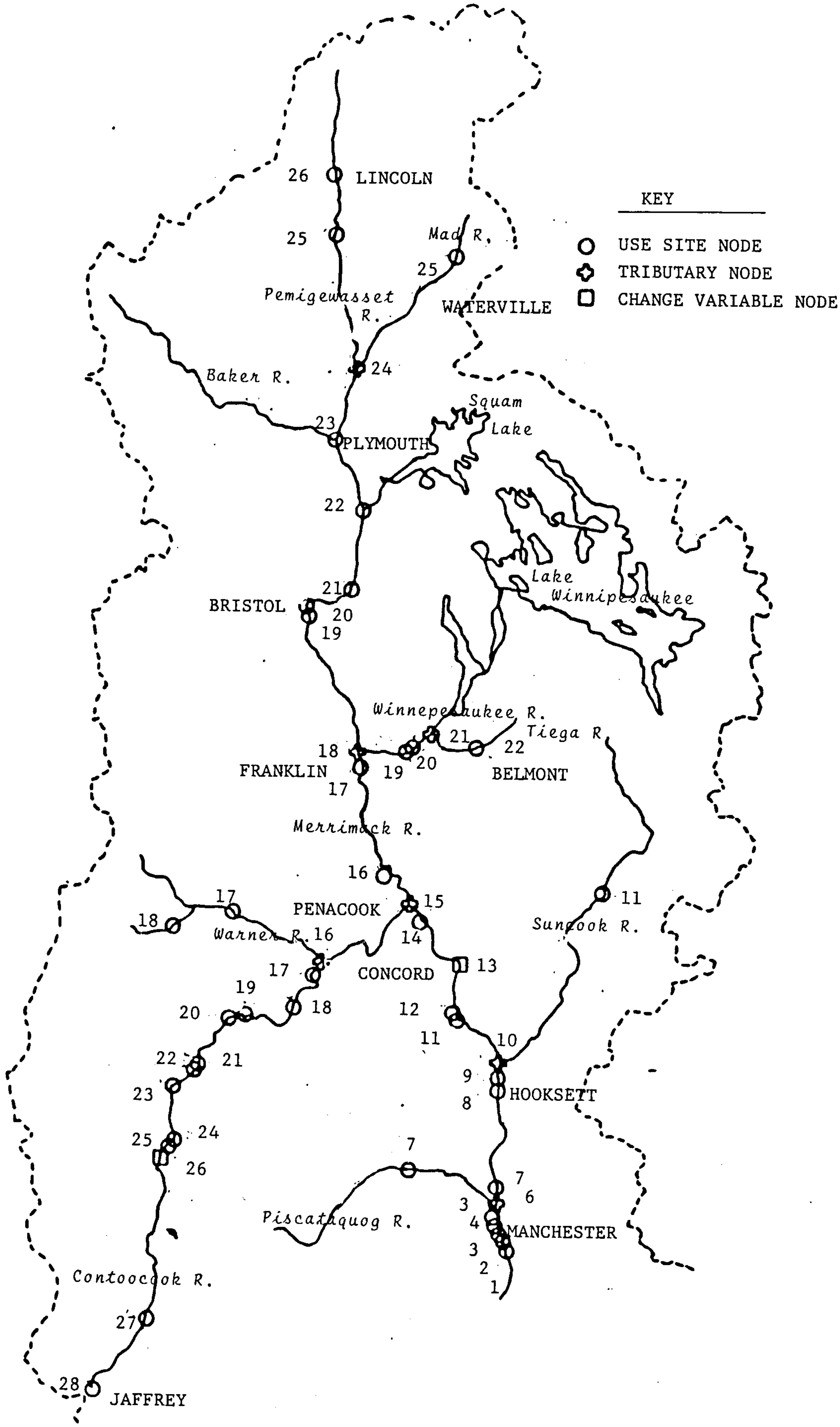


Figure 7: Locations of nodes in study area.

described in Section II; or (2) the value of the wastewater discharge to the river and its concentration of BOD (D and c_d in Equations 1-11). If the first of these options is chosen, the program uses Equations 1-11 to compute the concentration of BOD discharging into the river at each node. Mixing equations (Equations 18 and 19) are used to compute the mixed BOD concentration in the river, and the Streeter-Phelps equations (Equations 23-28) are used to compute the dissolved-oxygen concentration at a sag point if it exists in the reach, or at the downstream end of the reach if there is no sag point.

The essence of the model is the logic for moving through the river network from one use-site node to the next node downstream, applying the use-site, mixing, and Streeter-Phelps equations in the correct sequence. When a tributary node is encountered, the sequence skips to the farthest upstream node on the tributary, continues downstream to the junction, mixes flows at the junction, and continues downstream.

One of the major uncertainties in dissolved-oxygen modeling generally is that of estimating the reaeration coefficient (K_2 in Equations 23 and 25-27). This is generally considered to be a function of stream velocity and depth, but several empirical relations have been given in the literature, and these often give quite different estimates of K_2 for the same conditions. Larson's (1982) model used two alternative estimating equations, which gave only slightly different estimates of dissolved oxygen, but she identified this problem as an important one requiring further research in order to improve the precision of models of this type.

Results

An attempt was made to evaluate the model by comparing computed dissolved-oxygen values with values measured by government agencies in summer when flows were "low" (in general, actual measurements of streamflow rates were not available). For the most part, data on discharges and BOD concentrations at use sites were taken from pollution-discharge permits; at eight sites the discharges were estimated via the use-site equations, making assumptions to compensate for missing data.

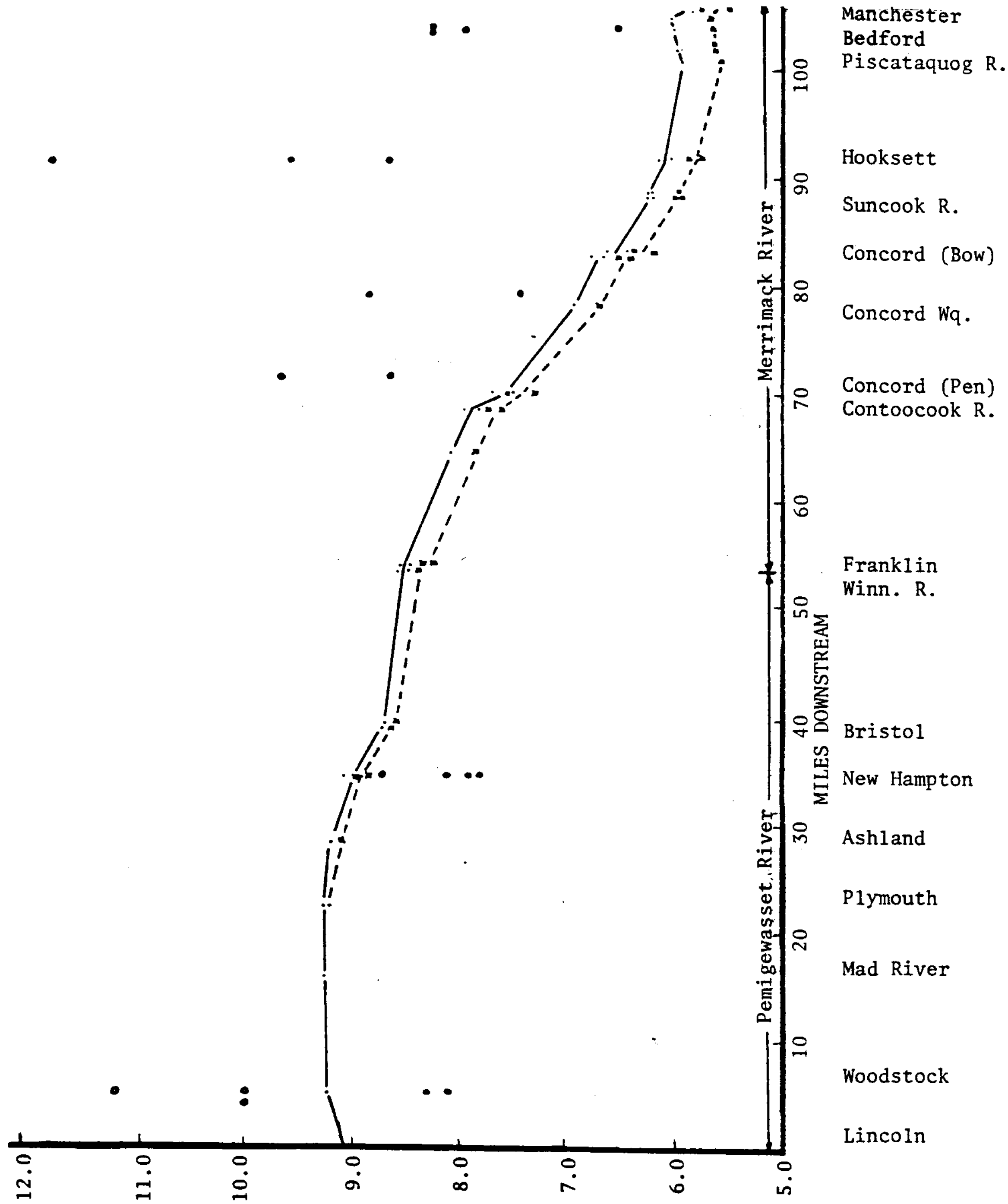
The comparisons between predicted and measured values of dissolved oxygen are shown in Figure 8 for the mainstem Merrimack River and in Figure 9 for its major tributary, the Contoocook. There is a tendency to underestimate the concentrations in the lower reaches of the Merrimack, while for the Contoocook and the upper Merrimack the computed values show reasonable agreement with the few available data. However, it is clear from the intent of the model and the nature of the available data that these comparisons must be interpreted cautiously. The modeled conditions are in a sense "fictitious", in that they are intended to represent a situation that is statistically defined. The measured values do not represent the precise conditions that were modeled and, indeed, the different "low-flow" measurements of dissolved oxygen vary over a fairly wide range at each site. Larson (1982) pointed out that more attention should be given to flow and related conditions when water-quality sampling is done, so that future comparisons of the type attempted here will be more meaningful.

Conclusions

The major result of Larson's (1982) work is to demonstrate the feasibility of constructing river-basin planning models with the features described in Section III, to fulfill the needs described in Section II. Comparisons with existing data suggest that one can confidently employ such models in river-basin planning, and that it should be possible to increase confidence in such models with further research and attention to sampling procedures.

Solid line: Covar's method used to determine reaeration coefficients.

Dotted line: Isaacs-Gaudy equation used to determine reaeration coefficients.



DISSOLVED OXYGEN (mg/l)

Figure 8: Predicted DO profile (mg/l) of the Merrimack River compared to existing data (circles).

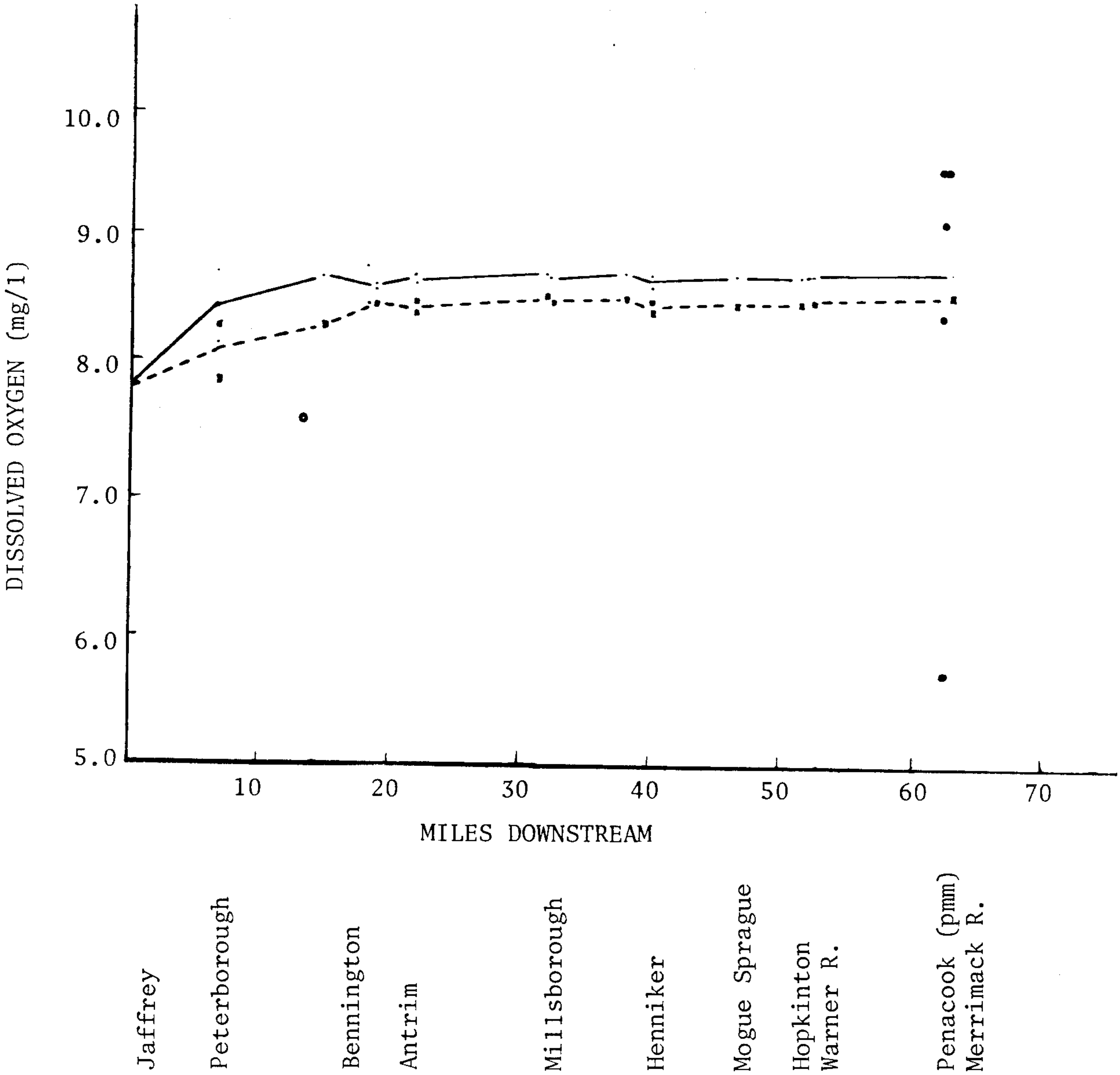


Figure 9: Predicted DO profile of the Contoocook River compared to existing data (circles).

Solid line: Covar's method used to compute K_2

Dotted line: Isaacs-Gaudy equation used to compute K_2

V. FIELD STUDIES OF PHOSPHORUS IN STREAM NETWORKS

Introduction

As noted earlier, phosphorus is the most critical dissolved-solid constituent for water-quality planning in New Hampshire, and in New England generally. Previous studies (see literature review of Green (1982)) have shown that phosphorus occurs in several forms with varying chemical behavior, and that it may be readily absorbed or released by sediment and organisms under various conditions. Thus its presence in dissolved form in a stream depends on a complex set of factors and is highly variable in space and time. The detailed field investigation of Green (1982) was undertaken to provide further knowledge about this important water-quality constituent.

The background, methods, results, and conclusions of the study are described in detail in Green's (1982) thesis. The present section gives a summary of the major elements of his work, and relates his results to the overall project objectives.

Study Area and Approach

Green (1982) studied Dudley Brook, which is in the town of Brentwood in the Piscataqua River Basin in southeastern New Hampshire (Figure 10). Its watershed has an area of 12.8 km^2 (4.94 mi^2) above a gaging station that has been maintained by the U.S. Geological Survey since 1962.

The topography of Dudley Brook watershed is gentle, with a total relief of 35 m (120 ft). Figure 11 shows its mix of soil types and land uses, which are generally representative of the region. A significant source of secondary-treated sewage effluent (from a jail and an elderly-

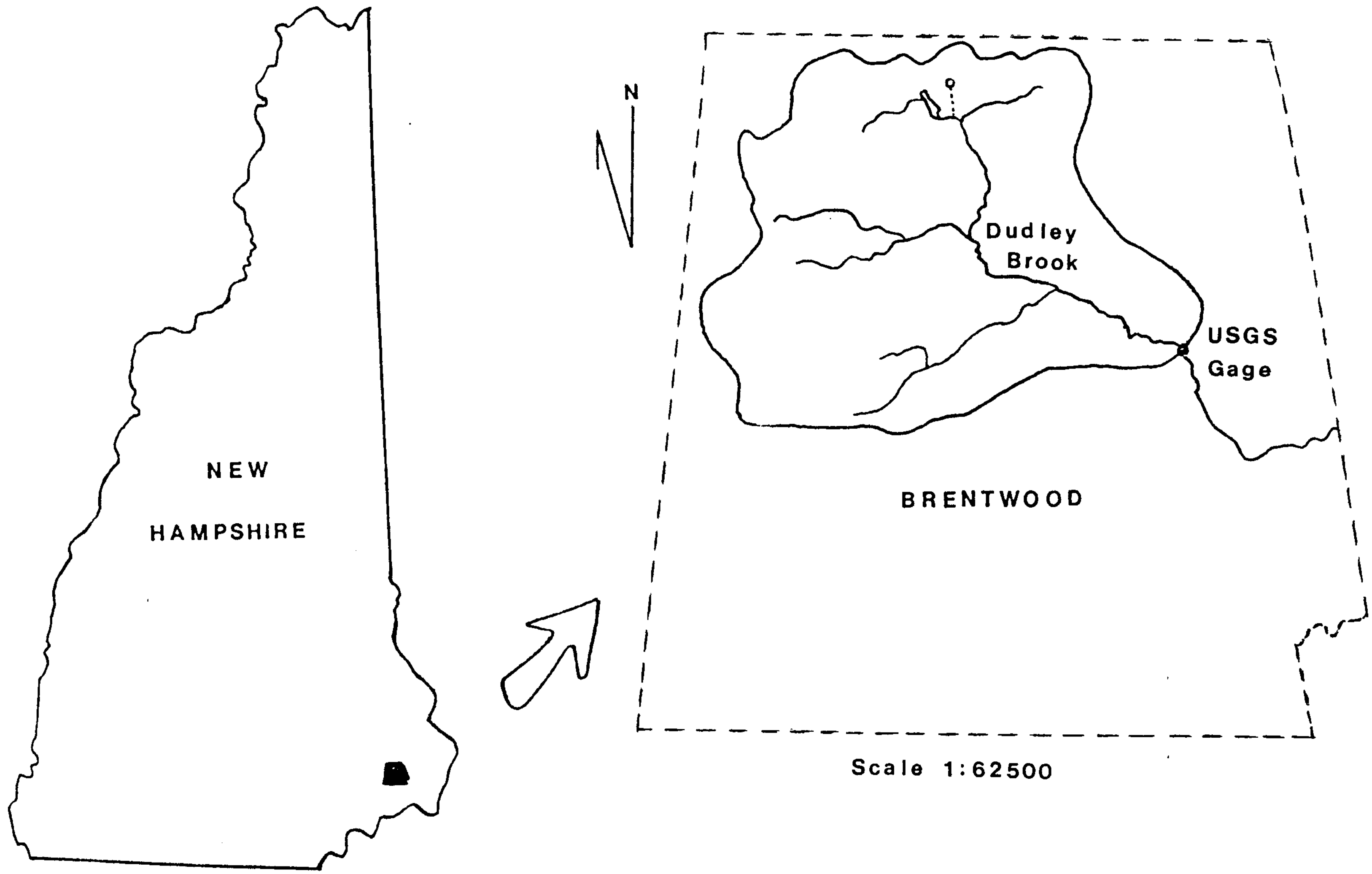


Figure 10: Location of Dudley Brook watershed.

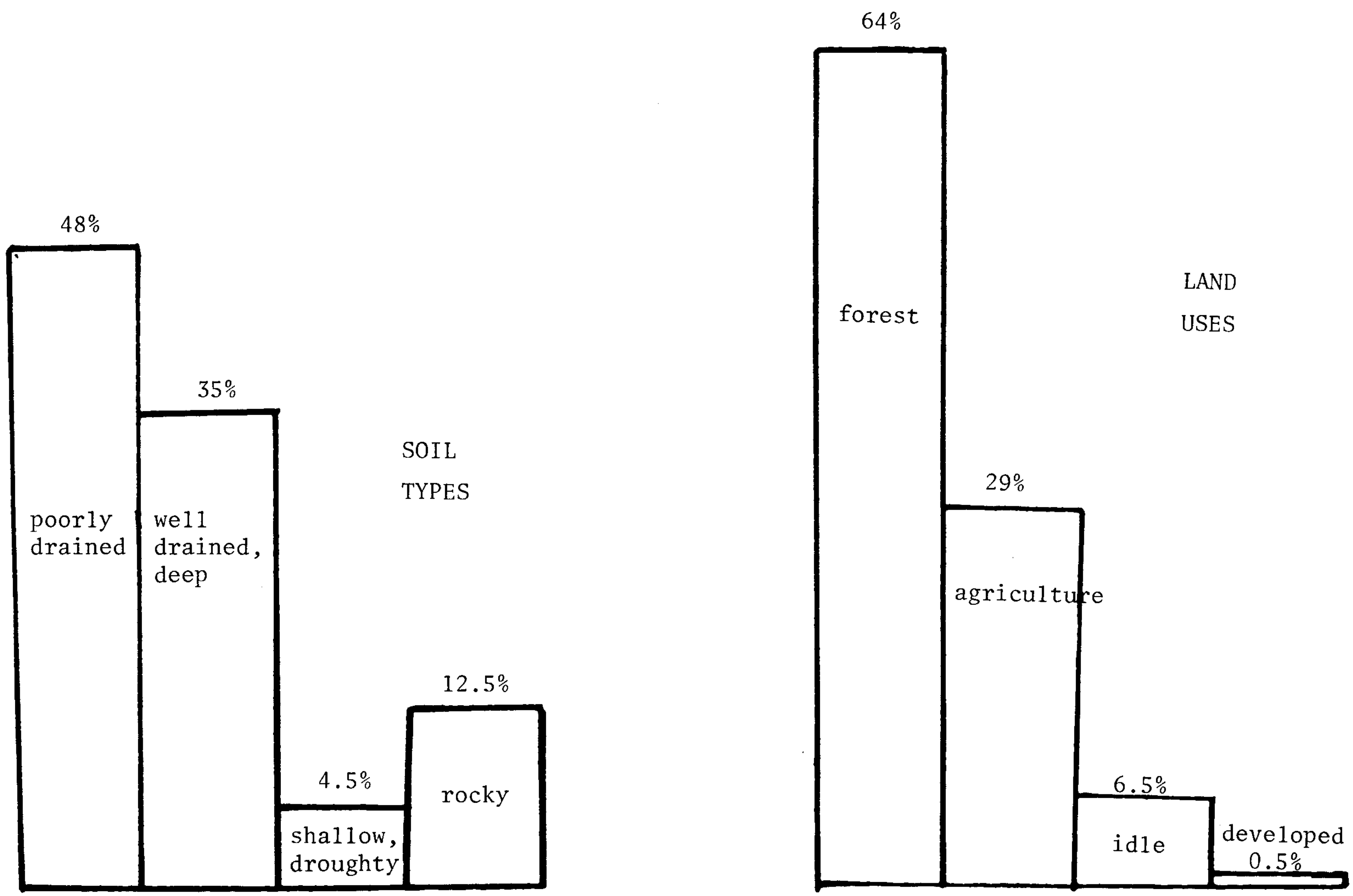


Figure 11: Distribution of soil types and land uses in Dudley Brook watershed.

care facility) is present in the upper basin (Figure 12). Effluent from this source enters upper Dudley Brook continually except for periods of low flow during the growing season, when it is stored in a lagoon if capacity permits. This situation allows observations of phosphorus dynamics to be made under conditions of both high phosphorus loading from a point source and purely non-point-source loading.

Twelve sampling stations were established on Dudley Brook and its tributaries, as shown in Figure 12. These were visited on an approximately weekly basis, with 38 visits between 10 June 1980 and 30 June 1981. Measurements of streamflow rate and some water-quality parameters were made at each visit. Water samples were also collected at each site for laboratory analysis of pH, four forms of phosphorus, and chloride.

Major Results

Concentrations of the various forms of phosphorus showed no significant correlations with streamflow rate at any station, whether or not point-source loading was occurring. On average, phosphorus concentrations tended to decrease downstream when the stream was being loaded with treatment-plant effluent, but not when loading ceased (Figure 13; "FMRP" stands for "filterable molybdate-reactive phosphorus", the most abundant form and the one most readily available for uptake by organisms). However, phosphorus loads (the mass of phosphorus transported in the water column per unit time) tended to increase downstream both with and without point-source loading (Figure 14).

The concentration of total phosphorus in the channel sediments of Dudley Brook tended to decrease downstream, but there was a major peak at Station 3 reflecting the sedimentation of fine sediments and organic matter in a particularly slow-flowing segment of the stream (Figure 15). On the

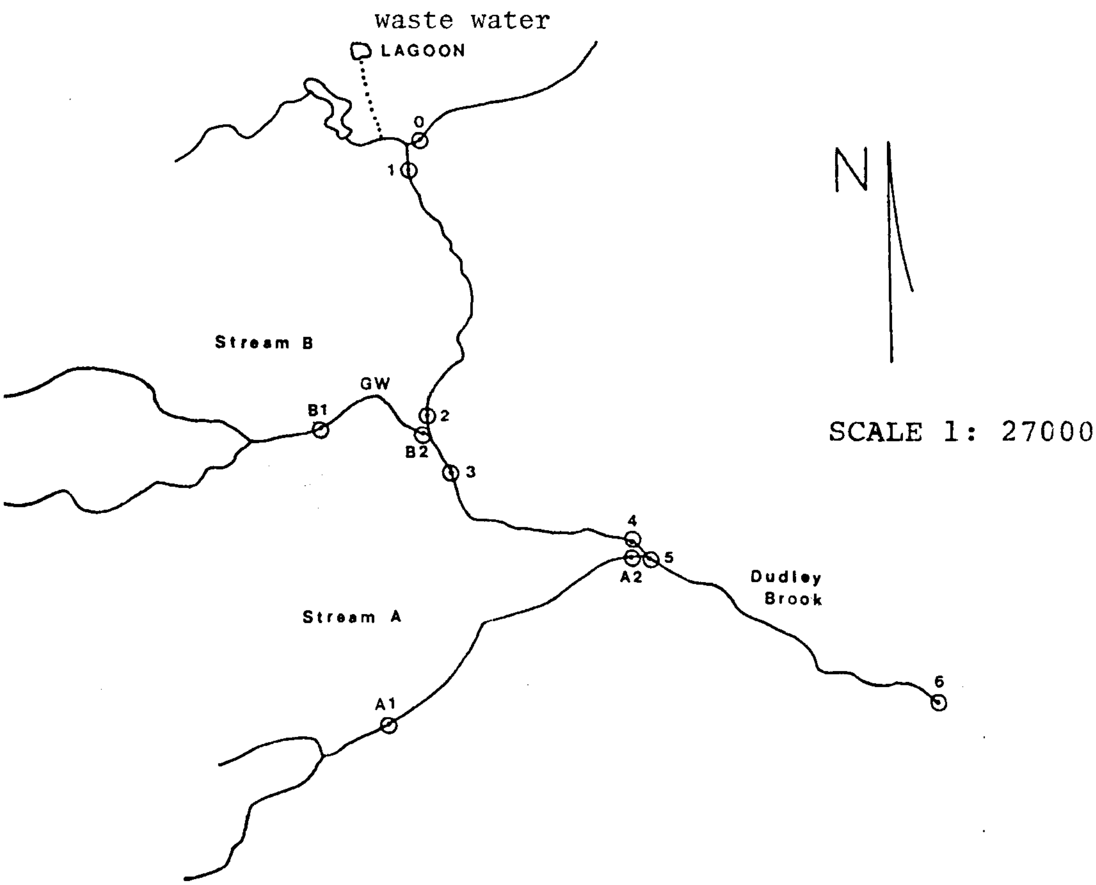


Figure 12: Location of Sampling Stations.

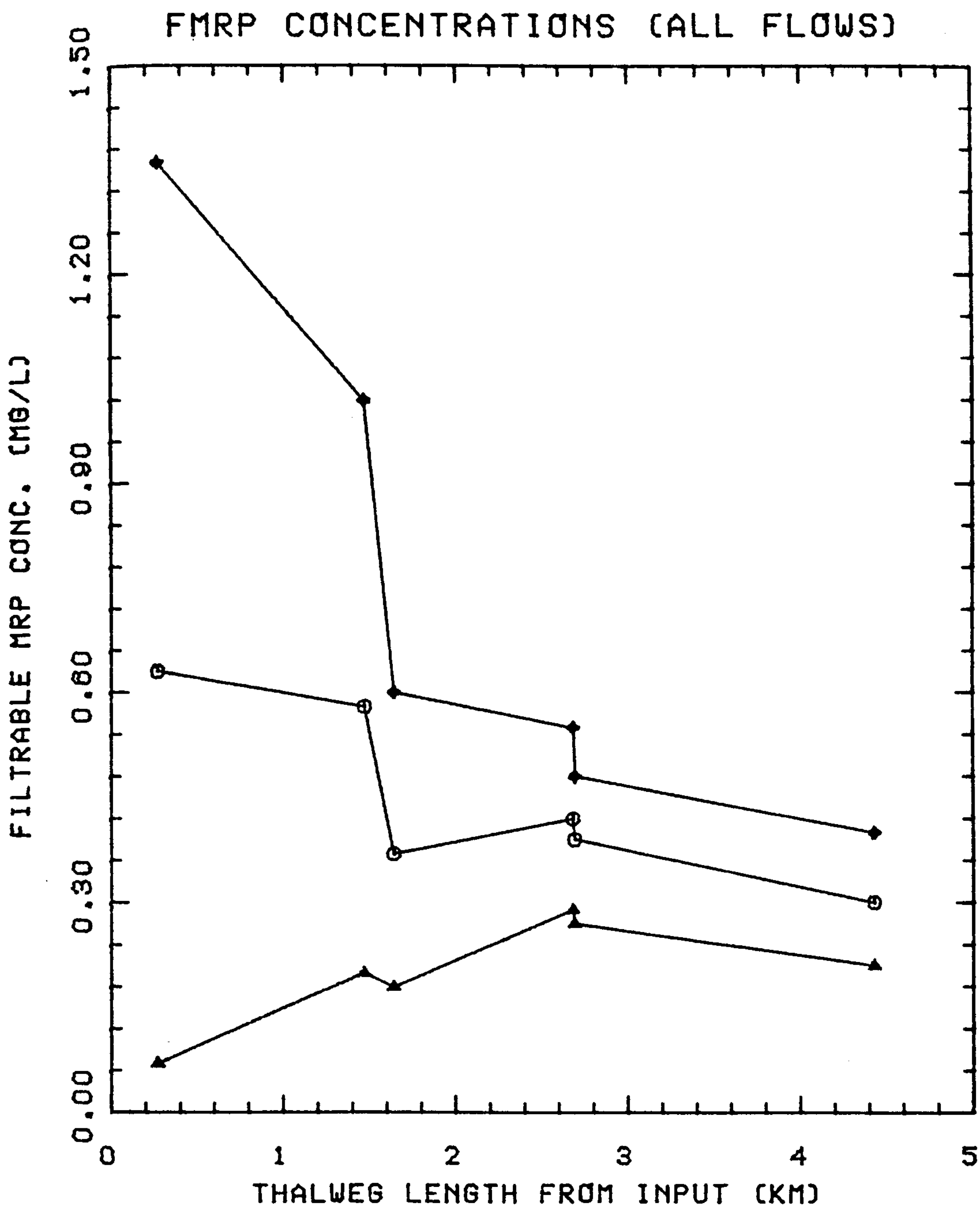


Figure 13: Mean Filtrable MRP Concentrations - All Flows

⊕ = Station Means of All Flows

✦ = Station Means of All Flows With Loading

▸ = Station Means of All Flows Without Loading

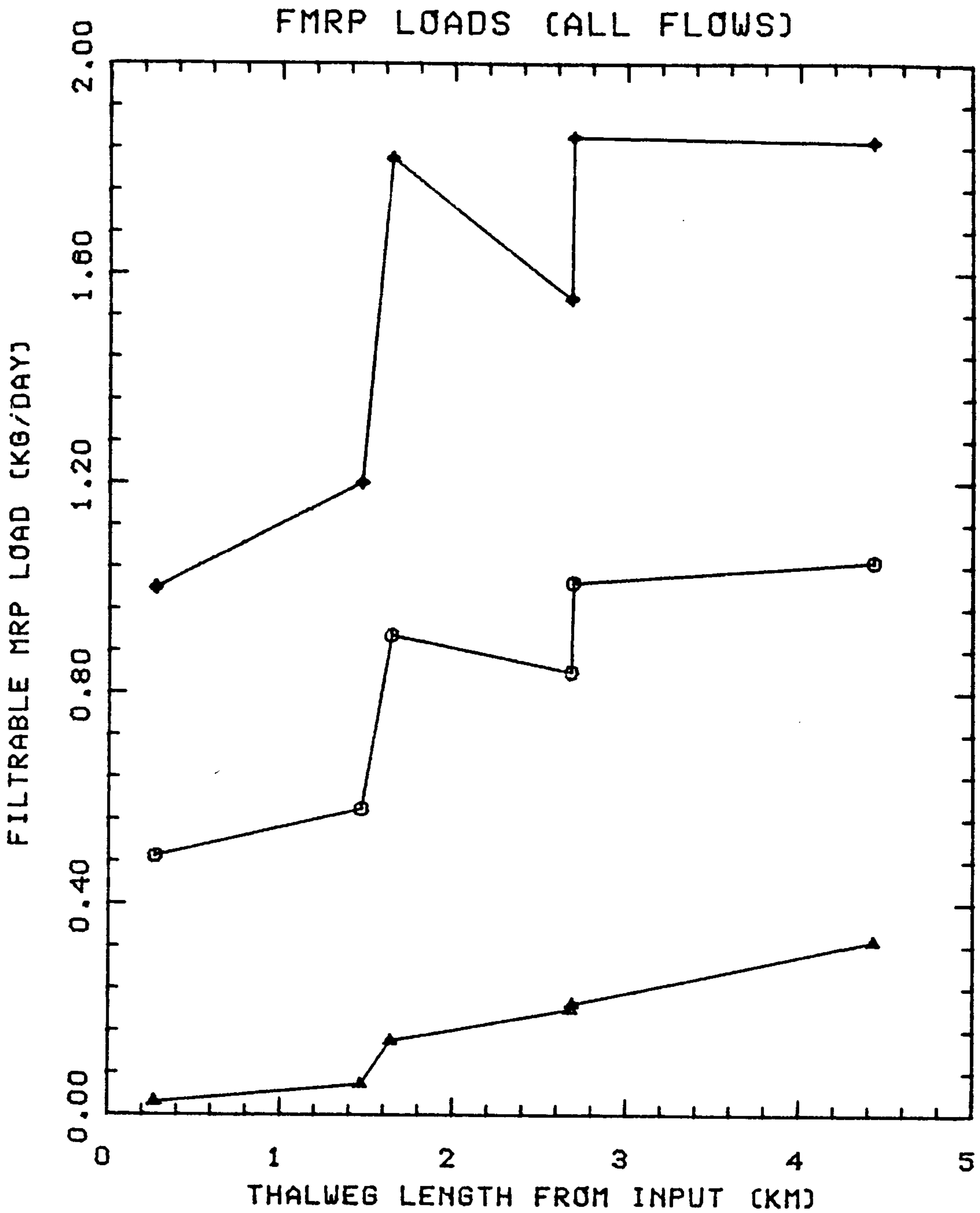


Figure 14: Mean Filtrable MRP Loads - All Flows

⊕ = Station Means of All Flows

✦ = Station Means of All Flows With Loading

▲ = Station Means of All Flows Without Loading

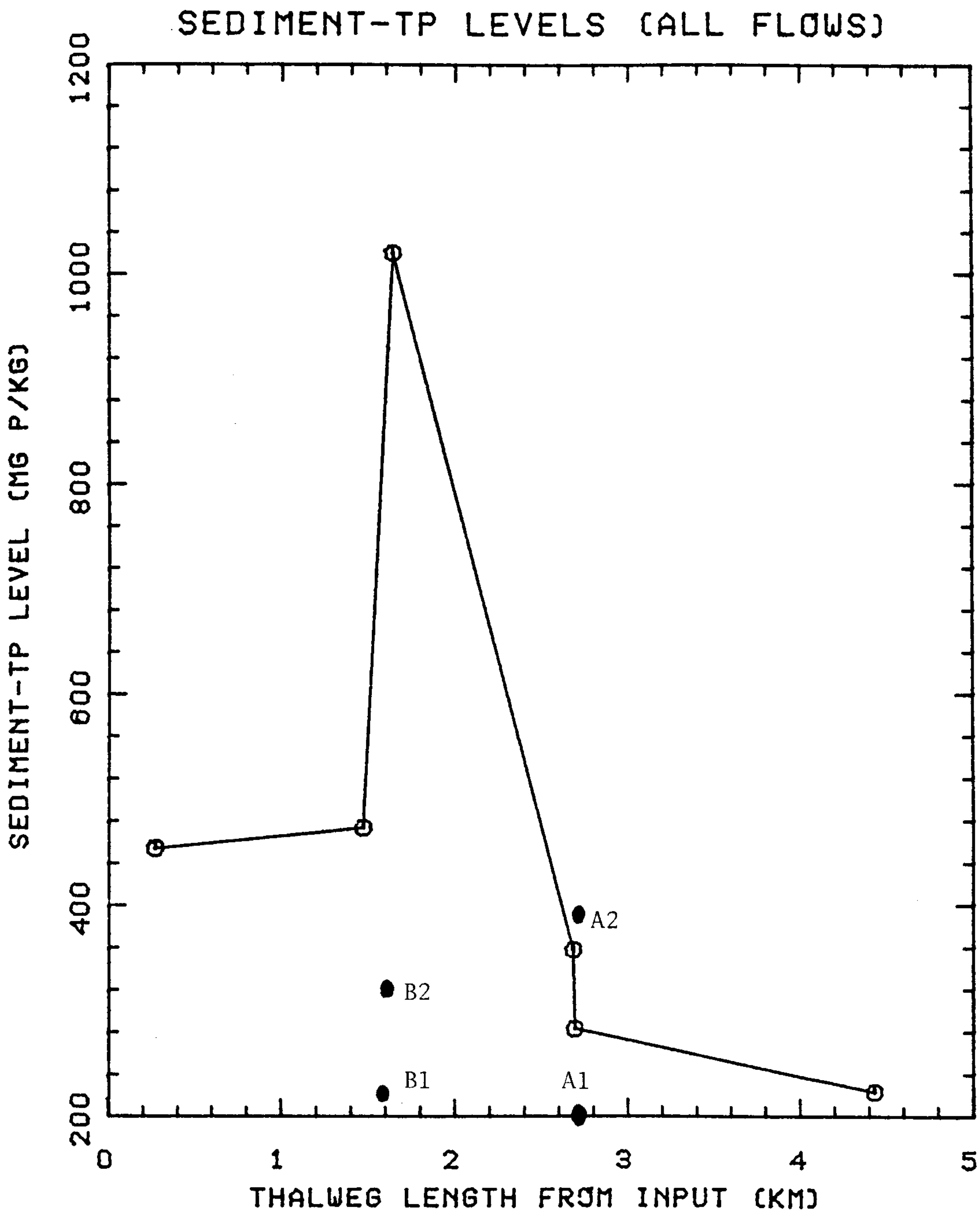


Figure 15: Mean Sediment - Total Phosphorus Levels at Stations - All Flows.

two tributaries, sediment phosphorus increased in the downstream direction. Thus it appears that the main stream pattern reflects the inputs of phosphorus from the waste water and the operation of a "trap" in a slow-flowing reach.

Computation of the phosphorus sink-source term (r_s in Equations 20-23) in three reaches of Dudley Brook showed that, with few exceptions, the channel functioned as a sink (i.e., $r_s < 0$) only when waste-water loading was occurring (Figures 16-19). When there was no loading, the channel virtually always acted as a source (i.e., $r_s > 0$). With loading, the picture was more complicated. For the two upstream reaches (Reaches 1-2 and 3-4), the channel tended to continue as a source at low flows, became a sink at moderate flows, and was again a source at the highest flows. However, for the lowest reach (Reach 5-6), there was no clear pattern to the sign of r_s as a function of flow rate when loading was occurring.

Figure 19 shows the net behavior of the entire channel of Dudley Brook between Stations 1 and 6. When there was no waste-water loading, there was a net transport of phosphorus out of the watershed at a rate roughly proportional to flowrate (reflected by stage height). The stream acted as a sink only when loading occurred.

Conclusions

The results described above suggest that a chemical-kinetic process in part determines whether a given reach acts as a source or sink of phosphorus. In simple terms, a kinetic effect is one in which a chemical constituent moves from regions of higher concentration to regions of lower concentration. In the present case, one can partially test the hypothesis that a reach tends to act as a sink when concentrations in the stream water are high and as a source when they are low. The data for this test are

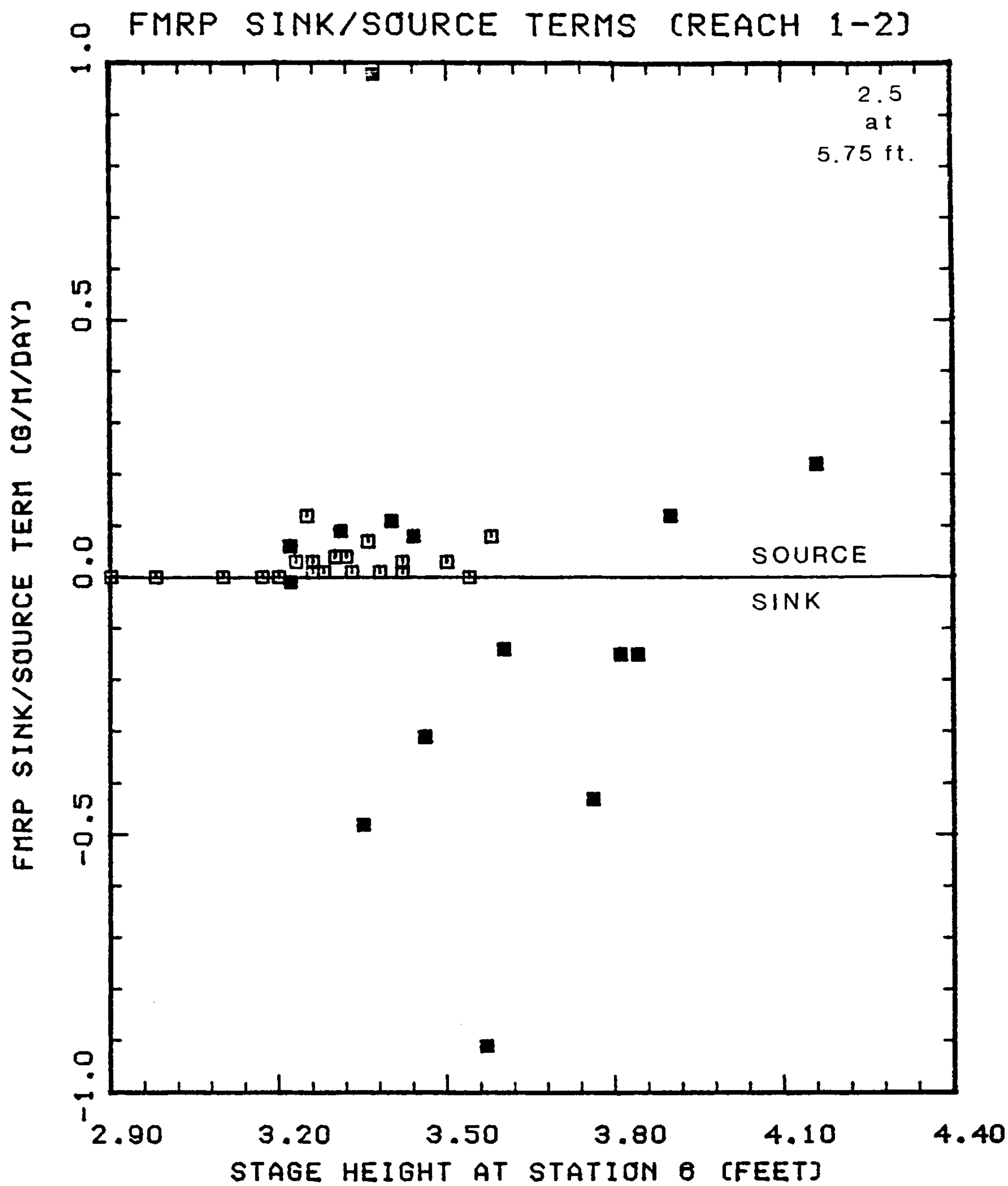


Figure 16: Daily FMRP Sink/Source Terms (Reach 1-2)

■ = With Wastewater Loading

□ = Without Wastewater Loading

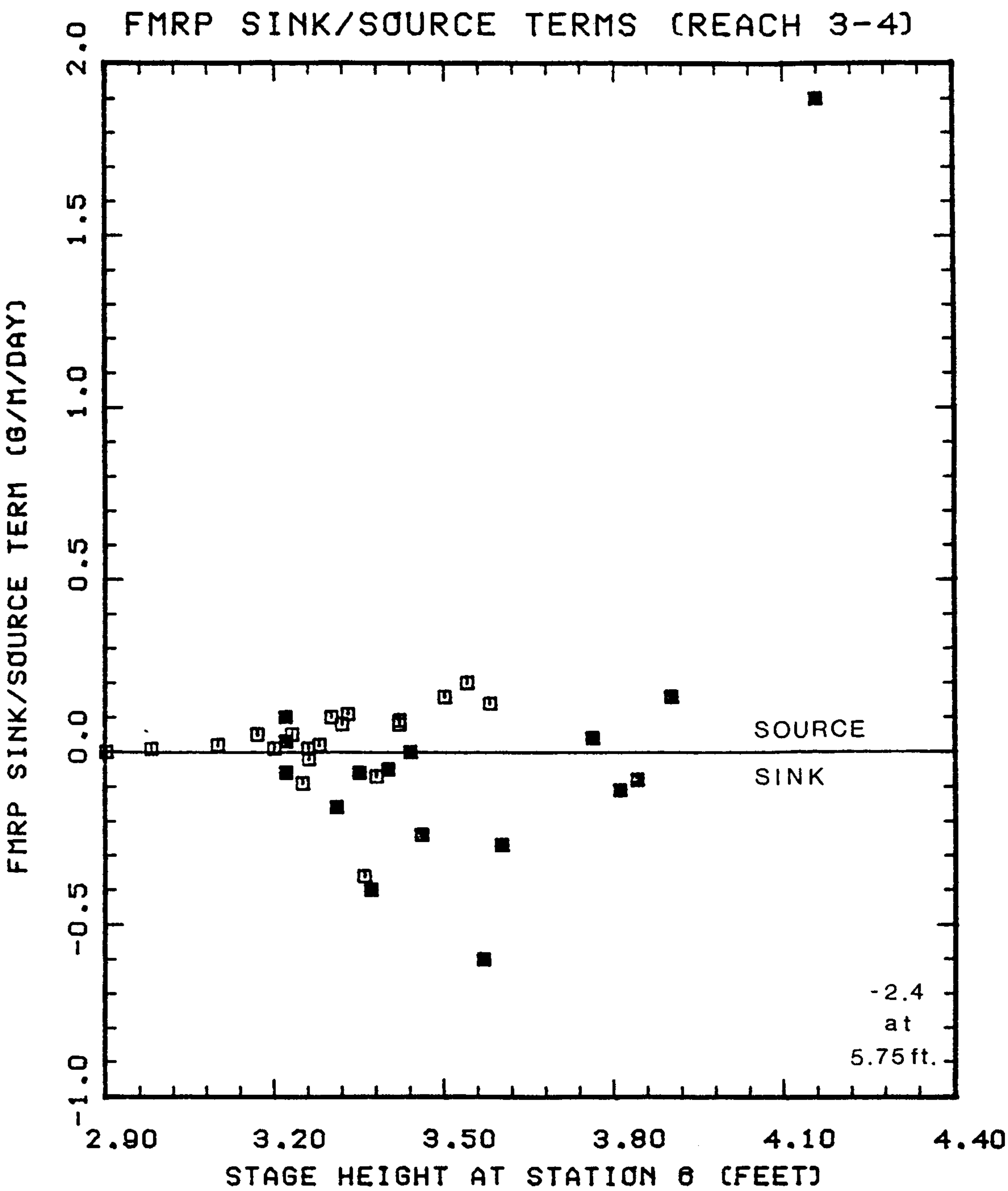


Figure 17: Daily FMRP Sink/Source Terms (Reach 3-4)

■ = With Wastewater Loading

□ = Without Wastewater Loading

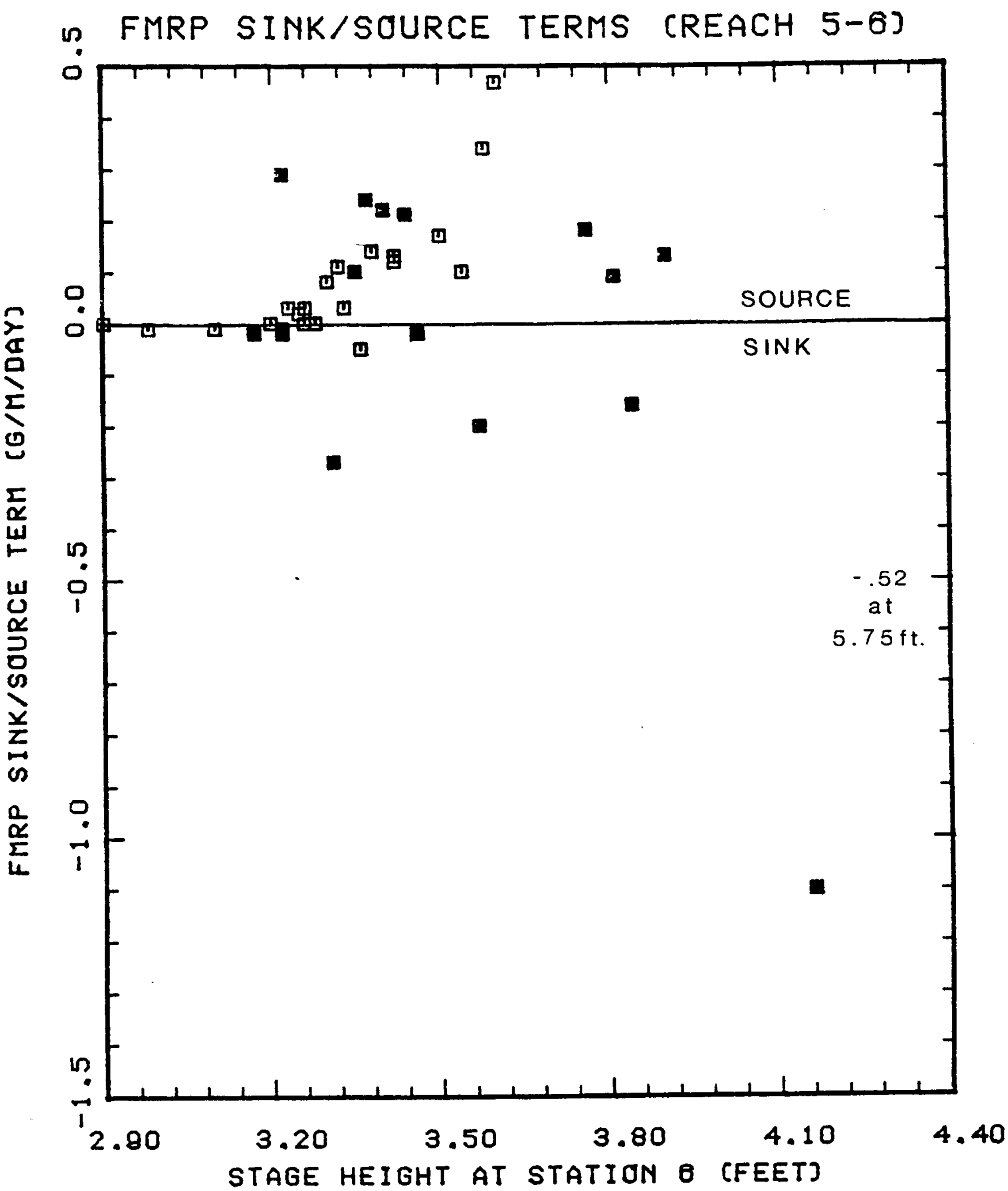


Figure 18: Daily FMRP Sink/Source Terms (Reach 5-6)

■ = With Wastewater Loading

□ = Without Wastewater Loading

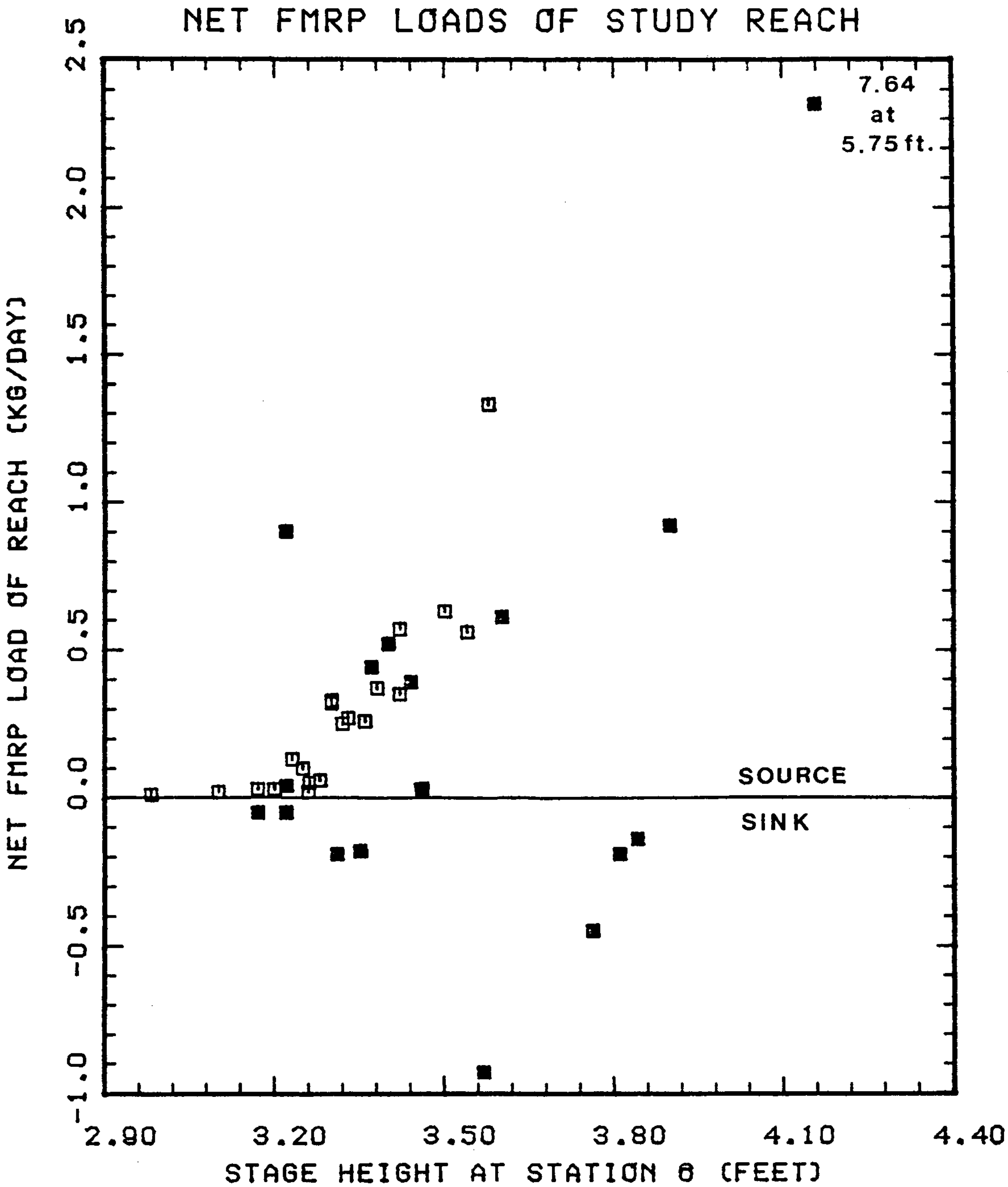


Figure 19: Daily Net FMRP Loads of Study Reach

■ = With Wastewater Loading

□ = Without Wastewater Loading

shown in Table 2, which gives the frequencies with which reaches act as sources and sinks when the average FMRP concentration in the reach is above and below 0.25 mg/1. A statistical test (the Pearson chi-squared test of association with Yates connection) of these data indicated rejection of the

TABLE 2

ASSOCIATION OF SINKS AND SOURCES WITH CONCENTRATIONS OF FMRP GREATER OR LESS THAN 0.25 mg/1, DUDLEY BROOK. Data from Green (1982)

Reach 1-2

	Sink	Source
<0.25 mg/1	1	21
>0.25 mg/1	7	8

Reach 3-4

	Sink	Source
<0.25 mg/1	7	15
>0.25 mg/1	7	7

Reach 5-6

	Sink	Source
<0.25 mg/1	4	15
>0.25 mg/1	10	9

All Reaches

	Sink	Source
<0.25 mg/1	12	51
>0.25 mg/1	24	24

null hypothesis of no association at the 5% level for all cases except Reach 3-4. This test is not fully conclusive because other factors which would be expected to affect the source-sink behavior such as pH, temperature, dissolved-oxygen concentration, flow rate, and sediment type have not been accounted for. The failure to reject the null hypothesis at Reach 3-4 could be due to the slow-flowing nature of the stream in that reach, which causes it to operate as a sink of phosphorus more often than the other reaches.

The overall "model" that emerges from Green's (1982) study reinforces the notion that phosphorus behavior in streams is complex. It suggests that phosphorus may leave the water column to be stored in sediments for some distance below a significant source of phosphorus loadings. However, the time and occurrence of such behavior depends on a number of variable chemical, flow, and sediment conditions. At high flows, sediments may be eroded and phosphorus may be reintroduced into the water column and transported downstream. If loading decreases to relatively low levels, phosphorus may be released into the water column due to the kinetic effects described above.

Clearly, the spatial and temporal variability of the behavior of phosphorus suggests caution in using a steady-state design-flow approach to model it for basin-planning purposes. Equations 20 or 22 are sound bases for modeling the downstream changes in phosphorus concentrations, but can be usefully applied only if the concentrations can be meaningfully associated with the design flow, and if appropriate values of c_b and r_s can be determined. As noted in the last section of the present report, these are areas to which future research should be directed.

The steady-state planning model of Gianessi et al. (1981) included phosphorus along with total dissolved solids, BOD and DO, and nitrogen. They modeled phosphorus by assuming that loads attenuated downstream following an exponential decay. Figure 14 shows that such a model would not be appropriate for Dudley Brook.

VI. SUMMARY AND CONCLUSIONS

Water-resource planning models are designed to allow resource managers to evaluate a large number of alternative plans and to identify the most promising candidates for more detailed analysis. Given the context of water-resource decision making in the United States, planning models are most helpful if they are descriptive simulation models, rather than prescriptive optimization models. Simulation models are best adapted to allow planners to try out many combinations of alternatives and to evaluate them with respect to a realistically large number of objectives, some of which can be expressed only qualitatively.

Such simulation models are necessarily simplified constructs of the behavior of a river basin. To be of use, however, they must satisfy four criteria: 1) they must retain the hydrologically correct spatial relations of the river network in the basin; 2) they must simulate important water-quantity and -quality parameters in hydrologically appropriate relations; 3) they must give a "picture" of the river basin water quantity and quality under conditions that are critical for river-basin planning; and 4) they must contain enough complexity to allow evaluation of a full range of planning alternatives, including both structural and non-structural schemes on both local and regional scales. In order to meet these criteria, the model must incorporate explicit, comprehensive, and hydrologically sound definitions of water demand. Many previous planning models have not satisfied these requirements.

Section III of the present report describes in detail the elements of a model that satisfies the above conditions. It explicitly defines water demand to include withdrawal use, instream use, water-quality maintenance and consumptive use in a hydrologically realistic way. It defines supply as a specific low-flow condition: streamflow available on 95% of the days (Q_{95}). A simple but comprehensive model of water use and treatment at a water-use site (municipality or industrial location) is developed, which shows interactions among quantity and quality under any combination of planning alternatives.

In New Hampshire and New England generally, the most important water-quality constituents for planning purposes are dissolved oxygen, phosphorus, and suspended sediments. In general, dissolved-oxygen concentrations are most critical under low-flow conditions that are also critical for water supply. The behavior of phosphorus is variably related to flow rates, but simulation of low-flow conditions probably gives a good indication of phosphorus problems. Suspended-sediment behavior can not be simulated in a low-flow model and was not included in those studies. Detailed relations for modeling Q_{95}, dissolved oxygen, and phosphorus were derived using basic principles and empirical relations appropriate to New Hampshire.

The implementation of the planning model to simulate a portion of the Merrimack River Basin in New Hampshire was described in Section IV. Dissolved oxygen was the only water-quality constituent included in this simulation. Comparisons of estimated dissolved oxygen concentrations and dissolved oxygen measured under summer low-flow conditions were encouraging, and clearly demonstrate the feasibility of developing models of the type required for planning.

Extensive field studies of phosphorus in a small stream network in southeastern New Hampshire showed that the behavior of that element is complex. Phosphorus was taken up and released by sediments and stream organisms, and a given reach operated as a source under some conditions and a sink under others. There was no consistent relation of phosphorus concentrations to streamflow rate, but there was some indication that a chemical kinetic effect was operating. Further work is required to determine the most appropriate approach for including phosphorus in water-resource planning models.

Overall, this study has established the need for water-resource planning models, has described the specific elements of such models, and has demonstrated the usefulness and feasibility of developing and applying such models in New England. The impetus for this development and application must come from the region's water-resource planning agencies, particularly those at the state level.

The next section of this report identifies research needed to improve water-resource planning models.

VII. SUGGESTIONS FOR FURTHER RESEARCH

Specific suggestions for further research to improve capabilities for river-basin planning models are listed below, related to major topics addressed in Section III.

B. Definition of Demand and Supply

1. How does withdrawal demand vary seasonally for various types of use?
2. How can satisfactory estimates of industrial water use be obtained?
3. How should water demands be forecast?
4. How can yields (supply) of aquifers be estimated for planning purposes?

C. Evaluation of Alternatives

1. How can the effects of water-supply and flow-augmentation reservoirs on Q_{95} be estimated?
2. How can the effects of watershed management practices on Q_{95} be estimated?
3. What are the potential effects of water re-use, conservation and pricing policies on future demands?

E. Streamflow Modeling

1. How does Q_{95} change in the downstream direction in the region?

F. Dissolved-Solids Modeling

1. How best can critical values of phosphorus concentrations be represented in a planning model?
2. How do short-term inputs of phosphorus from urban and agricultural sources affect critical phosphorus concentrations?

3. Under what conditions does a river reach act as a source or sink for phosphorus?

4. What are appropriate values for base-flow contributions of phosphorus to rivers, and how are these related to land use and soils?

5. Under what conditions are other dissolved-solids species, such as nitrate, critical?

G. Dissolved Oxygen Modeling

1. How best can reaeration coefficients be estimated for critical low-flow conditions?

2. How can sampling programs be established that allow for meaningful interpretation of dissolved-oxygen measurements?

3. Under what conditions are factors other than carbonaceous deoxygenation and reaeration important in dissolved-oxygen dynamics?

APPENDIX 1

LIST OF VARIABLES AND SYMBOLS

Symbol	Definition, Dimensions, and Designation as Forecast (f) or Decision (d) Variable
A	Rate of addition of contaminant at use site [M/T] (f)
A_b	Area of watershed contributing streamflow between two nodes $[L^2]$
A_D	Drainage area $[L^2]$
C	Rate of consumptive use $[L^3/T]$
c_A	Concentration just upstream from node $[M/L^3]$
c_B	Concentration just upstream from node $[M/L^3]$
c_b	Concentration in non-point-source contribution to streamflow $[M/L^3]$
c_C	Concentration just downstream from node $[M/L^3]$
c_c	Concentration of dissolved oxygen at sag point $[M/L^3]$
c_D	Concentration in flow arriving at downstream node $[M/L^3]$
c_d	Concentration in waste discharge to stream $[M/L^3]$
c_e	Concentration in off-stream water source $[M/L^3]$ (f)
c_j	Concentration entering waste-water treatment facility $[M/L^3]$
c_T	Concentration of dissolved oxygen at travel time T below waste source $[M/L^3]$
c_t	Concentration entering water-treatment plant $[M/L^3]$
c_u	Concentration in water being used $[M/L^3]$
c^+	Maximum allowable concentration in stream $[M/L^3]$ (f)
c_o	Concentration just below upstream node $[M/L^3]$
c_1	Concentration in stream arriving at use site $[M/L^3]$ (f)

c_3	Concentration in stream leaving use site $[M/L^3]$
D	Rate of waste-water discharge $[L^3/T]$
E	Rate of withdrawal from off-stream source $[L^3/T]$ (d)
F	Rate of removal of contaminant at water-treatment plant [M/T]
G	Rate of removal of contaminant at waste-water-treatment plant [M/T]
J	Rate of flow entering waste-water-treatment plant $[L^3/T]$
K_1	Deoxygenation coefficient [1/T]
K_2	Reaeration coefficient [1/T]
k_c	Fraction of water used consumptively [1] (f)
k_d	Fraction of water used that is discharged to stream [1]
k_r	Fraction of water used that is recycled [1] (d)
L_b	Areal rate of addition of contaminant in non-point contributions to streamflow $[M/L^2T]$
L_X	Concentration of BOD just above downstream node $[M/L^3]$
L_o	Concentration of BOD just below upstream node $[M/L^3]$
P	Price per unit of water $[\$/L^3]$ (d)
Q_A	Streamflow rate just above node $[L^3/T]$
Q_B	Streamflow rate just above node $[L^3/T]$
Q_C	Streamflow rate just below node $[L^3/T]$
Q_D	Streamflow rate just above downstream node $[L^3/T]$
Q_{min}	Minimum flow rate in stream for instream uses $[L^3/T]$ (d)
Q_q	Streamflow rate required to maintain contaminant concentration at acceptable level $[L^3/T]$
Q_R	Total water requirement from stream $[L^3/T]$
Q_o	Streamflow rate just below upstream node $[L^3/T]$
Q_1	Streamflow rate arriving at use site $[L^3/T]$ (f)

Q_2	Streamflow rate between point of withdrawal and point of waste-water discharge $[L^3/T]$
Q_3	Streamflow rate leaving use site $[L^3/T]$
Q_{95}	Streamflow rate exceeded on 95% of the days $[L^3/T]$
R	Rate of recycling $[L^3/T]$
r_s	Rate of addition of contaminant due to physical, chemical, and biological processes in stream channel (source-sink) [M/LT]
S	Stream sinuosity [1]
T	Travel time [T]
T_c	Travel time to sag point [T]
t_f	Fractional removal of contaminant at water-treatment plant [1] (d)
t_g	Fractional removal of contaminant at waste-water treatment plant [1] (d)
U	Rate of use of water $[L^3/T]$ (f)
v_{95}	Average stream velocity at Q_{95} [L/T]
W	Rate of withdrawal from stream $[L^3/T]$
X	Distance between adjacent nodes [L]
x	Distance downstream of node [L]
X_t	Thalweg distance downstream of node [L]
δ	Empirical water-price coefficient (f)
π	Demand elasticity for water [1] (f)

APPENDIX 2

RELATIONS BETWEEN STREAM LENGTH AND DRAINAGE AREA

Introduction

This Appendix describes a preliminary analysis of the relations between: 1) inter-node stream distance and inter-node drainage area; and 2) first-order stream length and drainage area as a basis for estimating downstream changes in concentration of dissolved solids (Equations 20-23). This preliminary work is an analysis of streams on two arbitrarily selected 1:24,000-scale U.S. Geological Survey quadrangles in northern New Hampshire. A supplementary examination of relations between inter-node stream distance and drainage area was also done using data developed by the U.S. Army Corps of Engineers for the upper Connecticut River.

The "stream distance" measured in these analyses is actually the down-valley distance, because the actual stream (thalweg) distance is not hydrologically relevant. The relation between the two distances is:

$$x = x_t/S \qquad (2\text{-}1)$$

where x is down-valley distance (often called "stream distance" herein), x_t is thalweg distance, and S is stream sinuosity.

1:24,000-Scale Analysis

The Franconia, N. H., and East Haverhill, N. H., 7-1/2 minute quadrangles were arbitrarily selected for analysis. All complete stream networks shown on these maps were traced, and drainage areas at tributary junctions were delineated. A total of 32 first-order streams (all streams above their first tributary junction) and 28 inter-nodal links were present on the two maps. Tables 2-1 and 2-2 summarize the data.

Figure 2-1 is a plot of A_D vs. x for first-order streams. The

Table 2-1

Stream Lengths and Drainage Areas for First-Order Tributaries

Stream	Area (mi^2)	Length (mi)
East Haverhill Quad.		
N. Br. Oliverian Bk.		
1A	0.12	0.14
1A1	0.43	0.42
1A2	0.07	0.40
1A3	0.85	0.89
1A4	0.51	0.56
1A5	0.42	0.56
1A6	1.19	1.04
1A7	0.03	0.26
1A8	0.60	0.82
1A9	0.45	0.28
1A10	0.29	0.64
1A11	0.35	0.52
Titus Bk.		
1AB	1.89	1.67
1A1	0.34	0.85
Wilmot Bk.		
1A	0.86	0.75
1A1	0.03	0.28
Franconia Quad.		
Lafayette Bk.		
1A	0.78	0.75
1ABC	1.70	2.45
Skookumchuck Bk.		
1A	0.82	1.22
1A1	0.38	0.75
Jordan Bk.	0.82	1.55
Beaver Bk.		
1A	0.80	0.47
1A1	0.85	0.52
Meadow Bk.	2.76	2.96
Pemigewasset R.		
1AB	2.80	2.55
Walker Bk.		
1A	0.69	1.08
1A1	0.34	0.71
Dry Bk.		
1A	0.43	0.80
1A1	0.20	0.42
Scarface Bk.	1.90	2.22
S. Br. Gale R.		
1ABC	2.21	2.31
1A1	0.13	0.61

TABLE 2-2

Stream Lengths and Drainage Areas for Inter-node Links

Stream	Area (mi^2)	Length (mi)
East Haverhill Quad.		
N. Br. Oliverian Bk.		
2A	0.50	0.56
3A	0.16	0.40
3B	0.22	0.45
3C	0.19	0.52
3D	0.03	0.23
3E	0.02	0.23
3F	0.37	0.92
3G	0.03	0.19
3H	0.05	0.33
2A1	0.05	0.28
2B	0.34	0.45
2C	0.12	0.42
2A2	0.48	0.75
Titus Bk.		
2A	0.29	0.56
Wilmot Bk.		
2AB	0.69	1.55
Franconia Quad.		
Lafayette Bk.		
2A	0.41	0.47
3A	0.05	0.09
3BC	1.36	2.07
Skookumchuck Bk.		
2A	0.55	1.04
Beaver Bk.		
2A	0.67	0.94
3A	0.13	0.75
3B	0.10	0.33
Pemigewasset R.		
2A	0.63	0.56
3A	0.64	0.33
Walker Bk.		
2A	0.22	1.27
Dry Bk.		
2A	0.26	0.61
S. Br. Gale R.		
2C	0.40	0.89
2D	0.63	0.75

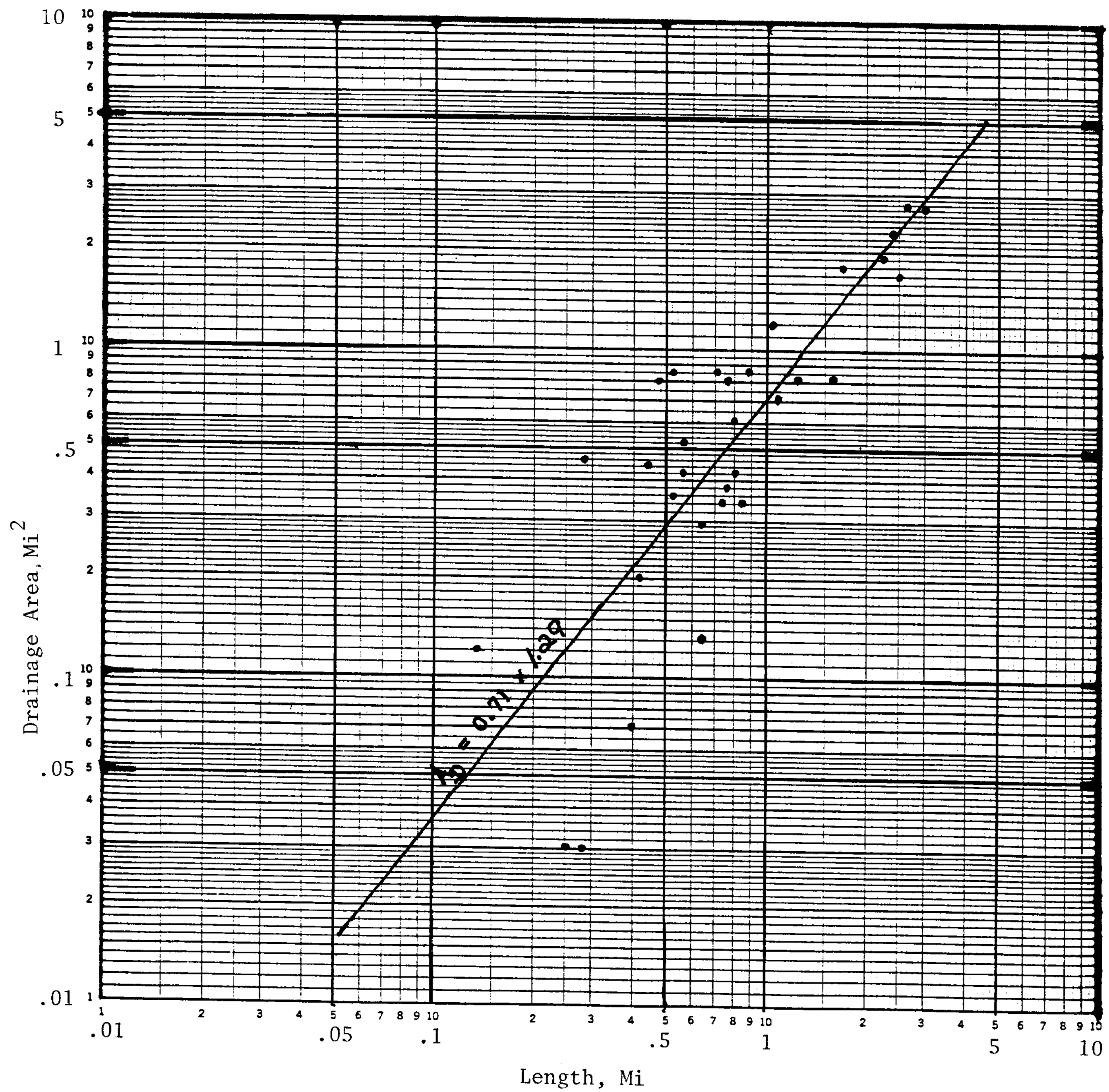


Figure 2-1: Drainage area vs. length for first-order streams, Franconia, N.H. and East Haverhill, N.H., quadrangles.

regression relation is:

$$A_D = 0.71x^{1.29} \tag{2-2}$$

with A_D in mi^2, and x in mi; $n = 32$, $r^2 = 0.67$, and standard error of estimate = 0.2913 $\log_{10}$ units.

Figure 2-2 is a plot of A_D vs. x for inter-node links. The regression relation is:

$$A_D = 0.50x^{1.30}$$

with A_D in mi^2 and x in mi; $n = 28$, $r^2 = 0.61$, and standard error of estimate = 0.3119 $\log_{10}$ units.

Connecticut River Analysis

As part of their flood study of the Connecticut River, the U.S. Army Corps of Engineers measured "local areas" contributing flow directly to the mainstem of the river. These are identical to inter-node links. However, in their analysis, only major tributary streams were considered, so their local areas contain many stream segments that would have been separately identified in a study done at a larger map scale. The data for these local areas are given in Table 2-3. The statistical analysis gives the following regression equation:

$$A_D = 3.32x^{1.28} \tag{2-4}$$

with A_D in mi^2 and x in mi; $n = 15$, $r^2 = 0.82$, standard error of estimate = 0.1703 $\log_{10}$ units.

Conclusion

The above analyses, although preliminary, indicate that drainage area increases approximately as the 1.3 power of stream length for both first-order tributaries and inter-node links. At a given scale, the

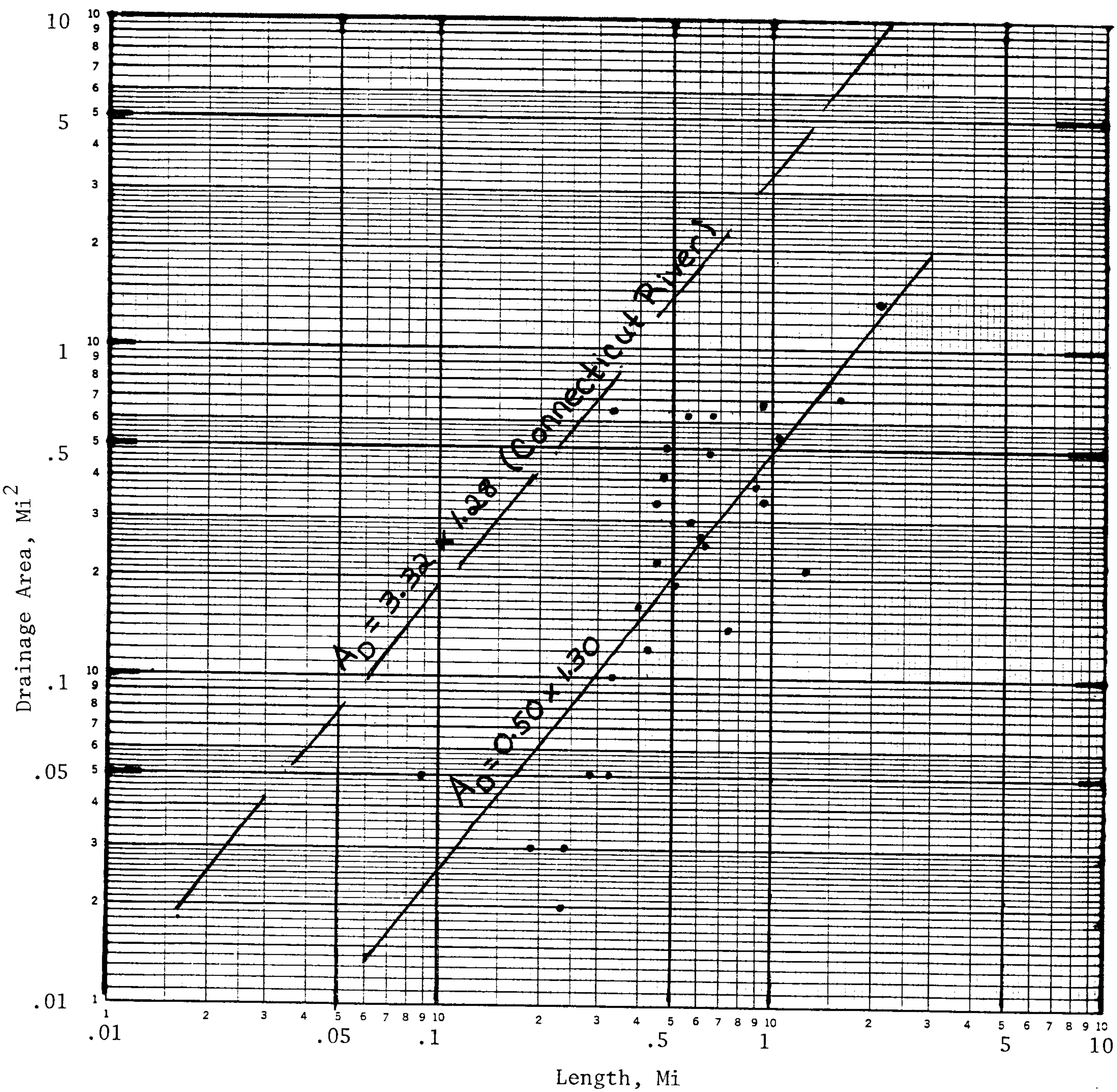


Figure 2-2: Drainage area vs. length for inter-node links, Franconia, N.H., East Haverhill, N.H., quadrangles, (solid line and dots) and Mainstem Connecticut River (dashed line).

TABLE 2-3

Stream Lengths and Drainage Areas for Inter-nodel Links,

Main Stem Connecticut River

Link ("Local Areas")	A_D (mi^2)	x (mi)
A(4, 5, 6)	132.1	19.3
B(7)	57.9	12.8
C(8, 9, 10)	136.0	12.0
D(11, 12)	129.0	21.1
E(14, 15, 16)	124.0	13.1
F(19, 20)	81.0	12.5
F(21)	39.0	6.8
H(24, 25, 26)	138.0	22.8
I(28)	62.0	8.8
J(31)	39.5	4.0
K(33)	167.0	14.8
L(35)	72.5	12.0
M(37)	24.0	6.8
N(39)	7.0	2.8
O(42, 43)	179.0	22.8

coefficient in the relationship is somewhat larger for first-order streams than for inter-node links because a certain size drainage area is required to support the formation of a stream. The coefficient in the relationship is determined by map scale (*i.e.*, the size of the smallest streams which are considered), and increases with scale.

Assuming direct proportionality between Q_{95} and A_D (see discussion of Equation 17), the average relationship between Q_{95} and drainage area for unregulated streams in New Hampshire is:

$$Q_{95} = 0.17A_D \quad , \tag{2-5}$$

where Q_{95} is in ft^3/S and A_D is in mi^2. (This relationship ignores known geographical variability within the state, as discussed by Dingman (1978)).

Substituting Equation 2-5 into Equation 2-3 gives:

$$Q_{95} = 0.085x^{1.3} \tag{2-6}$$

as a statewide estimate of the rate of increase of Q_{95} with distance downstream of a tributary node. Equation 2-6 is the basis for Equation 17.

APPENDIX 3

BASIC EQUATIONS FOR DOWNSTREAM CHANGES IN DISSOLVED-SOLIDS CONCENTRATIONS

Figure 3-1 illustrates the terms defining the one-dimensional steady-state mass balance of a dissolved-solids constituent in an Eulerian stream reach of length Δx, ignoring longitudinal diffusion. Q $(= Q(x))$ is the discharge at the upstream end $[L^3/T]$, c $(= c(x))$ is the concentration of the constituent at the upstream end $[M/L^3]$, c_b is the concentration in the water entering or leaving the reach $[M/L^3]$, and r_s is the rate at which the constituent is added by physical, chemical, or biological processes within the reach $[M/LT]$. Formulating the mass balance:

$$cQ + c_b \frac{dQ}{dx} \Delta x + r_s \, \Delta x - \left(c + \frac{dc}{dx} \Delta x\right) \left(Q + \frac{dQ}{dx} \Delta x\right) = 0 \, . \qquad (3\text{-}1)$$

Ignoring second-order terms, Equation 3-1 reduces to

$$\frac{dc}{dx} + \frac{1}{Q} \frac{dQ}{dx} c = \frac{c_b}{Q} \frac{dQ}{dx} + \frac{r_s}{Q} \, . \qquad (3\text{-}2)$$

Equation 3-2 is a first-order linear differential equation that has the solution:

$$c = \exp\left(-\int \frac{1}{Q} \frac{dQ}{dx} dx\right) \int \exp\left(\int \frac{1}{Q} \frac{dQ}{dx} dx\right) \left[\frac{c_b}{Q} \frac{dQ}{dx} + \frac{r_s}{Q}\right] dx$$
$$+ \, C \exp\left\langle - \int \frac{1}{Q} \frac{dQ}{dx} dx \right\rangle , \qquad (3\text{-}3)$$

and, since:

$$\frac{1}{Q} \frac{dQ}{dx} dx = \frac{dQ}{Q} \quad ,$$

we have:

$$c = \exp\left\langle - \int \frac{dQ}{Q} \right\rangle \int \exp\left\langle \int \frac{dQ}{Q} \right\rangle \left[\frac{c_b}{Q} \frac{dQ}{dx} + \frac{r_s}{Q}\right] dx + C \exp\left\langle - \int \frac{dQ}{Q} \right\rangle$$
$$(3\text{-}4)$$

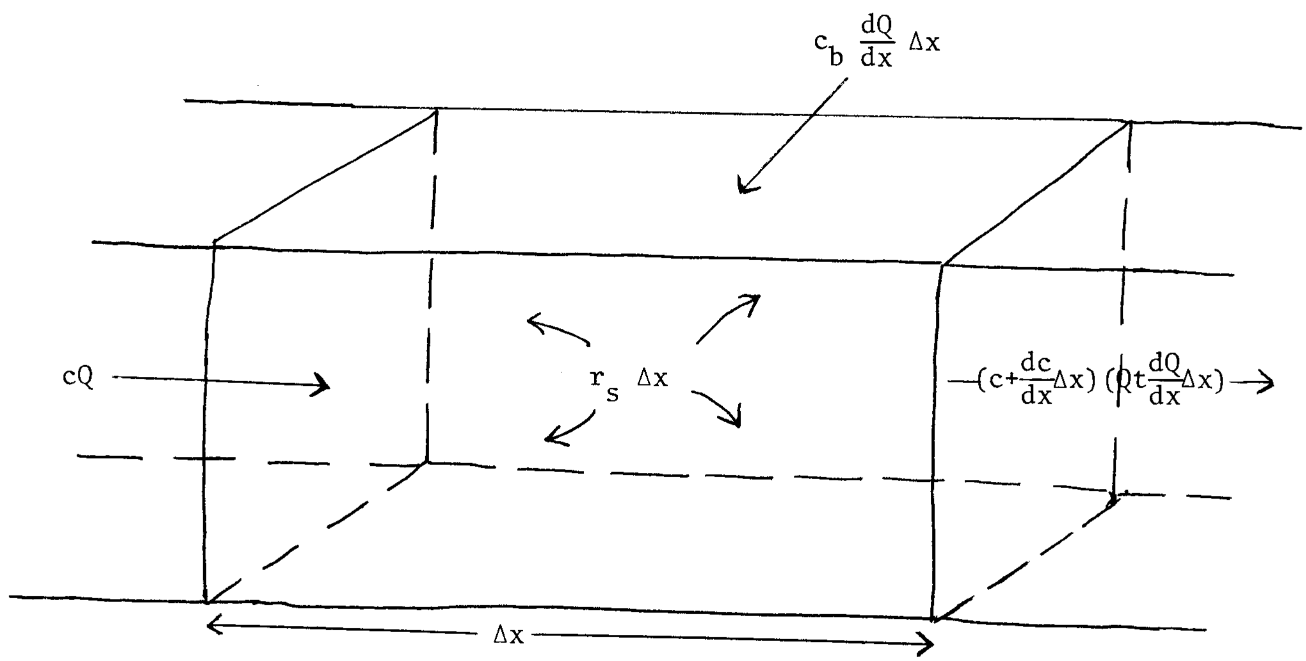


Figure 3-1: Definitions of terms for derivation of downstream dissolved-solids relations.

Since:

$$\int \frac{dQ}{Q} = \ell n\ Q \quad ,$$

Equation 3-4 reduces further to:

$$c = \exp\ (-\ell n\ Q) \int \exp\ (\ell n\ Q) \left[\frac{c_b}{Q}\frac{dQ}{dx} + \frac{r_s}{Q}\right] dx + C \exp\ (-\ell n\ Q) \quad ;$$

$$c = \frac{1}{Q} \int Q \left[\frac{c_b}{Q}\frac{dQ}{dx} + \frac{r_s}{Q}\right] dx + \frac{C}{Q} \quad ;$$

$$c = c_b + \frac{r_s x}{Q} + \frac{C}{Q} \quad . \qquad (3\text{-}5)$$

To evaluate C, note that $c = c_o$ when $x = 0$, so that:

$$C = Q_o\ (c_o - c_b) \qquad (3\text{-}6)$$

and Equation A5 becomes:

$$c = c_b + \frac{r_s x + Q_o\ (c_o - c_b)}{Q} \quad , \qquad (3\text{-}7)$$

where Q_o is the discharge at the upstream end of the reach.

Designating the concentration and discharge at the downstream end of the reach by C_D and Q_D, respectively, Equation 3-7 becomes:

$$c_D = c_b + \frac{r_s x + Q_o\ (c_o - c_b)}{Q_D} \quad . \qquad (3\text{-}8a)$$

This equation applies when $Q_D \geqslant Q_o$.

When $Q_D \leqslant Q_o$, there is leakage from the stream, and the concentration of the leaking water is c_o. Thus for this case, Equation 3-8a becomes

$$c_D = c_o + \frac{r_s x}{Q} \quad . \qquad (3\text{-}8b)$$

A final modification can be made to Equations 3-8a and 3-8b to account for the fact that r_s is likely to be expressed as a rate per mile of stream channel, whereas if one computes Q_d as a function of x, it is hydrologically more meaningful to let x represent the down-valley distance. To account for this, the sinuosity, S, defined as the ratio of channel distance to down valley distance, can be introduced into the above equations as follows:

$$c_D = c_b + \frac{Sr_s x + Q_o (c_o - c_b)}{Q_D} \quad , \; Q_o < Q_D \quad ; \tag{3-9a}$$

and:

$$c_D = c_o + \frac{Sr_s x}{Q_D} \quad , \qquad Q_o > Q_D \qquad ; \tag{3-9b}$$

where Q_D can be expressed as a function of x, e.g.

$$Q_D = Q_o + ax^b \quad . \tag{3-10}$$

Appendix 2 of this report explores the appropriate values of the constants a and b in Equation 3-10.

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