RR33

INTEGRATED ASSESSMENT AND MANAGEMENT OF WATER QUANTITY AND WATER QUALITY IN THE MERRIMACK RIVER BASIN, NEW HAMPSHIRE

By
S. Lawrence Dingman
and
Bruce E. Lindsay

INTEGRATED ASSESSMENT AND MANAGEMENT OF WATER QUANTITY AND WATER QUALITY IN THE MERRIMACK RIVER BASIN, NEW HAMPSHIRE

By

S. Lawrence Dingman

and

Bruce E. Lindsay

Institute of Natural and Environmental Resources

TECHNICAL COMPLETION REPORT

Project Number: A-050-NH
Annual Allotment Agreement No.: 14-34-0001-8031

The work upon which this publication is based was supported in part by funds provided by the Office of Water Research and Technology, Project No. A-050-NH, U.S. Department of the Interior, Washington, D.C., as authorized by the Water Research and Development Act of 1978.

Water Resource Research Center
University of New Hampshire
Durham, New Hampshire

February, 1981

ACKNOWLEDGEMENTS

We thank George Stevens and Ray Wenniger of the U.S. Soil Conservation Service office in Durham, New Hampshire, for making available data on reservoir sites and construction costs that were essential to our study. As former Hydrology graduate students, Wenniger and George Ryder also made valuable compilations of data and bibliographies that were important contributions to the present study, and Paul Sutton wrote Appendix B. Hydrology students Alicia Healey and Karen Howe compiled the maps of reservoir sites, groundwater sources, and political boundaries.

Dr. Gordon Smith and Ph.D. candidate Daniel Dunn of the Whittemore School of Business and Economics provided much time and effort adapting a computer program to the developed mixed integer economic model. Their input facilitated the obtainment of the empirical results.

We are grateful to Gordon Byers, Chairman of the Water Resource Research Center at the University of New Hampshire, for support and encouragement throughout this project. Support for this work was provided by a Department of Interior, Office of Water Research and Technology Grant, A-050-NH, administered by the Water Resource Research Center.

ABSTRACT

Sound management of water-resource problems has four major requisites: 1) precise definition of the problem; 2) consideration of all potential alternative solutions; 3) appropriate objectives and criteria for choosing among alternatives; and 4) ability to analyze the alternatives with respect to the choice criteria.

This report contributes to these objectives in ways that will be of practical use in water-resource planning in the New Hampshire portion of the Merrimack River Basin and in New England. It does this by separately considering the hydrologic and economic aspects. These two components can be combined into a single framework.

The hydrologic analysis examines the nature of supply and demand in the context of water resources. Additionally, a quantitative planning-level framework for identifying the existence and nature of water-resource problems is developed. This framework allows evaluation of the degree to which any proposed management strategy will contribute to the solution of such problems. This model constitutes a simulation model that can accommodate any combination of alternatives, including those that affect demand as well as those that increase supply.

The economic aspect of this study emphasizes a mixed-integer multiperiod programming model that utilizes hydrologic and economic data for identifying the discounted least cost of water supply, distribution, and scheduling for three communities. Preliminary sample data were used. This model can identify present water-supply sources that are economically feasible for the future, as well as new reservoirs, based upon projected water demands.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

Page

INTRODUCTION

Figure 1 shows the Merrimack River Basin, which includes 3810 mi^2 of New Hampshire's total area of 9304 mi^2. All or parts of 129 towns and cities and 57.8% of the state's population are within this area. Plate I (see Appendix I) shows the communities within the basin. This population has grown at an annual compound rate of 1.87% (see Figure 2) over the period 1950-1975, and manufacturing employment at an annual compound rate of 1.34%. As a result, severe stresses have been placed on the quantity and quality of the region's water resources. The following extract from the Summary Report on Severe Resource Problems and Recommendations for Their Solution, prepared by the New England River Basins Commission (NERBC, 1977) succinctly characterizes the basin's water problems:

> An overriding problem facing the Merrimack River is the conflicting demands being placed upon it to provide municipal water supplies for the urbanized areas of both eastern Massachusetts and southeastern New Hampshire. Potential competition for supplies may occur between water users if the present investigation into the feasibility of diverting Merrimack waters to Boston Metropolitan District Commission and/or coastal New Hampshire is implemented. The situation is aggravated by poor quality water and by increasing industrial demands.
>
> Ground water supplies are also insufficient to meet 1990 maximum daily demands in many of the surburban towns. Increased development causes contamination of ground water supplies from septic system leachates. Naturally high levels of iron and manganese also limit the supply of ground water in the Merrimack Basin. Without careful planning and conservation of both surface and ground water supplies, serious shortages will develop.
>
> Degradation of water quality is a problem in the Merrimack and most of its tributaries. The Merrimack River and estuary are being polluted by raw municipal sewage, combined sewer overflows, and industrial discharges including paper products wastes, textile wastes, and silver plating chemicals. Because the entire river is currently of U (nuisance) water

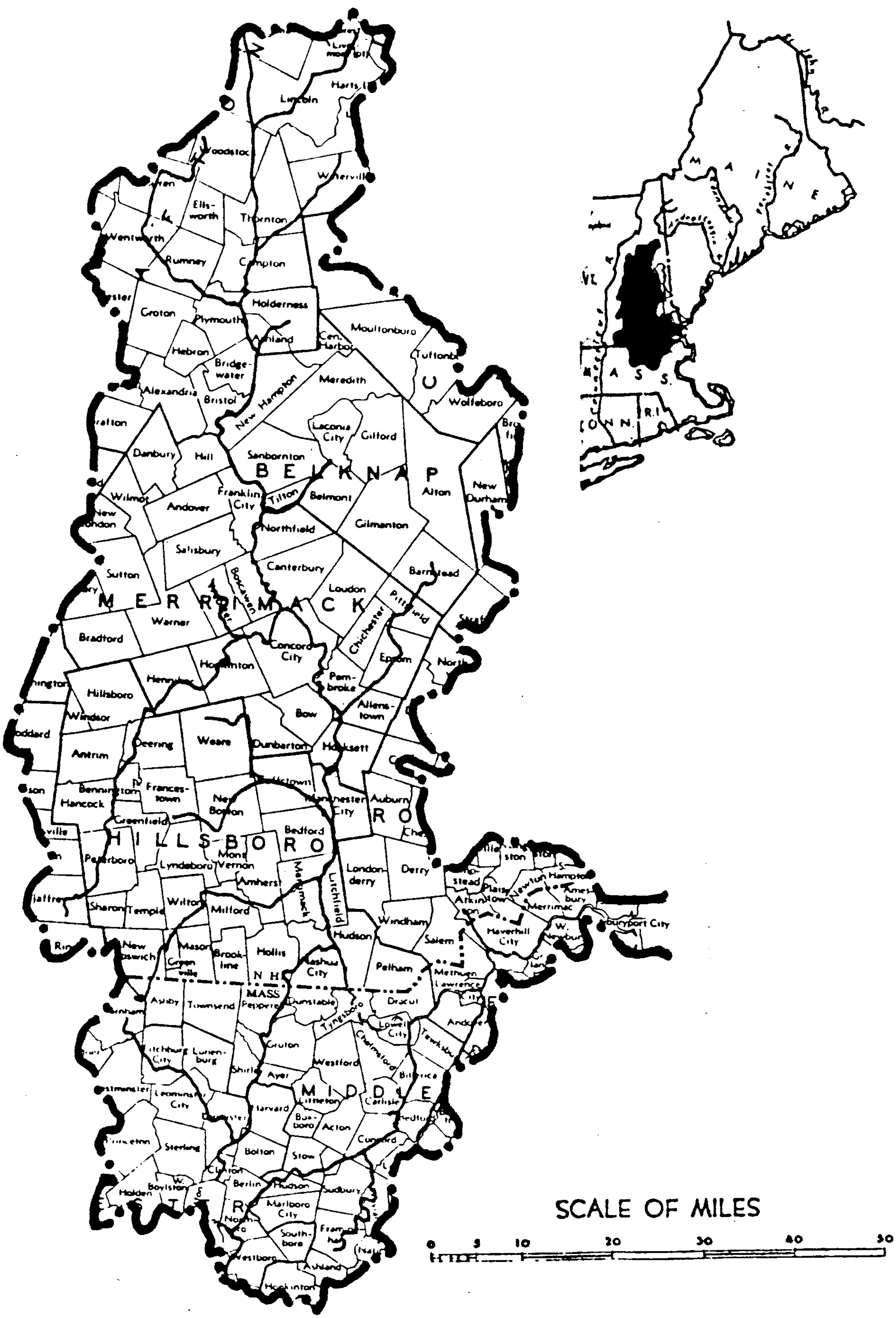

Figure 1. Location of Merrimack River Basin.

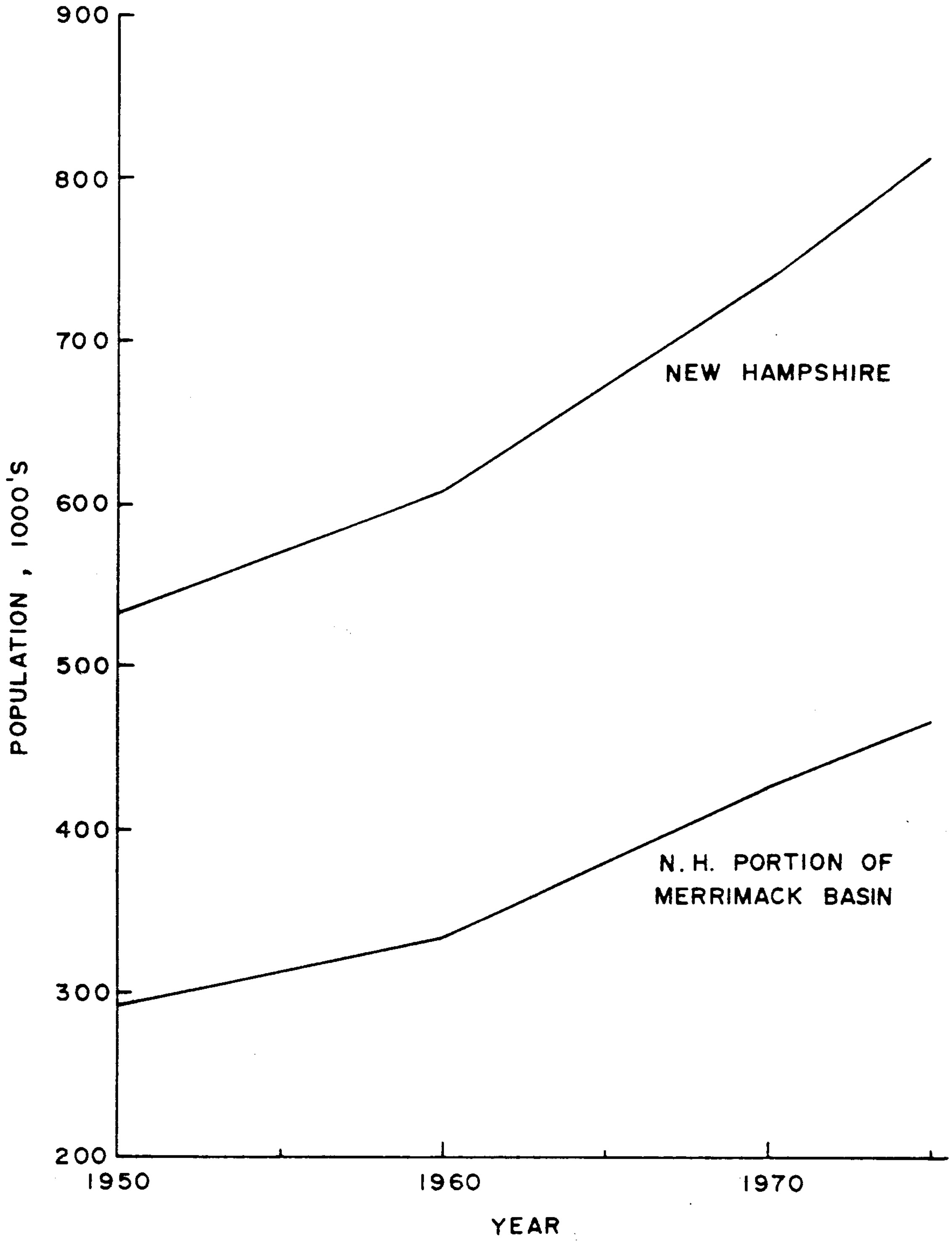

Figure 2. Population trends in New Hampshire and the New Hampshire portion of the Merrimack River Basin.

> quality, use of the water for drinking requires expensive treatment and filtration, and swimming, boating and fishing are limited.... Portions of the Merrimack will not meet Massachusetts' and New Hampshire's Class B standards by 1983.

Fifty-two of the 129 municipalities in the basin are served in part by public water supplies (NERBC, 1978, Map 13), of which 23 have surface-water sources, 13 have ground-water sources, and 16 have both surface and ground water. Fifty-one of the 129 communities are projected to have water-supply deficiencies by 2020 (New England Division, Corps of Engineers, 1977, Plate 9). In addition, the Merrimack Basin is adjacent to two major areas with projected serious water shortages - New Hampshire's coastal region and the Boston Metropolitan District - and has been suggested as a source of water for out-of-basin transfer to both those areas (NERBC, 1978).

Although the facts and studies cited above leave little doubt that the New Hampshire portion of the Merrimack Basin faces severe water-quantity and -quality problems, they tell very little about the true nature of these problems and of the possible solutions to them. The fact that the existing "safe yield" of the water supply of a given community is less than the demand projected for a future date may simply mean that further investment is required to build a reservoir, drill a well, or expand treatment or distribution facilities. The solution to such a problem is largely a matter of engineering, economics, and finance. While such problems are far from trivial, there is also the question of the extent to which the projected increasing demands approach the supply (and quality) limits set by the hydrologic cycle in the region. Most of the conventional assessments of water-resource problems provide very little information on this more fundamental question.

Traditionally, water-resource planning is carried out in a non-integrated, static fashion that may seriously distort and/or limit the rational assessment of problems and solutions. This lack of integration is commonly manifested in the following ways: (1) quantity and quality problems are considered separately; (2) portions of drainage basins--in many instances, individual towns--are considered separately;

(3) ground-water and surface-water sources are considered without regard for the connections between them; (4) the available water resource is commonly assumed to equal the average runoff rate; (5) the range of alternative solutions considered is often unnecessarily constrained and the evaluation of these alternatives is distorted with respect to the explicit tradeoffs associated with each option; (6) the planning process is typically a "one-shot" operation, without provision for readily up-dating demand projections and re-evaluating alternatives as new information becomes available; and (7) current water problems are generally analyzed statically, overlooking any recursive components in which previous time period's supply and quantity parameters affect future periods.

One noteworthy attempt to look at water-resources problems in a more comprehensive, hydrologically-based way was the study of Wollman and Bonem (1971). In this, the United States was divided into 22 water-resource regions, whose boundaries largely coincided with hydrologic divides. All "supply", "demand", and quality considerations within each region were aggregated, and "demands" were projected to 2020. "Supply" was defined as the aggregate river flow rate that is exceeded a specified percentage of the time (90, 95, and 98% were used), and (in the absence of water importation) is determined by the hydrology of the region and the amount of reservoir storage provided. "Demands" were defined as the sum of the required flow for water-quality maintenance plus the consumptive losses plus the discharge of fresh water into the ocean. The integration of ground water with surface water was implicit in their assumption that ground-water flows into rivers and will eventually become river flow. The quantity-quality interactions were specifically considered by modeling the effects of various degrees of waste-water treatment and river flow rates on instream concentrations of dissolved oxygen, nitrates, and phosphates.

Figure 3 summarizes Wollman's and Bonem's analysis for the New England region. This diagram is read as two diagrams combined. The first is a curve of flow available 95% of the time (left-hand vertical axis) vs. storage (bottom horizontal axis), with the existing storage and existing

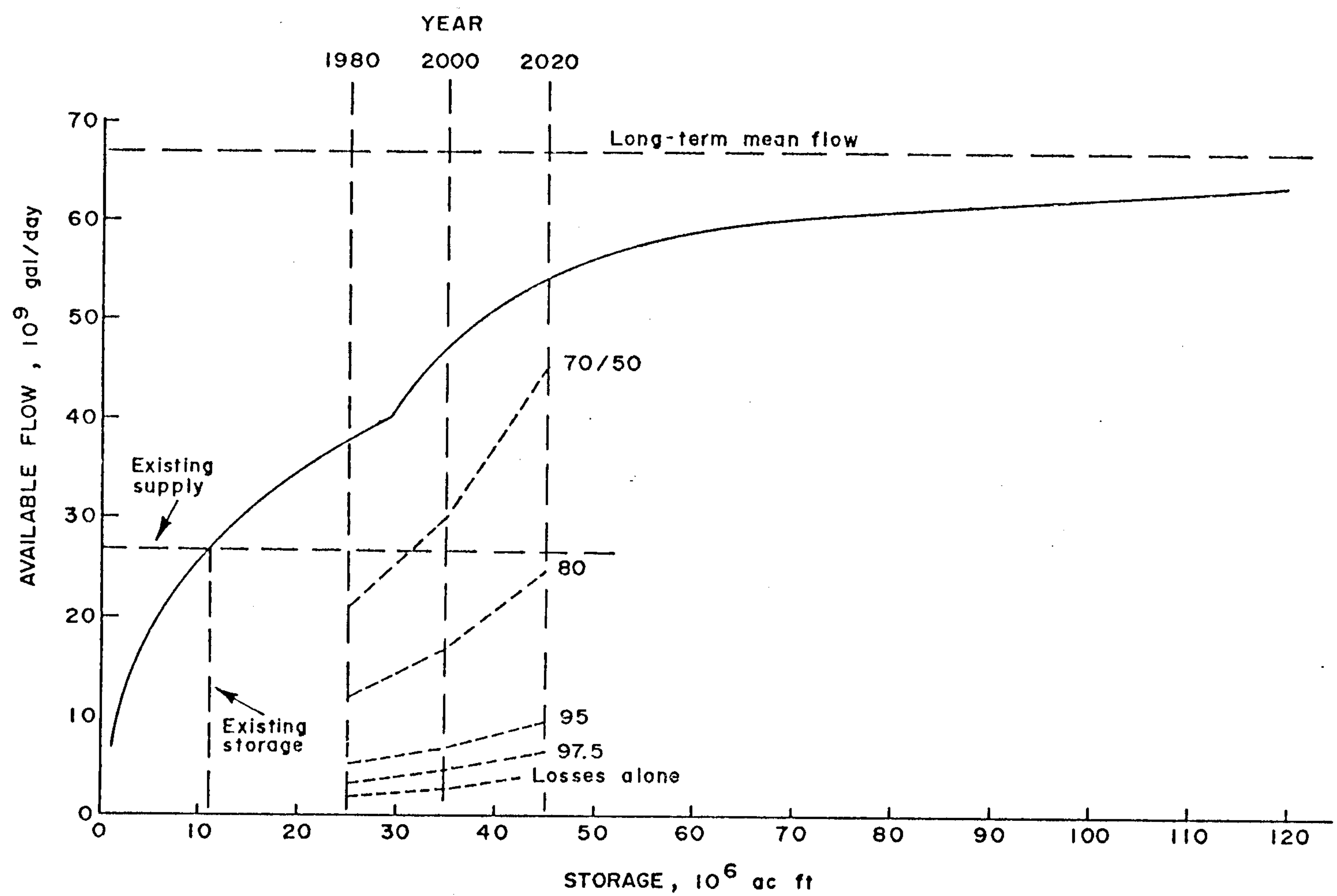

Figure 3. Forecast water demands (consumptive use losses plus water-quality maintenance compared with existing supplies and storage-flow relations for the New England Water-Resource Region. Numbers on dashed lines represent average degrees of waste-water treatment. Data from Wollman and Bonem (1971).

available flow indicated. Superimposed on this is a graph showing water requirements (left-hand vertical axis) vs. year of projection (top horizontal axis), with separate projections depending on the degree of treatment ("50/70" indicates 50% BOD removal for industrial wastes and 70% for municipal wastes; the other figures assume the same percentage reduction for both types of wastes). Together, the two graphs indicate that the existing flows in New England are sufficient until 2020 if treatment is 80% or more, but that a "deficit" (i.e., a reduction in water-quality below 4 mg/l dissolved oxygen) will occur between 1990 and 2000 at the 70/50 treatment level. This "deficit" can be averted by building higher levels of treatment, or more storage, or a combination of both strategies. If only consumptive uses are considered, the region appears amply endowed for a long time.

Figure 4 shows the results of applying Wollman's and Bonem's (1971) method to the Merrimack River Basin (Ryder, 1977). The picture for the Merrimack is worse than for the region as a whole, and indicates that even with a treatment level of 80%, a deficit will occur between 1990 and 2000. At the 50/70 treatment level, it appears that there is not sufficient flow in the basin to satisfy the assumed water-quality standards. However, if only consumptive uses are considered, or if a high degree of treatment is used, the Merrimack, like New England as a whole, has abundant water for a long time.

This last conclusion appears to contradict the statements of the various planning studies cited earlier, which indicated widespread water-supply problems as well as water-quality problems. This apparent contradiction reflects some of the difficulties one faces in characterizing water-resource problems. In general, the results of analyses of these problems depend on the precise definition of "supply" and "demand", the degree of aggregation, and the "demand" projections used.

It is our contention that sound management of water-resource problems has four major requisites, which ideally should be satisfied at the political level which has the responsibility for decision-making and implementation: (1) precise definition of the problem; (2) consideration of all potential alternative solutions; (3) appropriate

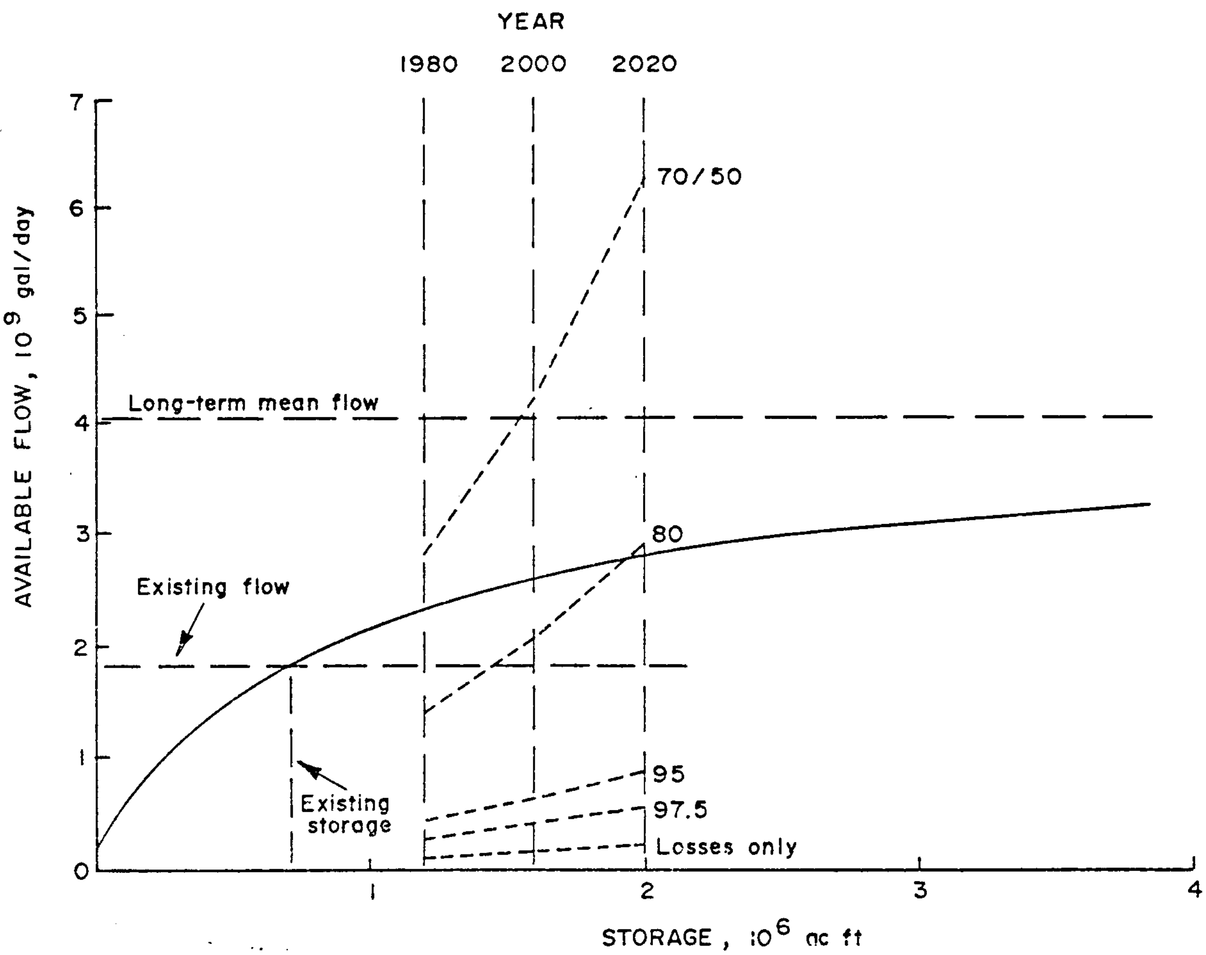

Figure 4. Forecast water demands (consumptive-use losses plus water-quality maintenance) compared with existing supplies and storage-flow relations for the New Hampshire portion of the Merrimack River Basins. Numbers on dashed lines represent average degrees of waste-water treatment).

objectives and criteria for choosing among alternatives, and (4) ability to analyze the alternatives with respect to the choice criteria.

This report attempts to contribute to these objectives in ways that will be of practical use in water-resource planning in the New Hampshire portion of the Merrimack River Basin and in New England and the humid Northeast generally. It does this by separately considering the hydrologic and economic aspects. The potential for merging of these aspects into a single framework, or model, for water-resource planning is clearly indicated, but for the most part could not be accomplished herein because of the limited resources available for this study.

The hydrologic portion of the report first examines the nature of supply and demand in the context of water resources. Water quality and other in-stream flow requirements are integrated in the definitions. Following this, a quantitative planning-level framework for identifying the existence and nature of water-resource problems is developed. Most important, this framework allows evaluation of the degree to which any proposed management strategy will contribute to the solution of such problems. This framework constitutes a simulation model that can accommodate any combination of alternatives, including those that affect demand as well as those that increase supply.

The economic portion of this study emphasizes a mixed-integer multiperiod programming model that utilizes hydrologic and economic data for identifying the discounted least-cost scheduling of water reservoirs for three communities. Preliminary sample data were used. This model can identify present water-supply sources that are economically feasible for the future, as well as new reservoirs, based upon projected water demands.

PROJECT OBJECTIVES

The overall objective of this project is to contribute to the fulfillment of the requirements enumerated above. The specific objectives are:

1) Establish basic hydrologic relations for water-availability computations with respect to surface storage, joint management of ground water and surface water, and watershed management; and
2) Develop economic relationships with respect to costs, supply, and demand associated with various alternatives and to incorporate both economic and hydrologic relations into a dynamic mixed integer programming model.

As will be documented in detail below, these objectives have been largely accomplished. Planning-level hydrologic relations for quantitatively estimating the effects of surface storage on water availability and on certain aspects of water quality have been developed. However, there was not sufficient time available to develop quantitative relations for the other two supply alternatives mentioned in Objective 1, and only generalized considerations are possible for these.

Major steps have been made towards accomplishing the second objective. A mixed-integer multi-period programming model that utilizes hydrologic and economic data for identifying the least-present-cost scheduling of water-supply reservoirs for a community has been developed and run with sample data. We have also formulated a quantitative model of water-quantity and -quality relations at a series of water-use sites along a river network.

In addition, our research has led to the development of a hydrologic and economic decision-making framework for identifying and evaluating water-resource problems and solutions in the Merrimack River Basin and similar areas.

RESULTS OF HYDROLOGIC ANALYSIS

Definition of "Supply" and "Demand"

Overview

In economics, "supply" and "demand" have specific definitions: both are dependent on price, and the operation of market forces tends to make supply equal to demand. In water resources, by contrast, we are usually concerned with the capacity of a water-supply system, which is essentially a fixed value at any time, as compared to the rate at which water is or will be required. In general, the two values are not equal, and the manager is concerned with maintaining a capacity that exceeds the use requirements that are projected for some period into the future. Typically, these requirements are also considered to be fixed needs determined by the population and the type and level of economic activity in the region of concern, and are usually considered to be independent of the price of the water.

Thus, there are fundamental differences between the concepts of supply and demand as applied in economics as opposed to water resources. To avoid any confusion, we will use the terms "supply" and "demand" only in a very general non-economic sense, and will define under these headings specific terms that will help make distinctions that are critical to thinking about and solving water-resource problems.

Supply

As noted above, "supply", as used in the water-resources literature, generally refers to the capacity of a system to provide water. However, this capacity has two parts: 1) the hydrologic capacity of the source, which is related to the total runoff (precipitation minus evapotranspiration, both of which are determined by climate), the timing of this runoff (also determined by climate), and the amount of surface and subsurface storage available (determined by geology and human activity); and 2) the engineering capacity of works designed to distribute and treat the water. In this report, we will be concerned only with the hydrologic capacity of the source, which is often referred to as "water yield" or simply "yield".

Yield is measured as a flow rate or volume per unit time, typically in units of cubic feet per second (ft^3/s), gallons per day (gal/d), or liters per second (l/s). The amount of water available from any source - for example a particular location on a river - is highly variable in time, and the definition of capacity must account for this variability. The definition used here is: "the rate at which a source can supply water on 95% of the days". Thus, if a source is said to have a yield of 123 ft^3/s, this means that, over a long period of time, the flow rate of that source averages 123 ft^3/s or more on 95% of the days. On 5% of the days, the flow rate will average less than 123 ft^3/s. This definition of yield is used as a basis for planning because it is assumed that a water-supply system should not be inadequate (i.e., flow rate less than desired use rate) more than 5% of the time. The symbol "Y_{95}" is used for yield in this report.

The above definition of yield does not explicitly provide information about the <u>duration</u> of any shortages, which might also be important for planning purposes. One measure of streamflow that includes information about duration is designated "${}_7Q_{10}$". This designates the <u>seven</u>-day average flow rate which has a 90% (i.e., 100-<u>10</u>%) chance of being exceeded in each year. For example, if the ${}_7Q_{10}$ at a location on a stream is 86 ft^3/s, there is a 10% chance in any year that there will be a period of seven consecutive days for which the average flow rate is less than 86 ft^3/s. Figure 1 of Appendix A shows that, with one anomaly, there is a very close relationship between Y_{95} and ${}_7Q_{10}$ for streams in the New Hampshire portion of the Merrimack Basin. The empirical equations describing this relationship are:

<u>for streams with $Y^*_{95} < .12$:</u>

$$ {}_7Q^*_{10} = -.0099 + .651\, Y^*_{95} \qquad (3\text{-}1a) $$

$$ (r^2 = .996) $$

for streams with $Y^*_{95} \geq .12$:

$$_7Q^*_{10} = -.0263 + .843\ Y^*_{95} \qquad (3\text{-}1b)$$

$$(r^2 = .934)$$

where $_7Q^*_{10} = {_7Q_{10}}/\overline{Q}$, $Y^*_{95} = Y_{95}/\overline{Q}$, and $\overline{Q}$ is the long-term mean flow rate. Because of the closeness of these relationships, use of Y_{95} as a measure of yield conveys implicit information about duration of shortages.

It should be noted that design flows are not additive in a downstream direction. For example, referring to Figure 5, the Y_{95} at point C is not the sum of the Y_{95} values at points A and B.

Demand

The major classes of water use are: 1) withdrawal uses (domestic and municipal supply, industrial supply, and irrigation); and 2) instream uses (waste transport and treatment, hydropower, navigation, fish and wildlife habitat, esthetic amelioration, and recreation). Withdrawal uses are usually further classified as consumptive (water that, in the process of being used, is lost by evaporation, incorporated into products, transferred out of the drainage basin, or otherwise made unavailable for further use within the basin) and non-consumptive (water that, after being used, is discharged in liquid form in the basin such that it is available for further use, with appropriate treatment).

As noted earlier, "demand" in the context of water resources usually is interpreted as fixed requirements. Subsequently, we will show that requirements are not fixed by population and economic activity, but can be modified by policies such as water and waste-water treatment and pricing. Thus, we use the term "use rates", which like supply are measured as volume flow rates (discharges), rather than "demands" or "requirements".

Figure 6 is a schematic diagram of some water-using activity (municipality, industry, farm) which uses the adjacent stream as a water source. Define Q_1 as the design streamflow just upstream from the withdrawal intake, W as the rate of withdrawal, and C as the rate of consumptive use by the activity. Then, assuming $Q_1 > W$, the flow

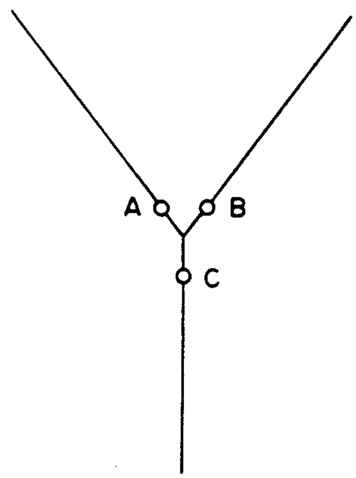

Figure 5. The Y_{95} at C is not equal to the sums of the Y_{95} values at A and B.

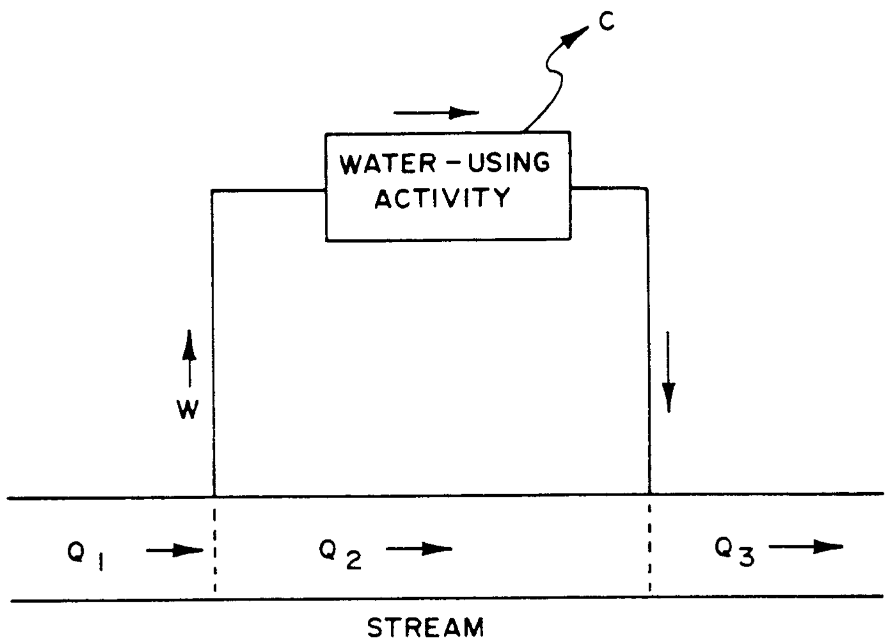

Figure 6. Conceptual diagram of a water-using activity obtaining water from and discharging to a stream.

in between the intake and the discharge, Q_2, is

$$Q_2 = Q_1 - W \tag{3-2}$$

and the flow downstream of the discharge, Q_3, is

$$Q_3 = Q_1 - C \tag{3-3}$$

Since $W \geq C$, the flow Q_2 is the smallest. In general, we would require

$$Q_2^+ \equiv Q_{min} \tag{3-4}$$

where Q_2^+ is the minimum allowable value of Q_2 and Q_{min} is a minimum-flow requirement for all instream uses except waste dilution and treatment. Downstream of the discharge pipe, we require

$$Q_3^+ \equiv \max\ (Q_{min}, Q_q) \tag{3-5}$$

where Q_3^+ is the minimum allowable value of Q_3 and Q_q is the flow required to maintain a specified water-quality standard (usually specified as maximum allowable concentrations of pollutants or minimum allowable concentration of dissolved oxygen. Appendix B is a discussion of standards that might be appropriate in the Merrimack Basin.). Upstream of the intake, the requirement Q_1^+ is simply

$$Q_1^+ \equiv W \tag{3-6}$$

The possible relations among the magnitudes of the required flows are

case a: $(Q_{min} + W) > Q_{min} > Q_q$

case b: $(Q_{min} + W) > Q_q > Q_{min}$

case c: $Q_q > (Q_{min} + W) > Q_{min}$

In case a, requirements Q_1^+, Q_2^+, and Q_3^+ are satisfied if

$$Q_1 \geq Q_{min} + W \tag{3-7a}$$

In case b, there are two possibilities. If $(Q_{min} + W - C) > Q_q$, then the requirements are also satisfied if

$$Q_1 \geq Q_{min} + W \tag{3-7b1}$$

but if $(Q_{min} + W - C) < Q_q$, the requirement becomes

$$Q_1 \geq Q_q + C \tag{3-7b2}$$

Finally, in case c, the requirement is again

$$Q_1 \geq Q_q + C \tag{3-7c}$$

If we consider a stream reach not supplying water to a use-point, W = 0 and case b cannot occur. The instream requirements are then given by the following variations of Equations 3-7a and 3-7b:

$$\underline{\text{if } Q_{min} > Q_q:} \tag{3-8a}$$

$$Q_1 \geq Q_{min}$$

$$\underline{\text{if } Q_{min} < Q_q} \tag{3-8c}$$

$$Q_1 \geq Q_q$$

In their study, Wollman and Bonem (1971) defined water use as $Q_q + C$, as in Equations 3-7b2 and 3-7c. This definition arose from considering $Q_{min} = 0$, i.e., flows sufficient to satisfy the water-quality requirement (Q_q) were considered to satisfy other minimum-flow requirements. However, their definition appears to ignore case 3-7b1, in which $(W - C) > Q_q$, and the required flow is W rather than $Q_q + C$. However, Figures 2 and 3 do suggest that the main water requirements in New England and the Merrimack basin are for water quality, so that case c may be most common. In such cases, water requirements are given by Equations 3-7c.

Fallacy of Adding Water Requirements or Deficits

The above development of equations for computing water needs makes it clear that one cannot estimate the water requirements of a drainage basin by adding the requirements for individual stream reaches or use points. This can readily be seen by considering the simple case of two use-points on a stream (Figure 7). Suppose the water requirements are given by Equation 3-7a for both locations, and assume the following values:

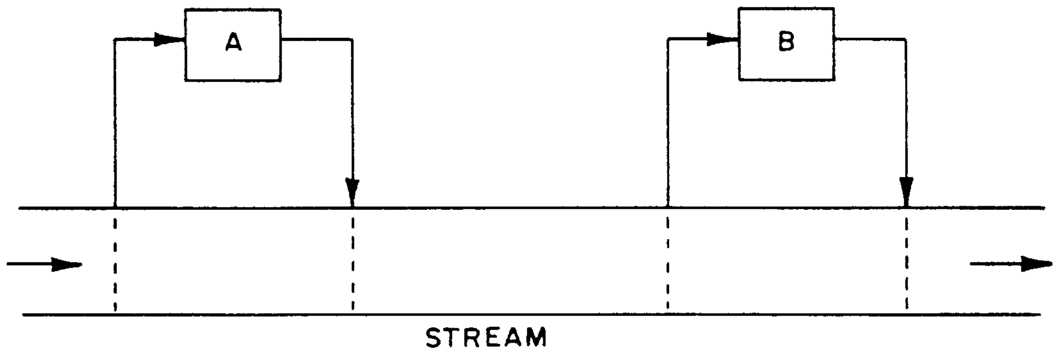

Figure 7. Two water-using activities on a stream. As explained in the text, the requirements at the two locations are not additive.

	Upstream Location		Downstream Location
	W = 100 units		W = 80 units
	Q_{min} = 20 units		Q_{min} = 25 units
Requirement	= 120 units	Requirement	= 105 units

Suppose the existing Y_{95} at the upstream location is 80 units and at the downstream location it is 85 units. The deficits at the two locations are then

Upstream deficit: 120 - 80 = 40 units

Downstream deficit: 105 - 85 = 20 units

The sum of these deficits is thus 60 units. However, suppose a reservoir is constructed upstream of both cities, and Y_{95} at the upstream location is thereby increased to the required 120 units. This will also cause Y_{95} to increase at the downstream location; however, as will be shown later, this increase will be less than the increase further upstream. Assume Y_{95} at the downstream location is increased to 100 units. There is no longer a deficit upstream, and the downstream deficit has been reduced from 20 to 5. Erasing both deficits might require, say, building additional upstream reservoirs to obtain a Y_{95} of 105 at the downstream location. Such an increase might result in an increase of Y_{95} at the upstream location to 135. Thus, solving the downstream problem in this way would result in a surplus at the upstream location.

Although this situation is fictitious, it illustrates that the sum of the two deficits is irrelevant to the magnitude of the solution to both problems. It also illustrates that adding the required water supplies is incorrect in assessing possible means for eliminating deficits. In short, the preceding analysis and definitions show that water requirements must be computed individually for each stream reach, and that an increase in supply (yield) implemented to alleviate a deficit at one location will affect, in a complicated manner, the supply at all locations downstream of where the increase in supply is implemented. Any framework for solving basin water problems must account for these facts.

Alternative Solutions to Water-Resource Problems

In the preceding discussion, the symbol Q_1 was used to designate any design flow entering a stream reach. To be consistent with earlier discussion, the design flow we have chosen is the specific value Y_{95}, so this symbol will be used henceforth. We have also shown that there are two definitions of water requirement, depending on the relative values of quantities in equations 3-7a - 3-7c. In order to further streamline the discussion, we will use the symbol Q_R for requirements, defined as $Q_{min} + W$ or $Q_q + C$, as appropriate.

Thus, we can now state that a water-resource problem exists at a stream reach or use point if it is predicted that $Q_R > Y_{95}$ at some time within the planning horizon. As shown in Table 3-1, there are several possible steps that might be taken to prevent the deficit from occurring, some of which increase Y_{95} and some of which decrease the predicted Q_R. A comprehensive framework for water-resources planning should permit the evaluation of all these types of solutions. The following section formulates such a framework.

Comprehensive Framework for Evaluation of Alternatives

Figure 8 is a schematic diagram of a use location adjacent to a river. This is essentially the same as the situation in Figure 6, except that provision has been made for recycling water within the use site and for treating the water before and after use. We now must define several additional terms, all of which are expressed as average (steady-state) discharge rates $[L^3/T]$:

$U \equiv$ rate of use of water

$J \equiv$ rate at which waste-water is treated

$R \equiv$ rate of recycling

$D \equiv$ rate of discharge to stream

Then the following relations are true

Table 3-1. 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	water pricing
water importation	water-use regulation/conservation
desalination	growth control
watershed management	
weather modification	

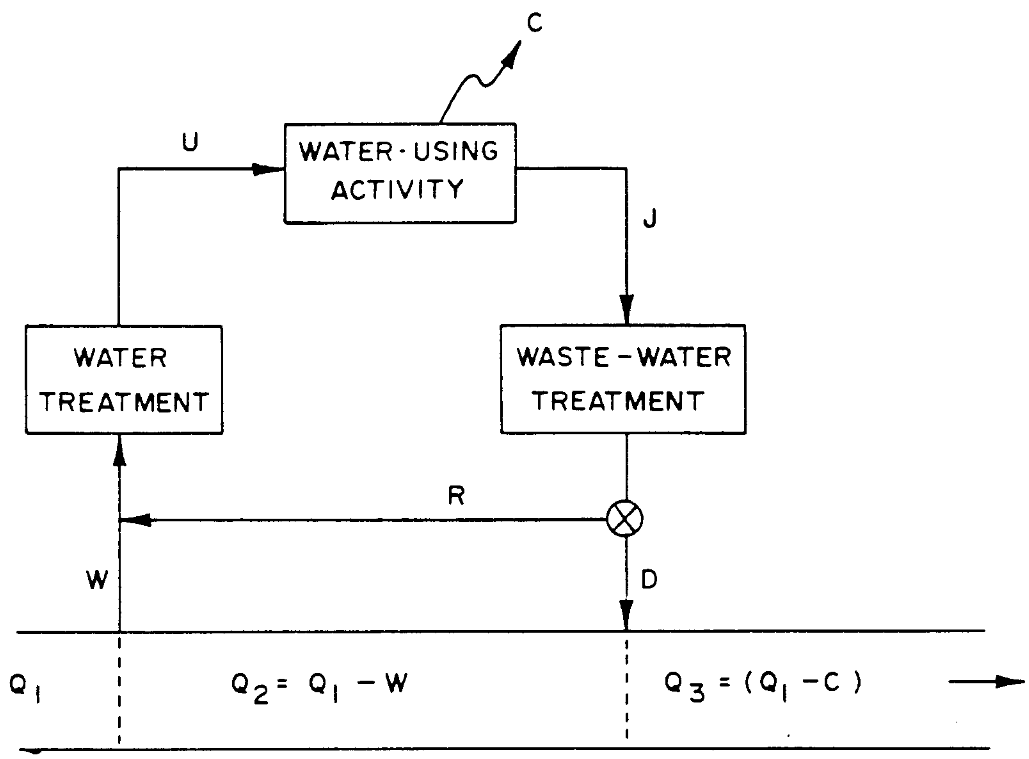

Figure 8. Definitions of water-quantity terms for framework equations. Symbols represent rates of water flow $[i^3/T]$.

$$U = W + R \tag{3-9}$$

$$J = U - C \tag{3-10}$$

$$D = J - R \tag{3-11}$$

$$D = W - C \tag{3-12}$$

Now we define the following demensionless factors:

$$k_c \equiv \frac{C}{U}, \text{ "consumptive-use factor"} \tag{3-13}$$

$$k_r \equiv \frac{R}{U}, \text{ "recirculation factor"} \tag{3-14}$$

$$k_d \equiv \frac{D}{U}, \text{ "discharge factor"} \tag{3-15}$$

Appropriate combinations of the above relationships will show that

$$W = (1 - k_r)U \tag{3-16}$$

and

$$k_c + k_r + k_d = 1 \tag{3-17}$$

For municipal supply, water use is expressed as

$$U_p = a_p P \tag{3-18}$$

where U_p is rate of municipal use $[L^3/T]$, a_p is the per-capita rate of water use $[L^3/T \cdot \text{person}]$, and P is the population served by the municipal system [persons]. The industrial use rate, U_I, is

$$U_I = \sum_i a_{Ii} \, I_k \tag{3-19}$$

where a_{Ii} is a process factor expressed as the volume of water required to produce a unit of product i $[L^3/\text{unit}]$, I_i is the rate at which product i is produced [units/T], and the summation is carried out over all water-using industries. Agriculture can be considered as an industry in this context. The factors a_p and a_{Ii} are inversely related to the price which the user must pay for water. The exact form of the relation must be determined by empirical data, which is not examined in detail herein.

At any time within the planning horizon, $U = U_p + U_I$, and is determined by the projected population, the projected types and levels of industrial activity, and the price of water to the users. Thus, U is considered to be determined by projections of population and economic activity for each point in time. The factor k_c is also fixed by the type of water-using activity - for example, in the domestic use of water, about 25% is consumptively used (Wollman and Bonem, 1971), so $k_c = .25$. Various industries have appropriate loss factors, for which estimates can also be obtained from Wollman and Bonem (1971). In the following, k_c is an appropriate weighted value for the use point at the time of interest.

With U and k_c determined by projected conditions and by a decision variable (the price of water), there remains one more decision variable that must be fixed: the recirculation factor, k_r. It might be of interest to point out the relation between the value of k_r and the number of times water is used, n_r:

$$n_r = 1 + \frac{k_r}{1 - k_r} \qquad (3\text{-}20)$$

which is the same as the "rate of recirculation", defined by Wollman and Bonem (1971) as U/W. Kuiper and Wecksler (1974) defined a "re-use factor", which is equal to $k_r/(1 - k_r)$ or $n_r - 1$. Table 3-2 shows the corresponding values of n_r and k_r over the range of values assumed possible by Wollman and Bonem (1971).

In the framework of Figure 8, when U, k_c, and k_r are fixed, Equations 3-9 to 3-17 determine the values of all the other values. In particular, we note that the required withdrawal rate, W, is determined by Equation 3-16 and the discharge rate, D, by Equation 3-12 or Equations 3-15 and 3-17.

If we were concerned only about water supply, and not about in-stream uses and quality, a deficit would exist if $W > Y_{95}$. In this case, all types of solutions in Table 3-1 except increased treatment could theoretically contribute to erasing the deficits. We would need hydrologic and related information to estimate how much increase in Y_{95} could be obtained from the supply-side alternatives. On the demand side, economic studies would suggest how much W could be reduced by

Table 3-2. Relation Between Measures of Water Re-Use

k_r	0	.1	.2	.3	.4	.5	.6	.7	.8	.9	.986
n_r	1	1.11	1.25	1.43	1.67	2	2.5	3.33	5	10	70

increasing price (thus reducing a_p and/or a_{Ii}). Trying different values of k_r in Equation 3-16 would show how much W could be reduced. Political and social information would be required to estimate the possibilities of regulation and reduction of projected population or industrial growth rates. To select the appropriate alternatives to implement, we would, of course, need estimates of the economic costs and environmental and social impacts of each.

We now proceed to introduce water-quality considerations into the framework. In Figure 9, each of the flows is multiplied by a concentration $[M/L^3]$ of some critical dissolved constituent designated by a small "c" and a subscript. As discussed in Appendix B, the critical constituents in New Hampshire are generally phosphorus and oxygen. Figure 9 and the following development are for an undesireable constituent added during use and removed by treatment, such as phosphorus. If oxygen, a desirable constituent that is removed by use and added by treatment, were used the arrows and signs designating the rate of removal in water treatment, F [M/T], the rate of addition in use, A [M/T], and the rate of removal in wastewater treatment, G [M/T], would be reversed.

In Figure 9, c_1 is fixed by upstream conditions, which must be specified or computed, as will be discussed later. From the water-supply model, U and C are determined by projections and prices, R (or k_r) is a decision variable, and all other quantities are determined by Equations 3-9 to 3-17 as described above. In considering water quality, an additional quantity, the rate of addition of pollutant A, is fixed by population and economic projections:

$$A = A_p + A_I \tag{3-21}$$

$$A_p = b_p P \tag{3-22}$$

$$A_i = \sum_l b_{Ii} I \tag{3-23}$$

where b_p is the per-capita rate of pollutant addition [M/T person] and b_{Ii} is the unit rate of addition for industrial process i [M/unit].

There are two additional decision variables for water quality: the degree of water treatment t_f and the degree of waste-water treatment

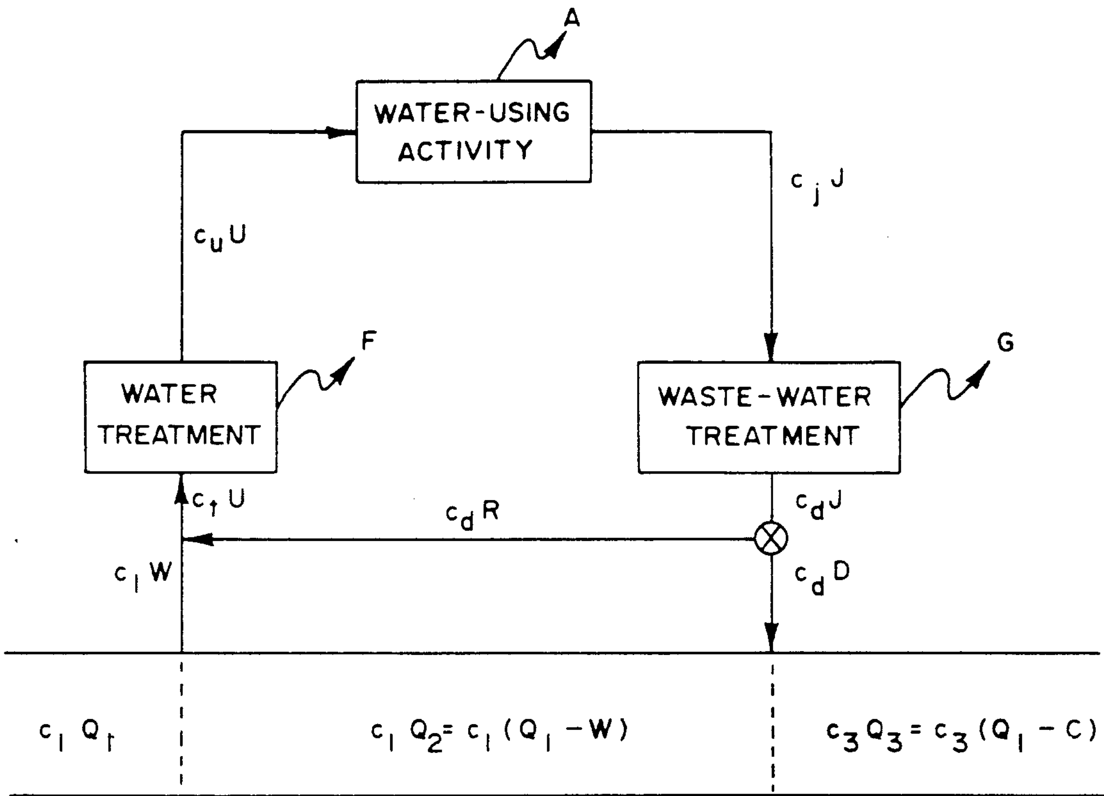

Figure 9. Definitions of water-quality terms for framework equations. Capital letters represent rates of water flow [i^3/T]; small letters represent concentrations of pollutants in flows [m/L^3].

t_g, where

$$t_f \equiv \frac{F}{c_1W + c_dR} \qquad (3\text{-}24)$$

and

$$t_g \equiv \frac{G}{c_jJ} \qquad (3\text{-}25)$$

Both t_f and t_g are dimensionless ratios. The remaining unknowns can be computed from the following system of equations, based on simple mass-balance considerations:

$$c_t = \frac{c_1W + c_dR}{U} \qquad (3\text{-}26)$$

$$c_u = \frac{c_tU - F}{U} \qquad (3\text{-}27)$$

$$c_j = \frac{c_uU + A}{J} \qquad (3\text{-}28)$$

$$c_d = c_j - \frac{G}{J} \qquad (3\text{-}29)$$

combining these, one can solve for c_j in terms of known quantities as

$$c_j = \frac{A + (1 - t_f)\ c_1W}{J - R(1 - t_g) + t_fR\ (1 - t_g)} \qquad (3\text{-}30)$$

Then Equation 3-25 can be solved for G, Equation 3-29 for c_d, Equation 3-24 for F, Equation 3-26 for c_t, and Equation 3-27 for c_u.

Of particular interest in light of the previous discussion of water requirements is the value of c_d, because the concentration of pollutant in the stream below the discharge, c_3, when the upstream flow is Y_{95} is

$$c_3 = \frac{c_1\ (Y_{95} - W) + c_dD}{Y_{95} - C} \qquad (3\text{-}31)$$

If this concentration is greater than the acceptable standard, c^+, a water-quality problem exists.

Equation 3-31 indicates that there are several ways in which c_3 can be reduced (assuming U and C fixed): 1) reduce c_1 by treatment, land-use practices, or other actions upstream; 2) reduce W and therefore D by increasing recycling or other actions; 3) reduce c_d by increasing treatment (t_g); 4) increase Y_{95} by building reservoirs or other actions (see Table 3-1). If this latter alternative is selected, the required flow to meet the water-quality standard, Q_q, can be calculated as

$$Q_q = \frac{c^+C - c_1W + c_dD}{c^+ - c_1}, \quad c^+ > c_1 \tag{3-32}$$

Appendix A is an application of these framework equations to a representative situation in the Merrimack Basin, while Appendix B is a discussion of water-quality criteria appropriate for New Hampshire.

Evaluating Alternative Solutions to Water-Resource Problems - Hydrologic Aspects

Introduction

Table 3-1 listed a number of alternative actions that are at least theoretically available to a water-resource manager faced with a projected water deficit. Some of these alternatives increase the yield (Y_{95}) available to the system, and others operate to reduce the water requirements as defined in previous sections. Although certain aspects of the hydrologic evaluation of some of these alternatives have been alluded to earlier, we now examine each in more detail.

Build Reservoirs

Reservoirs can be used to augment the Y_{95} available at a given use point in one of two ways: 1) the reservoir can be connected by an aqueduct directly to the use point; or 2) the reservoir can be used to regulate flows in a stream reach.

In the first instance (Figure 10), the water is normally provided

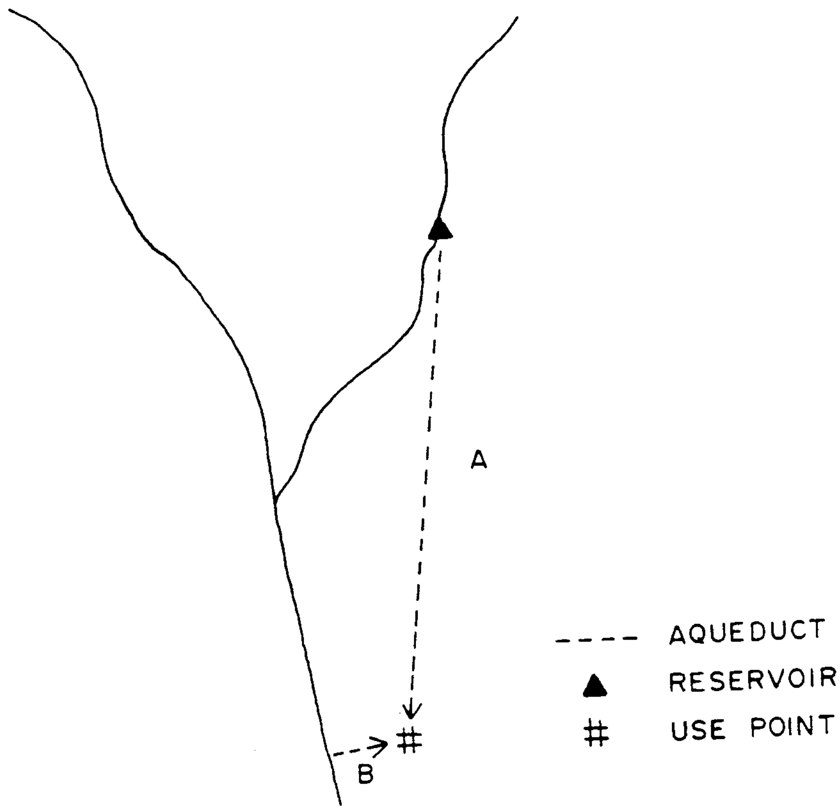

Figure 10. Alternative uses of reservoir for water supply: A - direct aqueduct connection utilizing safe-yield of reservoir; B - withdrawal of water from stream whose flow is regulated by reservoir.

for withdrawal uses. However, the Y_{95} is also increased in stream reaches downstream of the discharge point. As detailed in Appendix C, the yield (Y_{95}) provided by a reservoir-aqueduct system in the Merrimack basin can be estimated using the following relations:

$$Y^*_{95} = 1 - \frac{74.1}{78.0 + S^*} \tag{3-33}$$

$$Y^*_{95} \equiv \frac{Y_{95}}{\overline{Q}_{res}} \tag{3-34}$$

$$S^* \equiv u \frac{S}{\overline{Q}_{res}} \tag{3-35}$$

where S^* is the storage ratio of the reservoir expressed in days, $\overline{Q}_{res}$ is the long-term mean flow at the reservoir site, S is the active storage volume of the reservoir, and u is a unit-conversion factor.

If there is no other source of water to the use point, the Y_{95} value computed via Equations 3-33 to 3-35 would be the value of W in the framework equations. If there is another source of water used at a given location, the computation of W becomes more difficult, as one cannot simply add the Y_{95} values for two or more sources to get a combined Y_{95}.

Similarly, if the river at the use location is the source, or one wishes to compute the effects of discharges on Y_{95} values downstream, one cannot simply add. In this case, one can apply a slight modification of the method developed in Appendix C for computing downstream effects on Y_{95}, as follows. First, estimate the mean flow at the reservoir site. Then compute the regulation of the reservoir, R, as:

$$R_{res} = R^*_{res} \overline{Q}_{res} \tag{3-36}$$

where

$$R^*_{res} = \frac{1}{1 + (913/S^*)^{.625}} \tag{3-37}$$

Then compute the effective regulation at the reach of interest, R_{rch}, as

$$R_{rch} = \frac{R_{res}}{\overline{Q}_{rch}} \qquad (3\text{-}38)$$

where $\overline{Q}_{rch}$ is the long-term mean flow in the reach. Then Figure 6 of Appendix C is used to estimate Δ^*_{95} and the new value of Y_{95} in the reach, Y'_{95}, is

$$Y'_{95} = Y_{95} + \Delta^*_{95}\ \overline{Q}_{rch} \qquad (3\text{-}39)$$

The second type of use of reservoirs is for downstream flow augmentation, either for withdrawal or instream uses. The effect of upstream reservoirs on Y_{95} in a given downstream reach is computed via the method described in Appendix C. Again, this involves computation of R_{res} at all upstream reservoirs 1/ via Equations 3-36 and 3-37. Then one computes

$$(\Sigma R)^* = \frac{1}{\overline{Q}_{rch}}\ \Sigma R_i \qquad (3\text{-}40)$$

where R_i is the R_{res} value for each reservoir, Δ^*_{95} is found for the reach from Figure 6 of Appendix C, and the new Y_{95} is found from Equation 3-39.

Plate II (see Appendix I) is a map showing the location of potential reservoir sites identified in the Merrimack River basin by the U.S. Soil Conservation Service.

Ground-Water Extraction

In New Hampshire, ground water occurs in both surficial (glacial) deposits and bedrock. The water in surficial deposits is continually moving toward surface-water bodies, and is the source of most of the region's streamflow. Thus, ground water extracted from such deposits is water that would eventually become streamflow, and its extraction is conceptually no different from withdrawal from the stream. In fact,

1/ If two or more reservoirs are linked in series, only the downstream-most reservoir is used.

the yield of an aquifer hydraulically connected to a stream and without significant additional sources of recharge is ultimately the same as the yield from the stream reach affected by the withdrawal. For example, if one attempts to pump continuously at a rate of Y_{95} from such an aquifer, the water being pumped is being recharged from the stream and the rate of streamflow depletion will eventually reach Y_{95} (Jenkins, 1968; Jenkins and Taylor, 1974). This means that on 5% of the days, there will not be sufficient streamflow to support this rate of continuous pumping. Thus, although the timing of "shortages" may be different, the ultimate yield is the same for both the stream and the aquifer connected to it.

For surficial aquifers that are not connected to streams that provide significant recharge, the yield is more difficult to determine. Hall (1979) estimates that recharge to such aquifers in southeastern New Hampshire occurs at the average rate of 30 cm/yr, which is equivalent to 0.87 $ft^3/s\ mi^2$ or 9.5 $\ell/s\ km^2$. No studies of the statistical variations of yields of such aquifers have been done, but Hall (1979) does report sustained pumping rates in the range 10 to 50 ℓ/s (0.40 to 1.5 ft^3/s).

A preliminary discussion of computations of yields from isolated aquifers is given in Appendix D and concludes that, for planning purposes, the Y_{95} is equal to the long-term average rate of recharge. Using the above figures, a value of 9.5 $\ell/s\ km^2$ of aquifer could, therefore, be used. As shown in Appendix C, an average value of Y^*_{95} for New Hampshire streams with no surface storage is 0.05. If the long-term average streamflow is 17 $\ell/s\ km^2$, this amounts to about 0.85 $\ell/s\ km^2$, or less than 10% of the yield of an isolated aquifer on a per-unit-area basis.

Plate III (see Appendix I) shows the location of potentially productive aquifers identified by the U.S. Geological Survey. Many of New Hampshire's significant aquifers are adjacent to streams and would receive recharge from both the stream and as percolation from above. The total yield of the aquifer could be estimated as the sum of the yields available from each source.

Conjunctive Use of Surface and Ground-Water

Aron *et al*. (1974) have made the only detailed study of effective strategies for integrated use of ground and surface water in humid regions such as the Merrimack Basin. Their simulation study suggested that there would be considerable economic advantage to supplementing reservoir supplies with ground-water pumping. Figure 11, taken from their study, is an example of the effects of various levels of emergency ground-water pumping in increasing the yield of reservoirs of various sizes. For small storages such emergency use effectively increases the yield, but very large well-field pumping capacities are required to bring about significant yield increases for larger reservoirs. In these cases, Aron *et al*. (1974) recommended a program of "preventive pumping" in anticipation of seasonal streamflow deficits. In the New Hampshire section of the Merrimack Basin, 16 of the 52 communities with public water supplies have both surface- and ground-water sources. It is not known to what degree the two sources in these communities are conjunctively managed to take most efficient advantage of the characteristics of each type. However, integrated ground- and surface-water use has great potential as a water-supply strategy in New England, and deserves considerable further detailed study. Here we explore the hydrologic aspects of joint use of a river and an aquifer hydraulically connected to it as a water supply.

Figure 12 shows the effect of pumping water for a finite period of time from an aquifer that is hydraulically connected to a stream. Such pumping decreases the streamflow, but the effect is attenuated and drawn out in time. Jenkins and Taylor (1974) have shown that the magnitudes of the delay and attenuation are determined by a "streamflow-depletion factor", f_q:

$$f_q \equiv \frac{a^2 s}{T}$$

where a is the distance from well to the stream, s is the specific yield of the aquifer, and T is the transmissivity of the aquifer. The larger the value of f_q, the higher the delay and attenuation. Figure 13 illustrates how the delay in stream response can be exploited for a city which gets its primary water supply from a river, and whose water

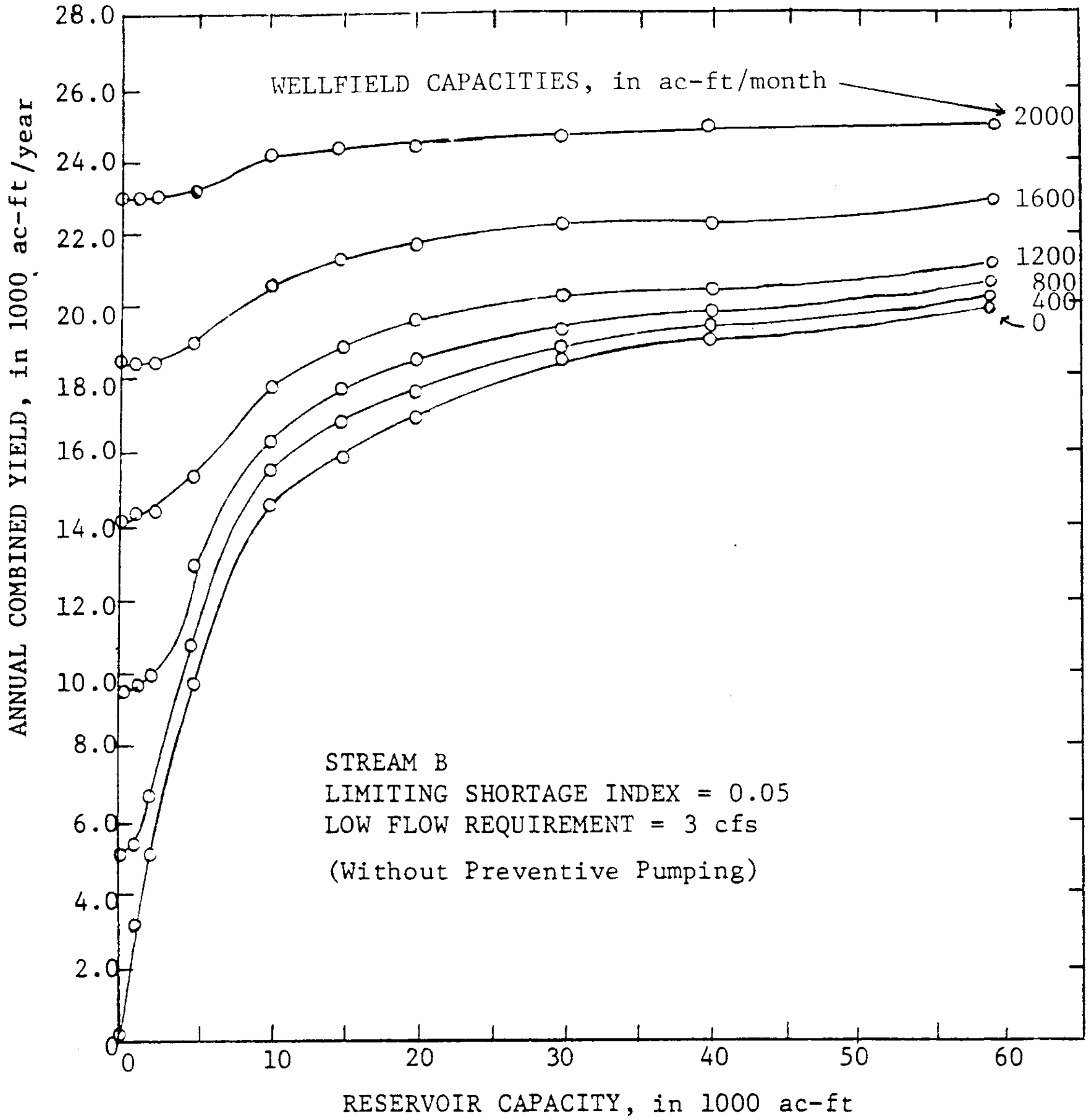

Figure 11. Effects of combining emergency pumping of ground water with reservoir for an example modeled by Aron, et al. (1974).

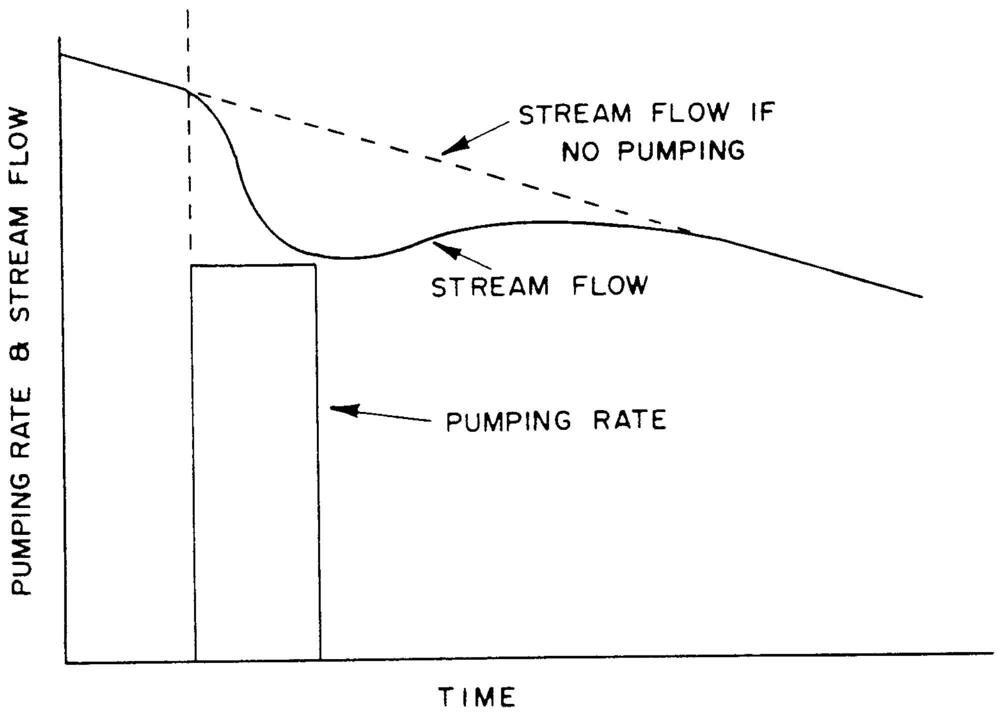

Figure 12. Schematic diagram of effect on streamflow of pumping of ground water from an aquifer connected to the stream.

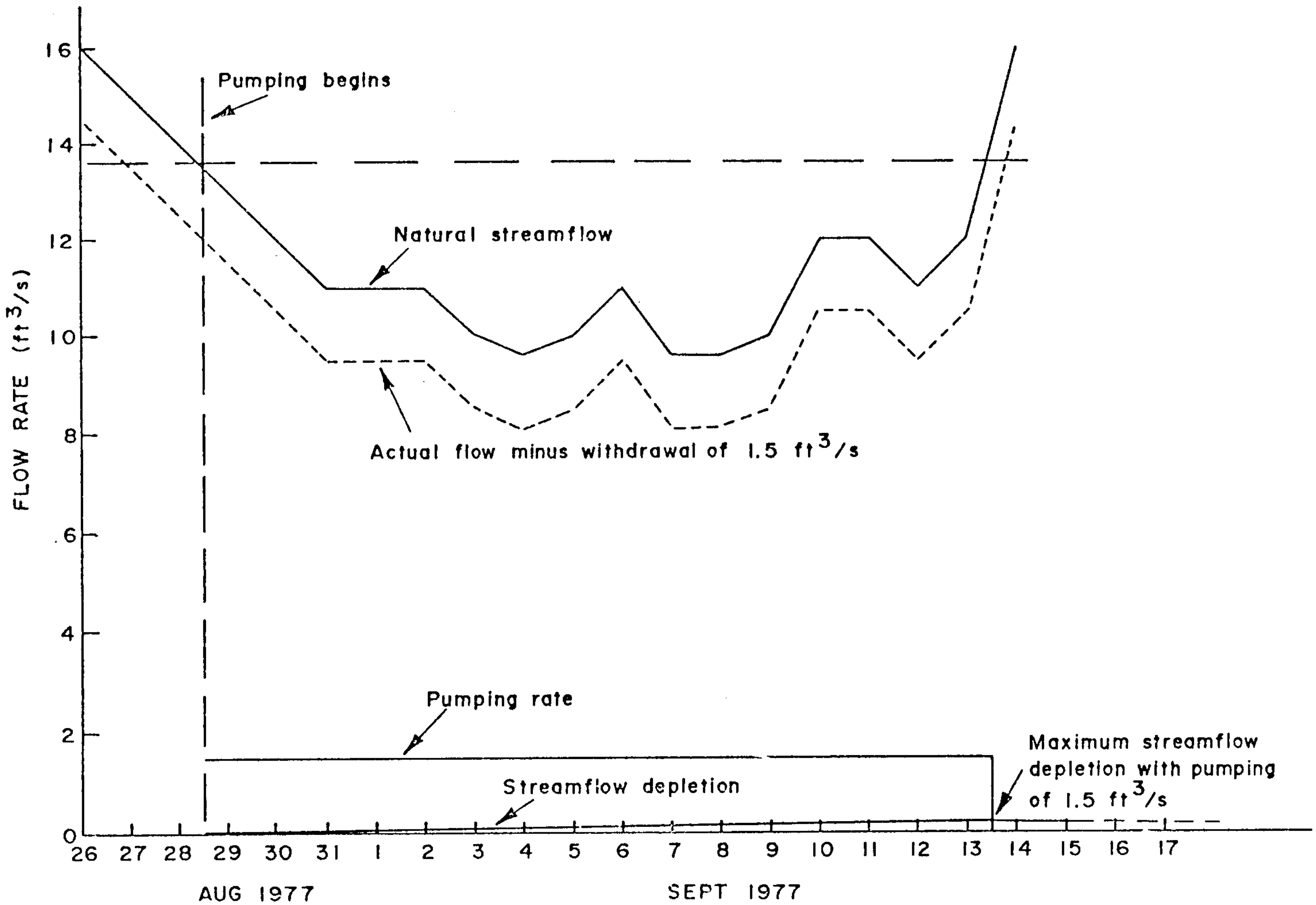

Figure 13. Example calculation of effect on streamflow of emergency pumping of ground water from adjacent aquifer compared with continual direct extraction from stream. Data for Peterborough, New Hampshire (Appendix E).

requirement, Q_R is determined by $Q_R = W + Q_{min}$. The calculations are made for the Contoocook River at Peterborough, New Hampshire (see Appendix E). The effect on streamflow of continuing to withdraw at the rate W in the absence of ground-water extraction is that the minimum-flow requirement is violated for an extended period. However, pumping from an aquifer connected to the stream reduces the impact on streamflow at any time during the pumping period, and reduces the time the minimum-flow requirement is violated. The degree to which one can avoid violating such a requirement depends, of course, on the exact magnitudes of f_q and of all the relevant flow requirements and the streamflow. This suggests that this mode of conjunctive use might be an effective strategy in the Basin.

Water Importation

Water importation is usually defined as the transfer of water into a drainage basin from a source located in another basin. By this definition, almost every connection of a reservoir to a use point constitutes importation, since the use point is seldom in the drainage basin that contributes water to the reservoir. Many ground-water sources also qualify as importation by this definition. Thus, the water available from an importation scheme is evaluated by applying the methods discussed earlier for reservoirs or ground-water sources.

It is important to re-emphasize that any withdrawal use of water, whether or not importation is involved, results in streamflow depletion between the withdrawal and discharge sites, and in general also causes a change in streamflow timing below the discharge site. Such changes may have significant effects on water quality and other instream uses such as fish and wildlife habitat, navigation, and hydropower. The importance of the changes can be estimated for planning purposes on the basis of the relative magnitudes and timings of the withdrawals as compared to the flows required for the other uses. If it appears that effects will be significant, more detailed simulation-model studies of the situation should be carried out.

Desalination

Desalination is the general term for several processes that separate dissolved solids in order to increase water quality to the extent that the water becomes suitable for some withdrawal use. There are two potential sources to which desalination could be applied: 1) water from a saline or brackish source, such as the ocean or estuary; and 2) water being discharged from a conventional waste-water treatment plant (recirculation).

Use of desalinated ocean water as a water source has traditionally been regarded as uneconomic in New England. However, in some coastal areas in southern New Hampshire where population pressure is high, suitable reservoir sites are scarce or committed to other uses, and ground-water sources are small and subject to salt-water intrusion, desalination of ocean water may be a viable alternative. Here, of course, the critical factors are economic (high energy costs) and environmental (brine disposal) rather than hydrologic.

As water is recirculated in a system its content of dissolved solids increases, and if the recirculation factor, k_r, is large, the water may have to be treated by desalination processes. The subsequent discussion of recirculation discusses this situation more fully in the context of the framework equations.

Watershed Management

This term covers a number of land-use alterations implemented with the goal of increasing water yield. Most commonly, the alterations involve replacing the existing vegetative cover with another that will result in lower evapotranspiration. In general, the annual evapotranspiration under a given climatic regime decreases in the following order:

conifer forest $\rightarrow$ hardwood forest $\rightarrow$ grass/shrubs $\rightarrow$ no vegetation

Thus, replacement of a vegetative cover with another lying to the right on the above scale results in an increase in the mean annual runoff, $\Delta\overline{Q}$, that can be computed as

$$\Delta\overline{Q} = - \Delta\overline{E} \qquad (3\text{-}41)$$

where $\Delta\overline{E}$ is the change in mean annual evapotranspiration.

A large number of experiments have been done to determine the magnitude of $\Delta\overline{E}$ under various conditions. For New England, the most pertinent of these studies are the deforestation experiments carried out at the U.S. Forest Service's Hubbard Brook Experimental Forest, which is in the Merrimack Basin in West Thornton, New Hampshire. These experiments have shown that for a watershed converted from a mixed hardwood forest to essentially no vegetation (maintained by application of herbicides), $\Delta\overline{Q}$ amounts to 24 to 34 cm/yr (7.6 to 10.8 ℓ/s km^2). Four years after herbicide application ceased, $\Delta\overline{Q}$ decreased to about 2 cm/yr (0.6 ℓ/s km^2) as regrowth occurred (Hornbeck and Federer, 1975).

While there is no doubt that increases in average runoff can be produced on managed watersheds, studies that demonstrate the viability of watershed management as an effective strategy for increasing yield, in New England and elsewhere, are largely lacking. A literature research has located only three studies that provide information on the increases in Y_{95} due to land-management practices, and the results of these are summarized in Table 3-3. Although the increases shown are substantial, it must be remembered that the periods of record are short, the effects decrease with time if regrowth is allowed to occur, and the watersheds are small.

A crude preliminary evaluation of the hydrologic potential of watershed management can be made by assuming that the increase of Y_{95} due to cutting (ΔY_{95}) is linearly related to the percent of watershed maintained in clearcut condition. (The effect is almost certainly non-linear, but in the absence of detailed studies the assumption of linearity can be made for approximate computations.) Assuming that the Pierce, et al. (1970) data apply to average conditions in the Merrimack Basin, we have

$$\Delta Y_{95} = 6.5\ A_c/A_a \qquad (3\text{-}42)$$

where ΔY_{95} is the increase in yield in ℓ/s km^2, A_c is the area of watershed maintained in clearcut, and A_d is total watershed area.

Table 3-3. Results of Studies on Increase in Y_{95} Due to Watershed Management Practices

Location	Watershed Area, km^2	Practice	Years of Record	Increase cm/yr	in Y_{95} $\ell/s\ km^2$	Source
W. Thornton, NH	0.16	clearcut, herbicides	1	20	6.5	Pierce et al. (1970)
Fernow, WV	0.23	clearcut, herbicides	1	12	3.8	Patric (1973)
Central PA	0.43	2/3 clearcut in two phases	6	3.5	1.1	Lynch et al. (1975)

Equation 3-42 can then be used to construct Figure 14, which relates percentage of clearcut area on watersheds of various sizes to increase in water yield. (Average per capita water use is about .048 ℓ/s).

It is also possible to modify Equation 3-42 to

$$\Delta Y^*_{95} = .37\ A_u/A_d \tag{3-43}$$

by taking an average value of mean flow, $\overline{Q}$, of 17.6 ℓ/s km^2. Equation 3-43 permits a direct comparison of the increases in yield possible from watershed management with those due to reservoir construction. By reference to Appendix C,

$$\Delta Y^*_{95} = .95 - \frac{74.1}{78 + S^*} \tag{3-44}$$

where S* is reservoir storage divided by mean flow and expressed in days, and the natural Y_{95} is assumed to equal .05 times the mean flow. Figure 15 shows the relation between S* and A_c/A_d given by the equations 3-43 and 3-44:

$$A_c/A_d = 2.7(.95 - \frac{74.1}{78 + S^*}) \tag{3-45}$$

This shows that the maximum possible proportion of clearcutting is equivalent to providing reservoir storage sufficient to hold 50 days of streamflow at the outlet of the treated watershed.

Both the absolute and relative effects of watershed management decrease downstream from the managed watershed. However, data do not exist to evaluate this effect beyond the linear assumption made in developing Equation 3-42.

Complete evaluation of watershed management as a strategy for increasing supplies will require using deterministic hydrologic models developed for the region, such as that of Federer and Lash (1978). These models should be run with input data representing at least several decades in order to develop an accurate picture of effects on yields.

Watershed-management practices will be most effective when reservoirs exist that can be used to increase ΔY_{95} over that obtainable from land-use alterations alone. The only study of this combined effect is that of Hawkins (1969), who used fictitious data. Application of his approach

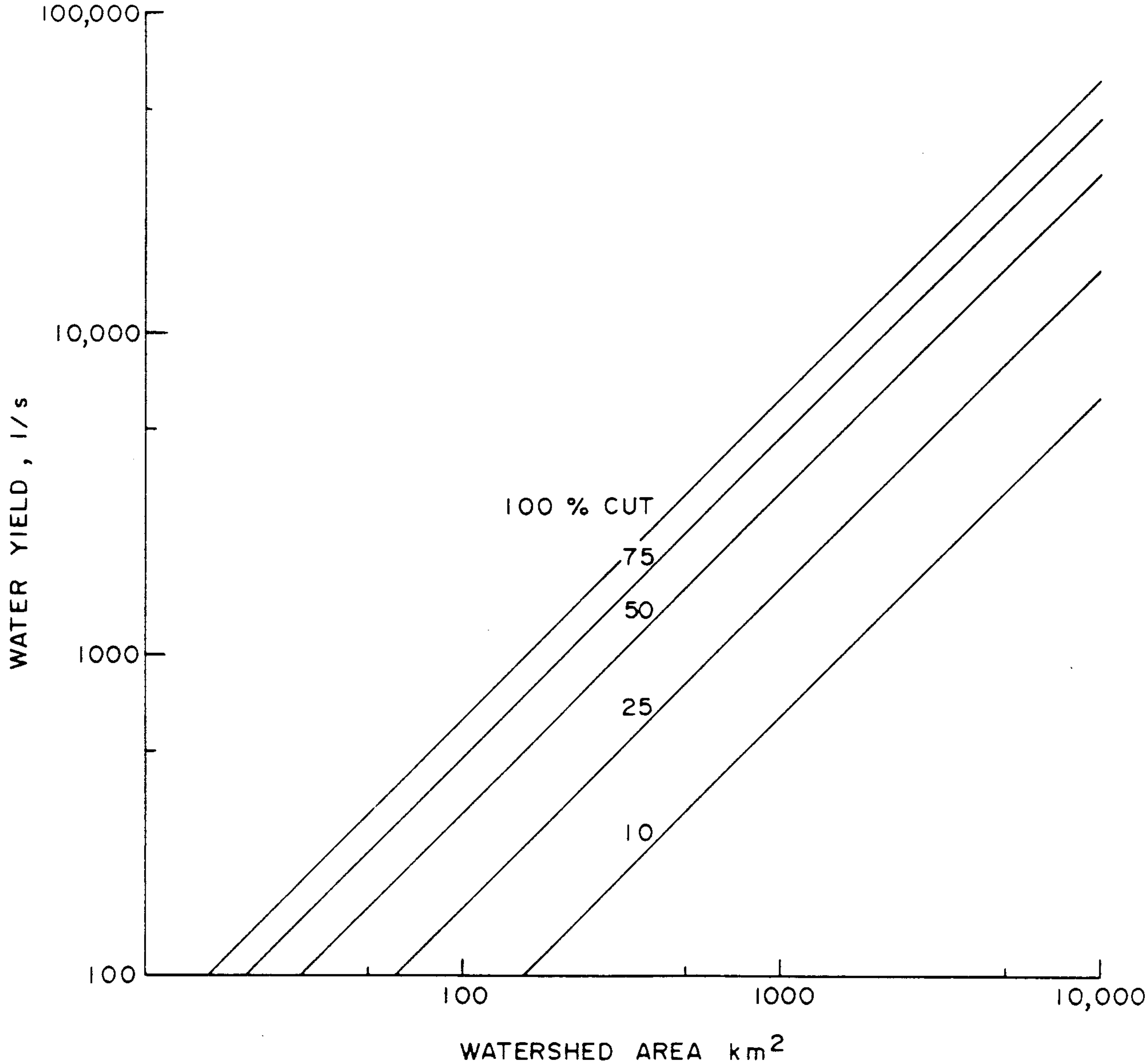

Figure 14. Estimated relation of water yield (Y_{95}) to watershed area and percent of area clearcut, assuming linear relation (Equation 3-42).

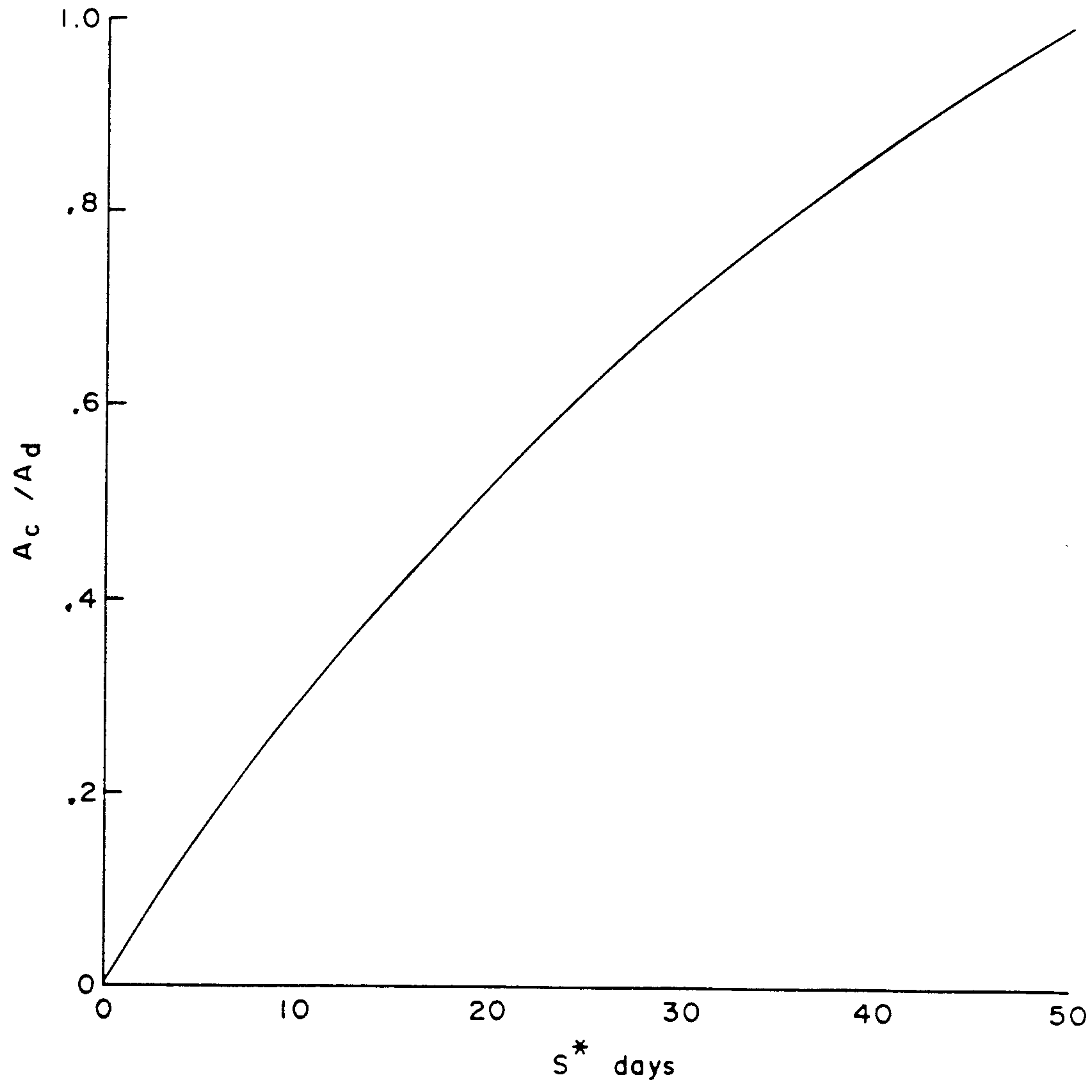

Figure 15. Estimated water-yield equivalence of clear-cutting a fraction A_c/A_d of a given watershed to providing a water-supply reservoir with a storage ratio S^* at the watershed outlet. Maintaining a 50% clear-cut area produces the same increase in yield as building a reservoir with a storage ratio of about 18 days; a 100% clear-cut is equivalent to a reservoir with a storage ratio of 50 days.

to New England, using model data as described above, would be most valuable.

It should be noted that the economic, environmental, and social impacts of watershed management are likely to be large, and assessment of these costs is essential for complete evaluation of watershed management as a strategy.

Weather Modification

Under some circumstances, it may be possible to increase the mean precipitation in a region by 10 to 14% (New England Division, 1977). A simulation study (Sopper and Hiemstra, 1970) analyzed the effects of precipitation increases of 10 to 30% in small watersheds in Pennsylvania, and concluded that significant increases in streamflow could result, particularly during the low-flow season.

However, there is still a large degree of uncertainty concerning the efficiency of rain-making in New England. A study in 1968 (Hoeh, 1968) indicated that New England water managers considered artificial enhancement of precipitation to be a strategy of last resort. This same attitude is reflected in recent considerations of this alternative for New England (New England Division, 1977). The feasibility of inducing significant increases in precipitation given New England's climatic situation, and the very formidable economic and social (especially legal) consequences militate against serious consideration of rain-making as a water-supply strategy at this time.

Further studies of this topic should include simulations of the type done by Sopper and Hiemstra (1970) using a model such as that of Federer and Lash (1978). As with watershed management, these studies should also be designed to evaluate the use of reservoirs to store the increased flow due to rain making. And, since the legal and environmental consequences of this strategy appear to be monumental, studies of these aspects are at least as important as the hydrologic questions.

Waste Treatment

Waste treatment is a potential management alternative when the water requirement for a stream downstream from a use point is given by $Q_q + C$ (see Equations 3-7b2 and 3-7c). The effect of this alternative is computed by Equation 3-31, which is repeated here:

$$c_3 = \frac{c_1(Y_{95} - W) + c_2 D}{Y_{95} - C} \tag{3-31}$$

As noted earlier, c_d is found by solutions of Equations 3-30, 3-25, and 3-29. Differentiating Equation 3-31 shows that the rate of change of c_3 with c_d is

$$\frac{dc_3}{dc_d} = \frac{D}{Y_{95} - C} \tag{3-46}$$

that is, it depends on the ratio of the rate of discharge from the use point to the river flow rate at the discharge point.

The computations in Appendix A (Table A-1) provide an example of how the flow rate required to meet a fixed water-quality standard changes as a function of treatment level; the data are plotted in Figure 16.

Recirculation

When the water requirement at a use point is determined by $Q_{min} + W$ (Equations 3-7a and 3-7b1), the effectiveness of recirculation as a policy can be determined directly from Equation 3-9:

$$W = U - R \tag{3-47}$$

or

$$W = U(1 - k_r) \tag{3-48}$$

Earlier discussion and Table 3-2 indicated that values of k_r as high as .986 might be possible. The principal physical limiting factor for k_r is the build-up of dissolved solids that occurs as water is re-used, and the framework equations can be used to evaluate this effect for selected dissolved constituents. The critical consideration is the concentration of pollutant in the water being used, c_u, and this can be computed by solving the framework equations in the following order: find c_j from Equation 3-30, G from Equation 3-25, c_d from

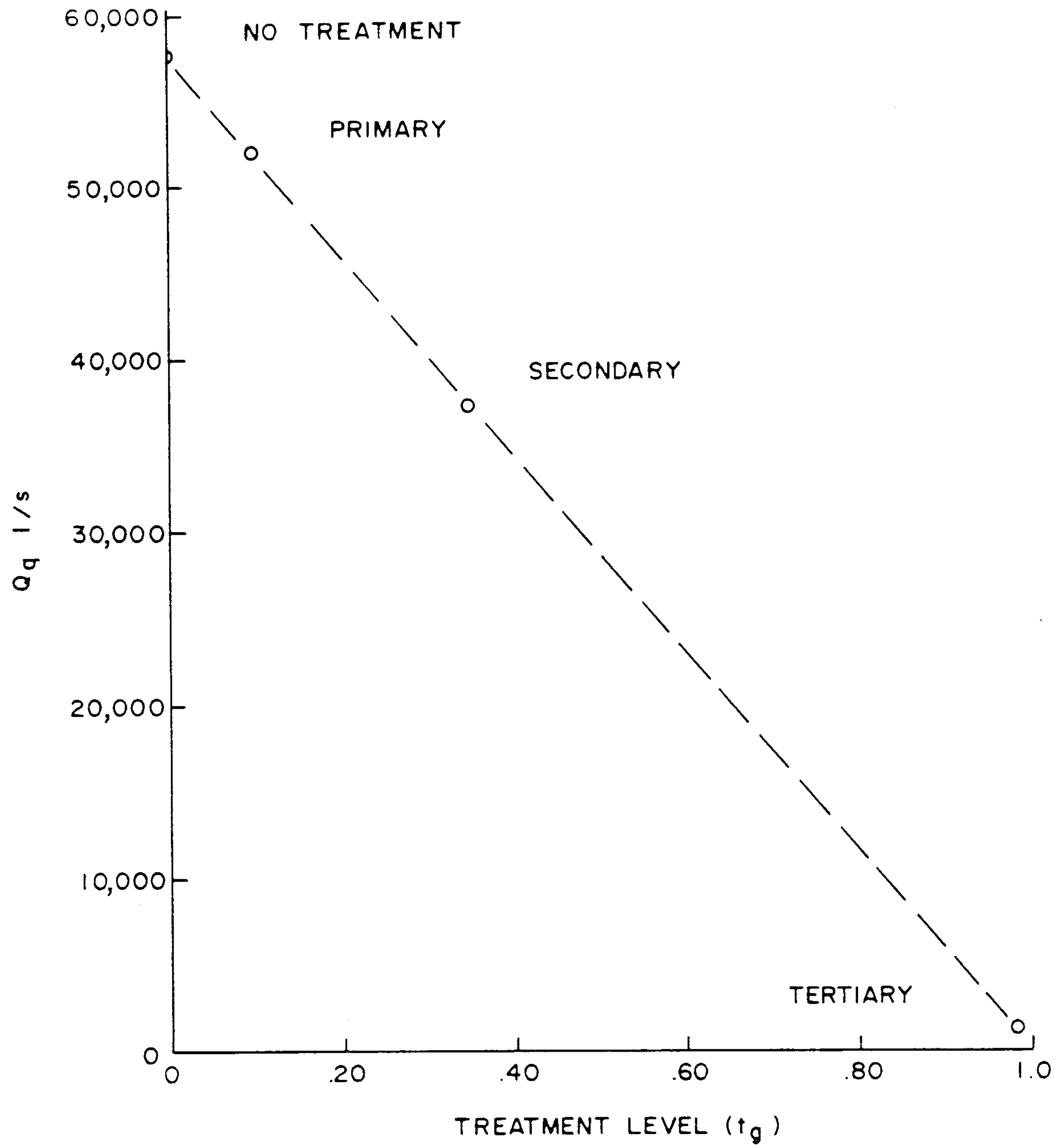

Figure 16. Illustration of effect of treatment level on water requirement for water-quality maintenance, Bow, New Hampshire example (Appendix A).

Equation 3-29, F from Equation 3-24, c_t from Equation 3-26, and finally c_u from Equation 3-27.

If we consider a pollutant that is not reworked by normal water or wastewater treatment (e.g., chloride or nitrate), so that t_g and $t_f = 0$, the framework equations reduce to

$$c_j = \frac{A + c_1(1 - k_r)\,U}{U(1 - k_c - k_r)} \tag{3-49}$$

and

$$c_u = c_1(1 - k_r) + c_j k_r \tag{3-50}$$

Equation 3-48 shows that k_r must be limited by the value of k_c for a particular use, such that

$$k_r \leq 1 - k_c \tag{3-51}$$

Table 3-4 summarizes data from Wollman and Bonem (1971) on values of k_c and hence maximum values of k_r for various withdrawal uses.

Within the limits dictated by Table 3-4, values of k_r are further restricted by the maximum allowable values of c_u for critical dissolved pollutants. Appendix F shows a sample computation for chloride, using data applicable to municipal use in Bow, New Hampshire, and the results are shown in Figure 17. It is assumed that neither water nor wastewater treatment processes remove chloride (this assumption would also be true for certain other critical pollutants, such as nitrate.) Assuming the U.S. Public Health Service limit of 250 mg/ℓ concentration of chloride in water for municipal use, Figure 17 indicates that the maximum permissible value of k_r for this case is about 0.48.

If treatment removes some fraction of a critical dissolved pollutant, the maximum permissible value of k_r can be raised. This is illustrated for phosphorus in the Bow, New Hampshire situation in Figure 18. For an assumed upper limit for c_u of 1 mg/ℓ, for example, the maximum k_r is .1, .125, and .15 for no, primary, and secondary treatment, respectively. With tertiary treatment, the upper limit of k_r is again controlled by k_c.

Interestingly, recirculation can also contribute to solution of water-resource problems when the water requirement is given by Q_q + C. Again, using the data for the Bow, New Hampshire situation, the

Table 3-4. Average Values of k_c and Limiting Values of k_r for Various Withdrawal Uses (from Wollman and Bonem, 1971)

Use	k_c	Maximum k_r
Municipal	.25	.75
Manufacturing-food	.10	.90
Manufacturing-pulp & paper	.06	.94
Manufacturing-chemicals	.09	.91
Manufacturing-petroleum & coal	.11	.89
Manufacturing-primary metals	.06	.94
Steam-electric power	.01	.99

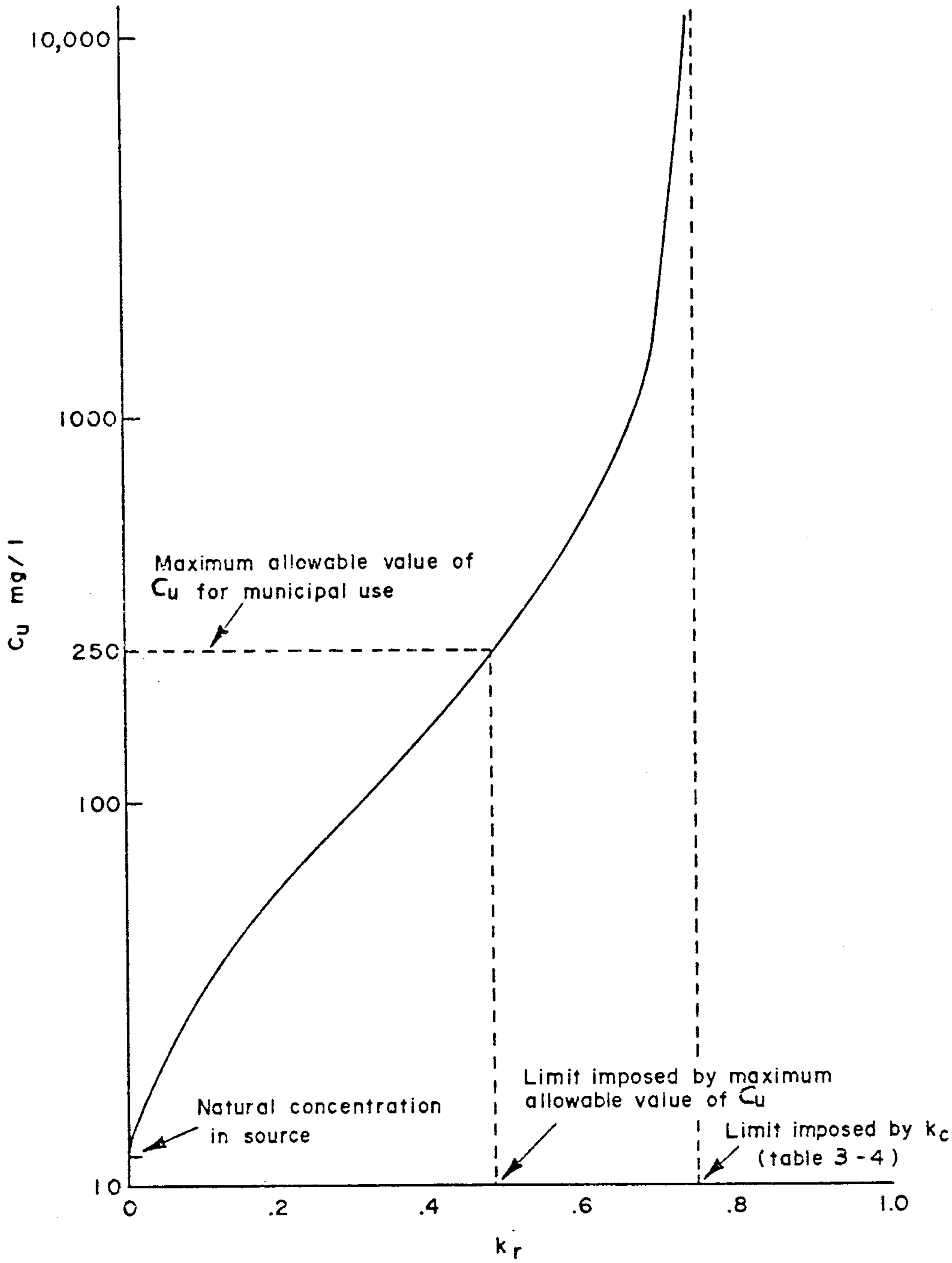

Figure 17. Illustration of effect of maximum allowable concentration of pollutant in use water, C_u, on maximum allowable degree of re-use, k_r, using example of chloride at Bow, New Hampshire (Appendix F).

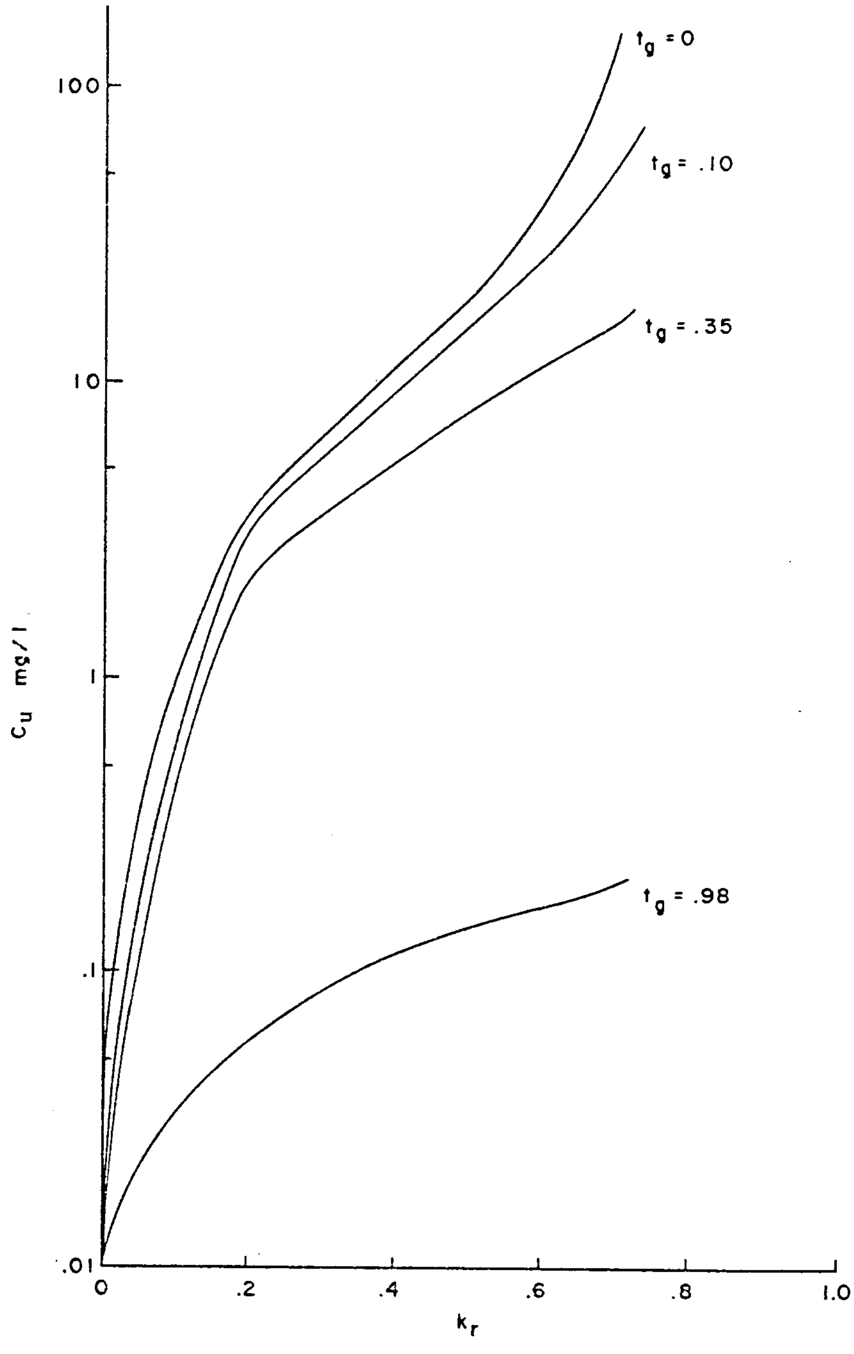

Figure 18. Relation between maximum allowable value of pollutant, C_u, degree of treatment, t_g, and maximum allowable degree of re-use, k_r, using example of phosphorus at Bow, New Hampshire.

concentration of phosphorus in the Merrimack River downstream from the discharge, c_3, can be computed as a function of treatment level and k_r via the framework equations. The results are shown in Figure 19, and indicate that, though the effects are relatively small, instream water quality is improved by recycling, particularly at primary and secondary treatment levels.

Water Pricing

Many articles have been written on the effects of water pricing on water use. Sharpe (1978) summarized some recent studies and concluded that pricing is not generally very effective in controlling water use, as domestic water use rates in particular are relatively unresponsive to changing prices (i.e., water use is price inelastic). However, large water-using industries may reduce water usage if water costs are significant, and pricing structures that charge more for a unit of water as usage increases may help induce such a response.

In the context of the framework equations, any effects of price on usage would enter through effects on the per-capita rate of water use, a_p, and the industrial process factors a_{Ii} (see Equations 3-18 and 3-19). Considering only a_p as an example, it might be possible to relate that factor and price by an equation of the form

$$a_p = \alpha \$^{-\beta} \tag{3-52}$$

where \$ is the price per unit of water and α and β are empirical constants. Then from Equation 3-18,

$$U = \alpha \$^{-\beta} P \tag{3-53}$$

where P is population. The definition of demand elasticity, E, is

$$E \equiv \frac{dU}{d\$} \cdot \frac{\$}{U} \tag{3-54}$$

From Equation 3-53,

$$\frac{dU}{d\$} = -\alpha\beta P \$^{-\beta-1} \tag{3-55}$$

and substituting Equations 3-53 and 3-55 into 3-54 gives

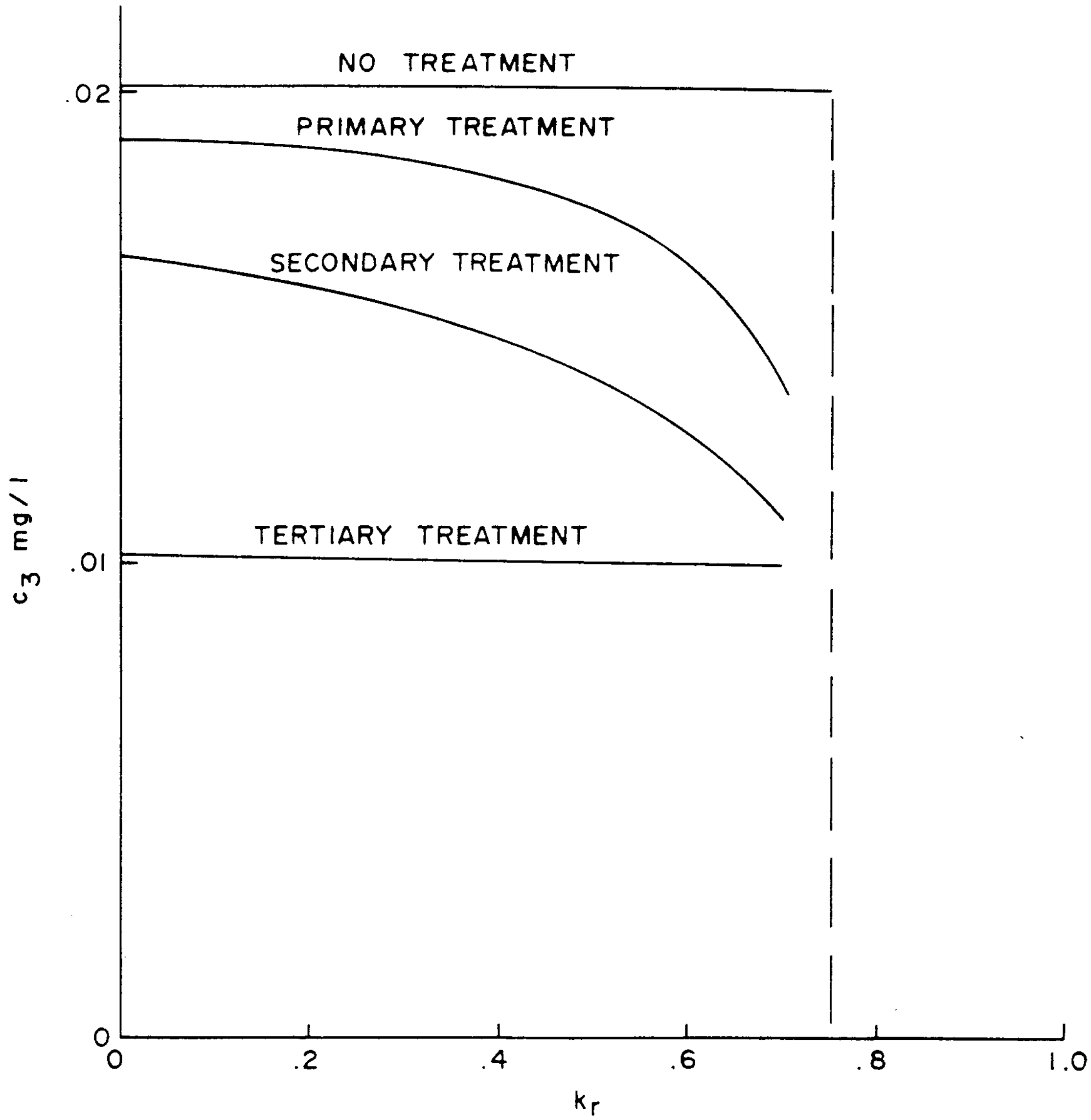

Figure 19. Effect of degree of treatment and degree of re-use, k_r, on downstream concentration of pollutant, c_3, using example of phosphorus at Bow, New Hampshire.

$$E = (-\alpha\beta P\$^{-\beta-1}) \frac{\$}{\alpha\$^{-\beta}P} = -\beta \qquad (3\text{-}56)$$

Thus, if Equation 3-52 is valid, the exponent in that relation is the demand elasticity for water.

Water-Use Regulation/Conservation

This term is used herein to include both official restrictions on water use and "conservation", which is defined as any voluntary actions taken to reduce usage while maintaining a given population or level of industrial activity.

Usually water-use regulation is considered as a strategy when the water requirements are determined by $Q_{min} + W$, and the goal is to reduce W. In these cases, as with pricing, any effects enter the framework equations via the per-capita water-use factor a_p or one or more industrial process factors a_{Ii} (Equations 3-18 and 3-19). To evaluate this strategy, then, one would simply apply appropriately reduced values of a_p and/or a_{Ii}, which would result in proportional reductions in W.

In situations where water requirements are given by $Q_q + C$, which is probably the case for most communities in the Merrimack Basin, water-use regulation/conservation is not an appropriate strategy.

Growth Control

The framework equations are designed for planning use, in which projections of population and industrial activity are made for future planning horizons. Then W, Q_q, C, and Q_{min} are computed as a basis for estimating water requirements by Equations 3-7a - 3-7c, as appropriate. All these terms except Q_{min} depend on the level of population and industrial activity used in the computations, through Equations 3-18, 3-19, 3-22, and 3-23. Thus, a possible strategy for reducing future water requirements is to enact policies such as zoning regulations which would limit future population or industrial growth.

RESULTS OF ECONOMIC ANALYSIS

Introduction

Water resource planning usually involves a decision-making authority faced with a situation of how to select a water project from a set of alternatives with known costs and capacities and to implement this project at a point in time based upon a predetermined objective. For a defined planning period, the problem becomes a dynamic one in which the first project selected is dependent upon the other decisions remaining over time. Butcher, *et al* (1969), Morin (1973), Morin and Esogbue (1974), and Haimes and Nainis (1974) illustrate this type of problem.

The above discussed problem in general terms can be classified as either a sequencing or scheduling problem. According to Knudsen and Rosbjerg (1977), if future water demand is assumed to be deterministic, operation and maintenance costs are considered a negligible component of total costs and/or all the projects under consideration have roughly equivalent per unit variable costs, and the aggregate capacity for all proposed projects equals demand at the end of the planning period, then a *sequencing* problem exists. This is because all proposed projects must be constructed with the basic problem being the optimal order of implementation. When the assumption of aggregate supply for all proposed projects equated to final time period demand is relaxed, the situation then becomes a *scheduling* problem. The optimizing framework conceptually becomes one of both selecting and sequencing a required number of projects. This implies that it does not necessarily follow that all proposed water projects must be selected, as was the case under the sequencing problem.

For our purposes, the Merrimack River Basin water planning model is classified as a scheduling problem. The first two previously discussed assumptions--deterministic future water demands and operation and maintenance costs are deemed to be a small percent of total costs--are satisfied within the developed framework.

A model was developed for three towns in the Merrimack River Basin with consideration given to four future time periods for water planning purposes based upon projected water demand requirements for each of the time intervals. The objective of the model is the selection of a plan which will minimize total discounted cost over the planning horizon of the construction and operation of the water system, subject to various constraints (water demands, water system yields, . . .).[1/] Seven potential reservoir sites were evaluated and will be discussed in a later section. A mixed integer programming algorithm was utilized for computational purposes and a general overview is presented below.

Mixed Integer Programming

Mathematical programming is generally based upon the simplest of all programming activities -- linear programming (LP); where, its principles are expounded in Dantzig (1963) and Hadley (1962) with practical applications contained in Vajda (1961), Beale (1959) and Heady *et al* (1967). LP basically relates to a problem consisting of maximizing (or minimizing) a linear objective function subject to linear constraints. The optimal solution will generally consist of noninteger values. Notationally, the LP problem is expressed as follows:

$$(1)\ \text{Maximize (minimize) } Z = C'X$$

$$AX \lesseqgtr B,\ X \geq 0$$

1/ A recent trend in project or public investment evaluation has been the development of decision-making frameworks which consider more than one objective function. This developing field, as reviewed by Cohon and Marks (1975) and Loucks (1975), is commonly referred to as vector optimization or simply, multiobjective programming. This technique makes it possible to incorporate and evaluate the explicit tradeoffs among noncommensurable objectives. For our present purposes, emphasis is upon a single objective optimization problem with future possibility resting with the adoption of an appropriate multiobjective framework.

where, A is a m x n matrix of technical coefficients, C is a n x 1 vector of prices or other weights for the objective function, X is a n x 1 vector of activities, B is a m x 1 vector of resource or other restraints, and C'X = Z is the objective function.

A special case of linear programming is mixed integer programming (MIP). As was the case under the LP framework, the objective function and constraints are linear, but this particular mathematical programming variant allows for the optimal solution to contain integer values as well as noninteger values. Constraints are designated which force some variables to take on either zero or one values and allow for these variables to be introduced only once, if at all. This aspect is entered into the MIP framework because lack of this feature would cause the variables representing decisions on whether to introduce particular investments to take inadmissable fractional values. Such results would have no real world interpretation. For example, it makes no sense to derive a solution that builds six-tenths of a reservoir in period one and three-tenths in period two, and so forth. Notationally, the MIP problem is as follows:

(2) Maximize (minimize) $Z = C'X$ subject to

$$AX \begin{smallmatrix}\le\\=\\\ge\end{smallmatrix} B, \quad X \ge 0$$

x_j is an integer = 0 or 1

x_i is a noninteger

where, x are elements in the X vector of activities and the interpretation of notation that applied under the LP framework also holds for the MIP general model.

Prior to 1958 no computational procedure existed that would result in integer solutions to mathematical programming problems. In 1958 Gomory (1958) developed the "cutting-plane" method for solving integer programming problems. In contrast to the efficient simplex method used in solving LP problems, this method yields an optimum integer solution in a finite number of steps and raises concerns about computational efficiency.[2/] Researchers are working on alternative methods of solving MIP problems with the objective of improving computational efficiency. For example, some noteworthy contributions to the literature on integer programming are Dantzig (1960), Glover (1966), Gomory and Baumol (1960), Markowitz and Mame (1957), and Gomory (1960).

In contrast to the "cutting-plane" method, the "branch-and-bound" procedure has been commonly used.[3/] The basic MIP problem is first solved as if a LP problem exists. Next, a subset problem is generated which forces one of the possible integer variables to have a value of zero and another subset problem which forces this same variable to be one. Since two branches are formed for comparative purposes, the program chooses for a minimization problem the cheaper branch and allows the other branch for additional comparisons. Two additional subset problems are organized to investigate comparisons with another variable and the process is repeated.

For our purposes, the MIPZ1 mixed integer programming package developed by Bravo, et al (1970) was utilized. The algorithm used in this program is basically a modification of the Additive Algorithm of Balas (1965). The major modifications include a reordered enumeration tree and the addition of the mixed integer option. McCarl, et al (1973) present an indepth discussion of each modification.

Study Area

The New Hampshire towns of Hudson, Merrimack, and Nashua contained in the Merrimack River Basin were selected as the unit of analysis for application of the previously mentioned model.

2/ In general, computational efficiency is related to the number of computational steps required to reach an optimum solution. Balinski (1965) intensively discusses this concept.

3/ This method is often considered to be more efficient than the former method based on feasible solutions being designated sub-optimal early in the procedure.

In 1970, these towns had average daily water demands in million gallons of .50, .74, and 9.55, respectively; and it is anticipated that by 1990 these demands will reach 2.80, 4.00, and 10.00, respectively (Merrimack River Basin Water Supply Study, 1977). It was felt that the demand levels are indicative of communities that can be classified as low, medium, and high water users and therefore represent a study area with variation accrued to use. It is also projected that by the year 2000 Hudson, Merrimack, and Nashua will have populations per square mile at levels of 771, 1103, and 3116, respectively (Nashua Regional Planning Committee, 1977). These figures reflect varying population density which again points to a heterogeneous study area.

Presently, Hudson is supplied with water by the Hudson Water Company, which is a privately owned firm. The water source is four gravel packed wells and has an estimated present sustainable yield of 1.75 m.g.d. For 1977, the number of parties (homes and firms) served was about 2300 with about 86 percent metered (NHWSPC, 1977).

Merrimack is supplied water by the Merrimack Village District which is classified as a municipal district. Five gravel packed wells serve as the present source with estimated safe yield to be 5.18 m.g.d. The District provides water for domestic, mercantile, commercial, industrial, and fire protection uses. For 1977, the number of units served was approximately 3500 with 100 percent metered (NHWSPC, 1977).

Since 1852, Nashua has been supplied with water by Pennichuck Water Works, an investor owned company. The sources of supply include both ground and surface water sources for an estimated present sustainable yield of 13.70 m.g.d. For 1977, about 14,400 parties were served with 99 percent metered (NHWSPC, 1977).

Consideration was given to seven potential reservoir sites for possible water supply augmentation for each of the three towns for future time periods. Of the seven sites, three are located each in Hudson and Nashua with the remaining proposed reservoir in Merrimack. Table 4-1 contains data pertinent to each site. All considered sites are located within the defined study area. This was done because it was unrealistic in the initial stages of model development to evaluate sites outside the study area based upon political jurisdictional

Table 4-1. Data Overview of Proposed Reservoir Sites.

	Reservoir	Construction Costs (1980)	Y (m.g.d.)	Q_{95} (m.g.d.)	$Y-Q_{95}$ (m.g.d.)	Surface Area (acres)
Hudson	1	$1,626,947	17.1	1.87	15.3	405
	2	319,474	0.52	0.06	0.46	96
	3	773,230	2.52	0.19	2.33	190
Merrimack	4	426,813	1.29	0.06	1.23	115
Nashua	5	229,870	0.45	0	0.45	49
	6	587,580	0.78	0.06	0.72	350
	7	366,324	1.03	0.06	0.97	135

Y : reservoir yield

Q_{95} : natural yield without reservoir

$Y-Q_{95}$: net reservoir yield

considerations. Future refinement of the model could possibly evaluate sites external to the boundaries of the three towns. These seven proposed water supply areas were selected not only because of location, but also based upon the variability that exists among costs, surface area, and Y-Q. It was felt that such a set of alternatives would make for a less constraining situation for the decision-making process. Further development of the potential sites could include some of the previously discussed alternatives contained in the hydrologic section of this report. Lack of data precluded enclosure at this time.

Conceptual Model

As was previously discussed, a mixed integer programming model was developed for the three towns of Hudson, Merrimack, and Nashua, New Hampshire in the Merrimack River Basin with consideration given to the time periods 1981-1990, 1991-2000, 2001-2010, and 2011-2020 for water supply augmentation purposes based upon projected water demand requirements for each of these time spans. Appendix G contains a discussion of the method used to derive "water use over time" values for each of the three towns. The objective of the model is the selection of a scenario that will minimize total discounted cost over the four time periods for the three towns of the construction and operation and maintenance (O & M) of new reservoirs, construction and O & M of new pipeline systems, O & M of existing wells, and O & M of currently existing pipelines.

Notationally, the developed model is presented below with an explanation following the objective function and each constraint set. Economic data utilized in this model are in Appendix H.

$$\text{Minimize } S = \sum_{t=o}^{T} \sum_{i=1}^{I} [C_i(1+r)^{-t} y_{it} + b_i(1+r)^{-t} W_{it}] + \tag{4-1}$$

$$\sum_{t=o}^{T} \sum_{i=1}^{I} \sum_{j=1}^{J} [d_{ij}(1+r)^{-t} V_{ijt} + e_{ij}(1+r)^{-t} w_{ijt}] +$$

$$\sum_{t=o}^{T} \sum_{z=1}^{Z} [g_z(1+r)^{-t} Q_{zt}] + \sum_{t=o}^{T} \sum_{j=1}^{J} \sum_{z=1}^{Z} [m_{zj}(1+r)^{-t} X_{ztj} +$$

$$k_{zj}(1+r)^{-t} u_{zjt}] + \sum_{t=o}^{T} \sum_{j=1}^{J} \sum_{z=1}^{Z} [1_{zj}(1+r)^{-t} q_{zjt}]$$

The above represents the objective function of minimized total discounted cost over four time periods for three towns of the construction and operation and maintenance (O+M) of new reservoirs, construction and O+M of new pipeline systems, O+M of existing wells, and O+M of currently existing pipelines.

Constraints

$$R_i \sum_{t=o} y_{it} - W_{it} \geq o \; ; \forall i=1,\ldots, I. \tag{4-2}$$

Current yield from reservoir i in time period t if implemented must be equal to or less than the capacity of the i^{th} proposed reservoir.

$$\sum_{t=o}^{T} y_{it} \leq 1 \; ; \forall i \tag{4-3}$$

This allows the i^{th} proposed reservoir to be constructed only once over four time periods.

$$Q_{zt} \leq P_z \; ; \forall z=1,\ldots, Z \; ; t=1,\ldots, T \tag{4-4}$$

This allows for the current yield from an existing well z in time period t to be equal to or less than the capacity of the z^{th} existing well.

$$W_{it} - \sum_{j=1}^{J} w_{ijt} = 0 \; ; \forall i = 1,\ldots,I \; ; t=1,\ldots, T \tag{4-5}$$

Current yield from reservoir i in time period t must equal the volume of water flowing from proposed reservoir i to all town j's in time period t.

$$Q_{zt} - \sum_{j=1}^{J} [q_{zjt} + u_{zjt}] = o \;;\; \forall\ z=1,\ldots,Z \;;\; t=1,\ldots, T \qquad (4\text{-}6)$$

Current yield from existing well z in time period t must equal the volume of water flowing from existing well z to all town j's through existing and new pipelines in time period t.

$$s_t^j \leq \sum_{i=1}^{I} w_{ijt} + \sum_{z=1}^{z} [u_{zjt} + q_{zjt}] \;;\; \forall\ j,\ t \qquad (4\text{-}7)$$

The volume of water flowing from existing sources and new reservoirs for period t is at least equal to the water demand requirements for each town in period t.

$$\sum_{t=o}^{T} V_{ijt} \leq 1; \forall i,j \qquad (4\text{-}8)$$

Once a pipeline is constructed between a new reservoir and town during any of the four time periods, it cannot be constructed again.

$$\sum_{t=o}^{T} X_{zjt} \leq 1; \forall z,j,j\neq z \qquad (4\text{-}9)$$

Once a pipeline is constructed between an existing well and town during any of the four time periods, it cannot be constructed again.

$$P_z \sum_{t=o}^{T} X_{zjt} - u_{zjt} \geq 0; \forall z,j,j\neq z \qquad (4\text{-}10)$$

The volume of water flowing from existing well z to town j through new pipeline in time t cannot exceed the capacity of well z.

$$R_i \sum_{t=o}^{T} V_{ijt} - w_{ijt} \geq 0; \forall i,j \qquad (4\text{-}11)$$

The capacity of the i^{th} proposed reservoir must be greater than or equal to the volume of water flowing from proposed reservoir i to town j in time period t.

Contained below is an explanation of all of the above notation.

Notation

C_i : capital cost of the i^{th} proposed reservoir

r : discount rate

y_{it} : value of 1 or 0 for the i^{th} reservoir in time t

b_i : unit operation and maintenance cost of proposed reservoir i

W_{it} : current yield from reservoir i in time period t

d_{ij} : capital cost of the pipeline constructed from proposed reservoir i to town j

V_{ijt} : value of 1 or 0 for proposed pipeline from proposed reservoir i to town j in time t

e_{ij} : unit operation and maintenance cost of the proposed pipeline from proposed reservoir i to town j

w_{ijt} : volume of water flowing from proposed reservoir i to town j in time period t

g_z : operation and maintenance cost per unit of current yield from existing well z

Q_{zt} : current yield from existing well z in time period t

m_{zj} : capital cost of proposed pipeline from existing well z to town j

X_{zjt} : value of 1 or 0 for proposed pipeline from existing well z to town j in time t

k_{zj} : unit operation and maintenance cost for proposed pipeline from existing well z to town j

u_{zjt} : volume of water flowing from existing well z to town j through proposed pipeline in time t, where $z \neq j$

l_{zj} : unit operation and maintenance cost for existing pipeline from existing well z to town j

q_{zjt} : volume of water flowing from existing well z to town j through existing pipeline in time t, where $z=j$

R_i : capacity of i^{th} proposed reservoir

P_z : capacity of z^{th} existing well

s_t^j : water demand requirement for town j in time t

S : discounted total cost representing objective function value

i : refers to proposed reservoirs

t : refers to a time period with t=0 denoting the years 1981-1990, t=1 designating the years 1991-2000, t-2 depicting 2001-2010, and t-3 designating the period 2011-2020

j : indexes towns with j=1, j=2, and j=3 representing Hudson, Merrimack, and Nashua, respectively

z : indexes wells with z=1, z=2, and z=3 depicting existing wells in Hudson, Merrimack, and Nashua, respectively

Empirical Results

Table 4-2 contains an overview of the initial optimal solution for four time periods. The minimum discounted total cost representing the objective function value is given. The projected water demand requirements for the individual towns for each time period are stated, along with the current yield from the existing individual wells for each period t. Special note is made when a new reservoir i is constructed and the designated period initially utilized. Also included is the current yield of reservoir i for time span t as well as any new pipelines that must be built.

The objective function value is $3,545,089.70 which represents the minimum discounted total cost of carrying out a distinct water planning strategy which satisfies all of the necessary constraints designated in equations 4-2 through 4-11. Interestingly, the existing well in Hudson does not enter the solution and thus suggests that it should not be utilized over the entire four time periods. In

Table 4-2. Optimal Solution for Merrimack River Basin Model*
(million gallons)

Period 1981-1990, t=0

Demand requirement (s_t^j)
for town j

$s_0^1 = 6012.$

$s_0^2 = 9059.$

$s_0^3 = 32357.$

Current yield (Q_{zt})
from well z

$Q_{10} = 0$

$Q_{20} = 9059.$

$Q_{30} = 38369.$

New reservoir (y_{it})
constructed

none

New pipeline (x_{zjt}) constructed
from existing well z to town j

x_{310}

Period 1991-2000, t=1

Demand requirement (s_t^j)
for town j

$s_1^1 = 10355.$

$s_1^2 = 15392.$

$s_1^3 = 36588.$

Current yield (Q_{zt})
from well z

$Q_{11} = 0$

$Q_{21} = 0$

$Q_{31} = 36588.$

New reservoir (y_{it})
constructed

y_{11}

Current yield (w_{it}) from
reservoir i

$w_{11} = 25747.$

New pipeline (v_{ijt}) from
reservoir i to town j

v_{111}

v_{121}

Period 2001-2010, t=2

Demand requirement (s_t^j)
for town j

$s_2^1 = 17845.$

$s_2^2 = 26156.$

$s_2^3 = 41373.$

Current yield (Q_{zt})
from well z

$Q_{12} = 0$

$Q_{22} = 0$

$Q_{32} = 41373.$

New reservoir (y_{it})
constructed

none

Current yield (w_{it}) from
reservoir i

$w_{12} = 44001.$

Period 2011-2020, t=3

Demand requirement (s_t^j)
for town j

$s_3^1 = 30744.$

$s_3^2 = 44439.$

$s_3^3 = 46782.$

Current yield (Q_{zt})
from well z

$Q_{13} = 0$

$Q_{23} = 16115.$

$Q_{33} = 50005.$

New reservoir (y_{it})
constructed

none

Current yield (w_{it}) from
reservoir i

$w_{13} = 55845.$

*Objective Function Value: $3,545,089.70

time period 0, new pipeline is constructed from the existing well in Nashua and provides water from Nashua to Hudson. In time period 1, a new reservoir is utilized and located in the town of Hudson. For this period, new pipelines link this reservoir to water transmission to Hudson and Merrimack and the existing well in Merrimack is not used. For time period 2, no additional reservoirs are required. In the final period 3, Merrimack is again drawing water from its own well, but also with water provided by the Hudson reservoir. Hudson is relying upon Nashua and its own reservoir for water provision and Nashua is provided water by its existing well.

Table 4-3 provides additional information inherent in the optimal solution. The volumes of water transmitted through various sources (q_{zjt}, u_{zjt}, and w_{ijt} values) are given with the interaction network between the three towns summarized for each of the time periods. It can be readily seen that Hudson has water transmitted from Nashua's well in period 0, from its own newly constructed reservoir for periods 1 and 2, and from both Nashua's well and the reservoir for the final period. Merrimack utilizes its own well in period 0, Hudson's reservoir during t=1 and t=2, and for t=3 its own well as well as the reservoir. Nashua is self-sufficient over all four periods relying exclusively on its own well. Of seven alternative reservoir sites, only one is actually constructed.

Because the objective function can only change in discrete jumps with respect to changes in the integer variable, penalties associated with integer variables should be interpreted carefully. These jumps or steps are not necessarily the same interval at different values of the variable. As a result, shadow prices cannot be given the usual interpretation. Therefore, computed shadow prices attached to our model are left out.

The model consists of 296 variables and 235 constraints. The number of integer variables is 136 with continuous variables equal to 160. The matrix density is 1.725 percent which means that of a matrix 296 by 235, 1.725 percent of the elements are nonzero.

Table 4-3. Volume of Water Transmitted from Town i to Town j Through Various Sources for Designated Time Periods (million gallons)

i \ j	1981 - 1990, t=0		
	Hudson (j=1)	Merrimack (j=2)	Nashua (j=3)
Hudson (i=1) well, z=1	0 (q_{110})	0 (u_{120})	0 (u_{130})
Merrimack (i=2) well, z=2	0 (u_{210})	9059. (q_{220})	0 (u_{230})
Nashua (i=3) well, z=3	6012. (u_{310})	0 (u_{320})	32357. (q_{330})

i \ j	1991 - 2000, t=1		
	Hudson (j=1)	Merrimack (j=2)	Nashua (j=3)
Hudson (i=1) well, z=1	0 (q_{111})	0 (u_{121})	0 (u_{131})
Merrimack (i=2) well, z=2	0 (u_{211})	0 (q_{221})	0 (u_{231})
Nashua (i=3) well, z=3	0 (u_{311})	0 (u_{321})	36588. (q_{331})
Reservoir 1 (Y_1) (located in Hudson)	10355. (w_{111})	15392. (w_{121})	0 (w_{131})

Table 4-3 (cont'd.)

i \ j	2001 - 2010, t=2 Hudson (j=1)	Merrimack (j=2)	Nashua (j=3)
Hudson (i=1) well, z=1	0 (q_{112})	0 (u_{122})	0 (u_{132})
Merrimack (i=2) well, z=2	0 (u_{212})	0 (q_{222})	0 (u_{232})
Nashua (i=3) well, z=3	0 (u_{312})	0 (u_{322})	41373. (q_{332})
Reservoir 1 (Y_1) (located in Hudson)	17845. (w_{112})	26156. (w_{122})	0 (w_{132})

i \ j	2011 - 2020, t=3 Hudson (j=1)	Merrimack (j=2)	Nashua (j=3)
Hudson (i=1) well, z=1	0 (q_{113})	0 (u_{123})	0 (u_{133})
Merrimack (i=2) well, z=2	0 (u_{213})	16115. (q_{223})	0 (q_{233})
Nashua (i=3) well, z=3	3223. (u_{313})	0 (u_{323})	46782. (q_{333})
Reservoir 1 (Y_1) (located in Hudson)	27521. (w_{113})	28324. (w_{123})	0 (w_{133})

CONCLUSIONS

Water-resource managers attempt to forecast future supply-demand imbalances by projecting levels of population and industrial activity, relating those levels to demands for water supply and quality, and comparing the projected demands with the capacity of the existing supply system. If demands exceed the capacity a deficit is forecast, and the manager is faced with selecting the appropriate strategy for eliminating this deficit.

The results of the hydrologic and economic analyses of this report can be integrated to provide a framework for this process of water-resource planning that meets the four requisites discussed in the Introduction:

1. Precise definition of the problem - Water-requirements, or demands, at a future period are determined by the relative magnitudes of a) the flow rates required for withdrawal (W); b) the flow rates required for instream uses such as recreation, habitat, navigation, and hydropower (Q_{min}); c) the rate at which withdrawn water is consumed (C); and d) the flow rate required to maintain water quality at acceptable levels (Q_q). When these relative magnitudes are determined, the water requirement is precisely defined (see eqns. 3-7).
2. Consideration of all potential alternative solutions - Table 3-1 provides a list of all potential strategies that might be considered as solutions to a forecast deficit. Some strategies increase the available supply, while others act to reduce demands. Given the precise nature of a deficit as determined in step 1, various alternatives can be identified as appropriate or inappropriate. For example, increasing treatment reduces Q_q, but has no effect on W, C, or Q_{min}; desalination may satisfy a forecast withdrawal rate, W, but may have little effect on Q_q; a flow-augmentation reservoir can provide water for withdrawal

and/or satisfy water-quality needs. Preliminary planning-level estimates of the quantitative effects of the alternatives of Table 3-1 on water quantity and quality can be made based on the material presented in Section 3 and Appendixes A-F.

3. Appropriate objectives and criteria - Choice of alternative strategies for alleviating a forecast water deficit is made by evaluating each alternative against specific criteria, which measure the extent to which the alternatives meet selected objectives. Benefits and costs represent positive and negative measures relative to criteria. If benefits and costs are measured in more than one way (e.g., dollars and one or more measures of environmental effect), the problem is multi-objective; if only one measure is used, with other effects considered as constraints, the problem is single objective. One common way of approaching water-resource problems is as a single-objective scheduling problem, as described in Section 4. This particular objective is to find an optimal solution for choosing and implementing the available water-supply alternatives that has the least present value of costs while satisfying the projected requirements.

4. Ability to analyze alternatives with request to criteria - To evaluate the benefits of a water-resource strategy, one must know the degree to which that strategy will alleviate the problem - i.e., how much water the alternative will provide. (This may be converted to dollars of benefits if the price of the water is known.) The discussions of Section 3 provide information for planning-level estimates of the effects on water quantity and quality of a large number of alternatives. This information is required for any evaluation scheme, and was specifically incorporated into the MIP model described in Section 4.

The hydrologic and economic objectives were mainly accomplished through the development of simulation and optimization models, respectively. Introduction of the latter's results into the former model provided for an integrated assessment of water-management strategy for a sub-basin area over a future planning horizon. Each model on its own provides useful information for a particular aspect of river basin management; but in combination, linkages are formulated which allow for a clearer, more realistic evaluation and overview.

REFERENCES

Aron, G., T. Rachford, J. Borelli, and W. Stottman, " A method for integrating surface and ground water use in humid regions", Penna. State U. Inst. for Rsch. on Land and Water Res., Rsch. Pub., 1974, 76.

Balas, E., "An additive algorithm for solving linear programs with zero-one variables", Operations Research, Vol. 13, 1965, 517-46.

Balinski, M.L., "Integer programming: methods, uses, computation", Management Science, Vol. 12, 1965, 253-313.

Beale, E.M.L., "On quadratic programming", Naval Research Logistics Quarterly, Vol. 6, 1959, 227-243.

Bravo, A., J.G. Gomez, et al., A Mixed Integer Programming Code, Report 7043, University of Chicago, September 1970.

Butcher, W.S., Y.Y. Haimes, and W.A. Hall, "Dynamic Programming for the Optimal Sequencing of Water Supply Projects", Water Resources Research, Vol. 5, No. 6, 1969, 1196-1204.

Cohon, J.L. and D.H. Marks, "A review and evaluation of multiobjective programming techniques", Water Resources Research, Vol. 11, No. 2, 1975, 208-220.

Cotton, J., "Availability of ground water in the Pemigewasset and Winnipesaukee River Basins, central NH", U.S. Geol. Surv., Concord, NH, Open-File Rept., Water-Res. Invest., 47-75, 1975.

Cotton, J., "Availability of ground water in the middle Merrimack River Basin, central and northern NH", U.S. Geol. Surv., Concord, NH, Open-File Rept., Water-Res. Invest., 76-39, 1976.

Cotton, J., "Availability of ground water in the lower Merrimack River Basin, southern NH", U.S. Geol. Surv., Concord, NH, Open-File Rept., Water-Res. Invest., 77-69, 1977.

Dantzig, G.B., Linear Programming and Extensions, Princeton, NJ, Princeton Univ. Press, 1963.

Dantzig, G.B., "On the significance of solving linear programming problems with some integer variables", Econometrica, Vol. 28, January 1960, 30-44.

Federer, C.A. and D. Lash, "BROOK: A hydrologic simulation model for eastern forests", U. of NH Water Res. Rsch. Ctr., Rsch. Rept. 19, 1978.

Glover, F., "Generalized cuts in diophantine programming", Management Science, Vol. 13, November 1966, 254-268.

Gomory, R.E. and W.J. Baumol, "Integer programming and pricing", Econometrica, Vol. 28, July 1960, 521-550.

Gomory, R.E., "Outline of an algorithm for integer solutions to linear programs", Bulletin of the American Mathematical Society, Vol. 64, September 1958, 275-278.

Gomory, R.E., An Algorithm for the Mixed Integer Problem, Rand Corp. Res. Mem., RM-2597, 1960.

Hadley, G., Linear Programming, Reading, Mass., Addison-Wesley Publ. Co., 1962.

Haimes, Y.Y. and W.S. Nainis, "Coordination of regional water resource supply and demand planning models", Water Resources Research, Vol. 10, No. 6, 1051-1059.

Hall, F.R., "Chloride in natural waters of New Hampshire", NH Agric. Expt. Sta. Bull. 504, 1975.

Hall, F.R., "Physical management problems of small aquifers", Paper presented at Am. Geophys. Union Annual Meeting, 1979.

Hawkins, R.H., "Effect of changes of streamflow regimen on reservoir yield", Water Res. Rsch., Vol. 5, 1969, 1115.

Heady, Earl O., N.S. Randhawa, and M.D. Skold, "Programming models of interregional efficiency and land use in agriculture", Activity Analysis in the Theory of Growth and Planning, London, Macmillan, 1967, 245-269.

Hoeh, R.S., "A study of the feasibility of social science research designed to identify and analyze social responses to precipitation management operations in New England", U. of NH, Dept. of Political Science, 1968.

Hornbeck, J.W. and C.A. Federer, "Hubbard brook experimental forest, NH", Municipal Watershed Mgmt. Symp. Proc., U.S. Forest Service Genl. Tech. Rept. NE-13, 1975, 58.

Jenkins, C.T., "Techniques for computing rate and volume of stream depletion by wells", Groundwater, Vol. 6, No. 2, 1968, 37.

Jenkins, C.T. and O.J. Taylor, "A special planning technique for stream-aquifer systems", U.S. Geol. Surv., Denver, CO, Open-File Rept., 1974, 74-242.

Knudsen, J. and D. Rosbjerg, "Optimal scheduling of water supply projects", Nordic Hydrology, Vol. 8, 1977, 171-192.

Kuiper, D. and R. Wechsler, "Conceptual aspects of water re-use", Water Rsch., Vol. 8, 1974, 529.

Loucks, D.P., "Conflict and choice: planning for multiple objectives", Economy Wide Models and Development Planning, ed. by C. Blitzer, P. Clark, and L. Taylor, New York, Oxford U. Press, 1975.

Lynch, J.A., W.E. Sopper, E.S. Corbett and D.W. Aurand, "Penn. State Experimental Watersheds", Municipal Watershed Mgmt. Symp. Proc., U.S. Forest Service Genl. Tech, Rept. NE-13, 1975, 32.

McCarl, B., et al., MIPZ1 - Documentation on a Zero-One Mixed Integer Programming Code, Purdue Agricultural Experiment Station Bulletin No. 24, October 1973.

Markowitz, H.M. and A.S. Manne, "On the solution of discrete programming problems", Econometrica, Vol. 25, 1957, 84-110.

Morin, T.L., "Pathology of a dynamic programming sequencing algorithm", Water Resources Research, Vol. 9, No. 5, 1973, 1178-1185.

Morin, T.L. and A.M. Esogbue, "A useful theorem in the dynamic programming solution of sequencing and scheduling problems occurring in capital expenditure planning", Water Resources Research, Vol. 10, No. 1, 1974, 49-50.

Nashua Regional Planning Committee, 1977.

New England Division, U.S. Army Corps of Engineers, Merrimack River Basin Water-Supply Study, Waltham, MA, New Eng. Div. U.S. Army Corps of Engrs., 1977.

New England River Basins Commission, Summary Report on Severe Resource Problems and Recommendations for their Solution, Boston, MA, New Eng. River Basins Comm., 1977.

New England River Basins Commission, Merrimack River Basin Overview, Boston, MA, New Eng. River Basins Comm., 1978.

New Hampshire Water Supply and Pollution Control Commission, "Merrimack River Basin water quality management plan", NH Water Supply and Pollution Control Comm. Staff Report 90, 1978.

Patric, J.H., "Deforestation effects on soil moisture, streamflow and water balance in the central Appalachians", U.S. Forest Service Rsch. Paper NE-259, 1973.

Pierce, R.S., J.W. Hornbeck, G.E. Likens, and F.H. Bormann, "Effect of elimination of vegetation on stream water quantity and quality", IASH-UNESCO Symp. on Rsch. on Representative and Experimental Basins, Wellington, N.Z., Proc., 1970.

Ryder, G., ed., The Outlook for Water in New Hampshire, unpub. ms., U. of NH Inst. Natl. and Env. Res., 1977.

Sharpe, W.E., "Municipal water conservation alternatives", Water Res. Bull., Vol. 14, 1978, 1080.

Sopper, W.E. and L.A.V. Hiemstra, "Effects of simulated cloud seeding on streamflow of selected watersheds in Pennsylvania", Water Res. Bull., Vol. 6, 1970.

Vajda, S., Mathematical Programming, Reading, Mass., Addison-Wesley Publ. Co., 1961.

Wollman, N. and G.W. Bonem, The Outlook for Water, Baltimore, Md., Johns Hopkins Press for Res. for the Future, 1971.

APPENDIX A

Application of Water Supply-Demand Framework to a Merrimack Basin Community

GENERAL

The town of Bow, New Hampshire, is located on the Merrimack River just south of Concord. Its 1980 population is about 3,990 and its projected year-2000 population is 5,790. The town presently has neither a public water supply nor a sewage-treatment plant. We will compute water requirements for the Merrimack River at Bow for the year 2,000 population, using the framework equations developed earlier and the following values, largely taken from various planning studies (principally New Hampshire Water Supply and Pollution Control Commission, 1978). Computations are most conveniently done with flow rates in liters per second ℓ/s). Phosphorus is the only water-quality constituent considered (see Appendix B).

Parameter Values

$P = 5{,}790$ persons

$Y_{95} = 28{,}800$ ℓ/s (1017 ft^3/s)

$c_1 = .010$ mg/ℓ (present values are about 0.03 mg/ℓ, but we assume they will be improved by 2000).

$c^+ = .015$ mg/ℓ

$a_p = .0048$ ℓ/s person (110 gal/day person)

b_p (phosphorus) = .05 mg/s person (.009 lb/day person)

$k_c = .25$

Water Supply Requirements

$$U = a_p P = 27.8 \text{ ℓ/s } (.98\ ft^3/s;\ 0.63 \text{ million gal/day})$$

Assume no recycling ($k_r = 0$)

The required withdrawal is

$$W = (1 - k_r) \quad U = 27.8 \ \ell/s$$

Since W is only 1/1000 of Y_{95}, the Merrimack River would provide an ample quantity of water.

Minimum Flow for Aesthetics and Habitat, etc.

A common rule-of-thumb to compute flows required for support of fisheries in New England is 0.2 ft^3/s per square mile of drainage area. The drainage area of the Merrimack above Bow is about 2,500 mi^2, so Q_{min} = 500 ft^3/s = 14,200 ℓ/s. The present Y_{95} is about twice this value, so assuming there are no higher minimum flows needed for other purposes, this requirement is also satisfied.

Water-Quality Requirements

The rate of phosphorus contribution to the assumed municipal supply is

$$A = b_p P = 289.5 \text{ mg/s } (55.1 \text{ lb/day})$$

For initial computations, we assume that the water treatment process does not remove phosphorus, so $t_f = 0$. We now compute c_j, c_d, c_3, and Q_q using the appropriate equations and parameter values for four levels of treatment: t_g = 0, 0.10, 0.35, 0.98, corresponding to no, primary, secondary, and tertiary treatment.

$$c_j = \frac{A + (1 - t_f)c_1 w}{[(1 - k_c) - (1 + t_g + t_f - t_g t_f)k_r]U}$$

$$G = t_g c_j (1 - k_c)U$$

$$c_d = c_j - \frac{G}{(1 - k_c)U}$$

$$c_3 = \frac{c_1(Y_{95} - W) + c_d(1 - k_r - k_c)U}{Y_{95} - k_c U}$$

$$Q_q = \frac{[k_c c^+ - c_1(1 - k_r) + c_d(1 - k_r - k_c)]U}{c^+ - c_1}$$

The results are summarized in Table A-1.

Table A-1

t_g	0	.10	.35	.98
c_j (mg/ℓ)	13.9	13.9	13.9	13.9
G (mg/s)	0	29.0	101.4	284.0
c_d (mg/ℓ)	13.9	12.5	9.0	0.3
c_3 (mg/ℓ)	.020	.019	.017	.010
Q_q (ℓ/s)	57,900	52,100	37,500	1,200

Total Requirements

The data in Table A-1 show that for all levels of treatment less than tertiary,

$$Q_q > (Q_{min} + W) > Q_{min}$$

so the flow requirement is Q_q + C. With tertiary treatment,

$$(Q_{min} + W) > Q_{min} > Q_q$$

so the required flow is Q_{min} + W. These results are summarized in Table A-2.

Table A-2

t_g	0	.10	.35	.98
W (ℓ/s)	27.8	27.8	27.8	27.8
Q_{min}(ℓ/s)	14,200	14,200	14,200	14,200
Q_m + W(ℓ/s)	14,200	14,200	14,200	14,200
Q_q (ℓ/s)	57,900	52,100	37,500	1,200
Required Q (ℓ/s)	57,900	52,100	37,500	14,200
Y_{95} (ℓ/s)	28,800	28,800	28,800	28,800
Deficit (ℓ/s)	29,100	23,300	8,700	--

Table A-2 shows that with any level of treatment less than tertiary, there will be a water deficit in the Merrimack River at Bow (under the conditions assumed here) - i.e., the phosphorus concentration will exceed the water quality standard of .015 mg/ℓ. There are essentially four options for avoiding the deficit:

1. Build upstream storage reservoirs to increase Y_{95} to 57,900 ℓ/s;
2. Build a primary treatment plant and upstream reservoirs to increase Y_{95} to 52,100 ℓ/s;
3. Build a secondary treatment plant and upstream reservoirs to increase Y_{95} to 37,500 ℓ/s;
4. Build a tertiary treatment plant.

Each of these alternatives would have associated costs, including probably significant environmental and social costs for the reservoirs, which would have to be considered by decision-makers. It is very likely that the fourth alternative would turn out to be most attractive from the viewpoint of Bow alone. However, if sites are available, reservoirs would increase Y_{95} at all points downstream from where they were constructed, and therefore would help to alleviate water deficits in many stream reaches, so their costs might be spread out over many towns. The effects of reservoirs on downstream Y_{95} values is addressed Appendix C.

APPENDIX B

New Hampshire Water Resources Research Center Project A-050-NH

An Integrated Assessment of Management in Water Quantity and Quality in the Merrimack River Basin in New Hampshire

Working Paper #1

Selection of Water-Quality Parameters and Criteria

by

Paul G. Sutton

1 October 1978

The objective of this project is to provide the necessary information to assess the present and future water quantity and quality problems within the New Hampshire portion of the Merrimack River basin, and evaluate the alternative solutions of the encountered problems. Evaluation of the solutions will include a "least-cost" determination of the most effective method for solving the forecasted problems.

Within the operating model of this study, the basis of which is derived from the model developed by Wollman and Bonem (1970), one decision point will involve water quality. The water-quality decision will be predicated on the sub-basins water's compliance with the water-quality criteria established for the study.

The purpose of this working paper is to establish these water-quality criteria for use in making the water-quality decisions within the model. The criteria established will function within the operating model at the water-quality decision point, and should offer a reasonable evaluation of the water-quality problems within the Merrimack River Basin (M.R.B.).

Traditionally water-quality criteria are established according to the proposed use of the resource. The National Water Quality Standards Program initiated by the Water Quality Act of 1965 has recommended that water-quality standards be comprised of use designations for each water body, and water-quality criteria to support the designated use (EPA 1973).

Generally stated, the use classifications are; public water supply: industrial, municipal, agricultural, recreational, and protection and propagation of fish and wildlife. These use classifications coincide with the multiple uses of the water resources in the M.R.B.

According to the nationally established water-quality standards, recreation, and the protection and propagation of fish and wildlife require the highest quality waters (EPA 1973). The water-quality criteria to be used within the operating model of this study will follow the established criteria of several parameters within these use classifications.

The list of physical and chemical parameters incorporated into the criteria for recreational and the protection of fish and wildlife use classes is extensive. Inclusion of all these parameters into the working model is highly impractical. Wollman and Bonem in The Outlook for Water (1970) initially hoped to include several water-quality parameters into their model. As a result of the complexity generated by the number of factors initially studied, their study was finally restricted to dissolved oxygen as the only water-quality parameter. In the conclusion of their study Wollman and Bonem state (1970, p. 7), "Even the limited number of factors explored in this study presented a large array of possible solutions."

Wollman and Bonem's study had a national scope, consequently, detailing of their approach would have required large amounts of data and time. The M.R.B. is the object of the present study, thus, the water-quality criteria will focus on the water-quality problems within this river basin.

In an overview of the M.R.B., the New England River Basin Commission (Turner 1978) has identified industrial and municipal waste water and combined storm and sanitary sewers as the most significant sources of water-quality degradation within the basin. The estimated point-source discharges within the river basin are: 7.0 mg/l industrial effluent and 38.3 mg/l municipal waste water (Turner 1978). Combined storm and sanitary sewers are the cause of water-quality degradation in the towns of Pittsfield, Concord, Manchester and Nashua, New Hampshire. (N. H. Water Supply Pollution Control Com., 1978)

Nonpoint source pollution (NPS), i.e., pollution from diffuse, non-specific sources, within the M.R.B. has been "masked" by the aforementioned point sources. However, the N. H. Water Supply Pollution Control Commission and New England River Basin Commission both feel that NPS is suspect in the eutrophication of some lakes and ponds within the M.R.B. (N. H. WSPCC 1978, Turner 1978). These agencies along with EPA feel that as point sources of pollution are being corrected, i.e., improved industrial effluent treatment, construction and operation of municipal wash-water treatment plants, and separation or treatment of combined sewer overflows, NPS pollution will become evident as a source of water-quality degradation nationally and within the M.R.B (Mayo 1975).

The effluent content from some of the point sources can be summarized as follows:

Industrial Effluent Content for Apparel, Food and Materials Industry (Ciaccio 1971)

$\bar{x}$ BOD	-	100 -5,000 mg/l
$\bar{x}$ COD	-	80 -10,000 mg/l

Municipal Waste Water National Mean Concentrations mg/l (Ciaccio 1971)

	BOD	COD	TS	SS	TP	PO_4
Raw Sewage	147	288	453	145	-	6.6
Primary and Secondary Treatment	15-20	50-70	-	15-30	10-15	-
Tertiary Treatment	<10	<40	-	<10	-	-

Combined Sewer Overflows, Range of Results from Nine Cities in U.S. in mg/1 (Kothandaraman 1972)	
BOD	31-700 mg/1
COD	59-2000 mg/1
SS	30-2500 mg/1
TP	0.8-34.0 mg/1

Storm Water Runoff Ranges of Results from 5 Cities in U.S. (Kothandaraman 1972)	
BOD	1-283 mg/1
COD	20-1514 mg/1
SS	5-11,280 mg/1
TP	0.0-9.4 mg/1

TS = Total Solids, SS = Suspended Solids, TP = Total Phosphorus

The effects of NPS pollution cannot be easily identified due to the nature of their diffuse sources. The New England River Basin Commission cites the 208 and 303(3) planning programs as having identified NPS pollution sources within the M.B.R. as subsurface disposal systems, landfill leachate, soil erosion from construction sites, and road salting (Turner 1978). These sources of NPS pollution and additional sources were discussed in a survey conducted by N. H. Water Supply Pollution Control Commission (Elkind 1977). The survey sample included conservation commissioners, planning agencies, town planning boards and developers throughout New Hampshire. A list of the NPS pollution sources cited in the survey, the frequency of citing, and the pollutants expected to be present from these sources are listed below:

Perceived NPS Pollution in N. H. (Elkind 1977)

Contributing Source	Frequency of Mention	Pollutants from Source
Dumps and Landfills	MOD-High	Nutrients (N & P)* chloride, Metals, BOD, COD
Septic (tank-sludge) Disposal	MOD-High	Nutrients (N & P)*
Site Development	MOD-High	Suspended Sediment Nutrients (P)
Subsurface Sewer Disposal	MOD-High	Nutrients (N)

*N refers to nigrogen, P refers to phosphorus

Perceived NPS Pollution in N. H. (Elkind 1977) continued

Contributing Source	Frequency of Mention	Pollutants from Source
Highway Salting	MOD-High	Chloride, Metals?
Boat Discharge	MOD	Raw Waste, Oil and Gas
Filling & Dredging of wetlands	LOW-MOD	Nutrients (P) Metals
Highway Construction	LOW-MOD	Suspended Sediment Nutrients (P), Metals
Silviculture	LOW-MOD	Suspended Sediment, Nutrients

The water-quality criteria to be established for use in this study must function to 1) assure a water-quality level that will allow the intended uses; 2) adequately assess the present and future water-quality problems; and 3) remain applicable in light of the parsimony required in a Wollman and Bonem type model.

To best meet these ends, the water-quality parameters to be considered for use in the operating model will include dissolved oxygen, dissolved inorganic phosphorus, and sediment as sediment yield per sub-basin.

Dissolved Oxygen -

In order to sustain a healthy aquatic biota dissolved oxygen (DO) requirements will be established at 5 mg/l for all environmental extremes, i.e., low flow, and seasonal temperatures. A DO concentration of 5 mg/l is the lower limit acceptable in order to support the desirable fisheries. EPA recommends that in very extreme environmental situations that DO levels should not dip below 4 mg/l for a period longer than 24 hours (EPA 1973). Sustained DO levels below 4 mg/l will incur subacute and chronic damage to most fisheries and reduce their productivity (Durodoroff 1970).

DO concentrations of 5 mg/l may not be adequate to support some of the fisheries that have been mentioned for future production such as Atlantic salmon (Turner 1973). A DO concentration of 6.4 mg/l at 36°C is recommended to sustain spawning salmonid fishes (EPA 1973). The N. H. Water Supply Pollution Control Commission recommends a DO concentration of 6 mg/l or 75% of saturation for their high quality waters. These recommended DO concentrations seem unrealistic within the M.R.B. due to the present DO stresses and the cost intensity that would be required to meet these goals. A DO concentration of 5 mg/l is a good compromise given the desired and required resource uses. However, model runs will be made using both 5 mg/l and 6 mg/l dissolved oxygen, in order to most accurately respond to the situation within the M.R.B. in New Hampshire.

Phosphorus -

The excessive enrichment and resultant eutrophication of surface waters within the M.R.B. must be addressed as a cause of water-quality degradation. Advanced eutrophication will greatly reduce the utilization of the water resources within the M.R.B.

Phosphorus in the dissolved inorganic state (PO_4) will be included in the operating model as one of the water-quality parameters. The concentration of dissolved inorganic phosphorus will function as the index of enrichment within the river basin.

It is generally felt that phosphorus is the key element required by fresh-water algae, and the cause of eutrophication. Phosphorus is generally present in the least amount relative to need by fresh-water algae, therefore, an increase in phosphorus allows use of nitrogen and carbon, already present (nitrogen and carbon are available from the atmosphere), for algae growth (Shapiro 1970). It has been observed by Vollenweider (1975) that nitrogen may replace phosphorus as the limiting nutrient only in highly eutrophied lakes.

EPA has established dissolved inorganic phosphorus concentrations not to exceed 0.10 mg/l for flowing waters and 0.05 mg/l for flowing waters entering lakes and ponds (EPA 1972).

Lee(1970), while conducting a review of the literature, cites papers by Soyer and Vollenweider which state that phosphorus concentrations in lakes and ponds should not exceed 0.01 mg/l or nuisance algae blooms may occur.

This study is concerned with both flowing and standing waters of the M.R.B. The enrichment of standing waters is far more serious a problem than that of flowing waters, due to the aging and possible "death" of these waters from eutrophication. In flowing waters the level of enrichment has less long-lasting impact. Enrichment of flowing waters must be continuous and of higher concentrations to cause serious eutrophication problems (Hynes 1969).

In light of these varying critical concentrations for surface waters, the phosphorus parameters for this study will include dissolved inorganic phosphorus concentrations, 0.10 mg/l for flowing waters and 0.01 mg/l for lakes and ponds.

Sediment -

Sedimentation degrades the water quality by muddying the water and depositing sediment in large quantities such as sand bars. Sediment also disturbs the transmission of light through the water column thereby reducing the ability of aquatic plant life to produce oxygen. This consequent reduction in photosynthesis causes a decrease in the dissolved oxygen concentration within the water (Tuthill 1967). Increases in suspended solids also cause a marked decrease in the macroinvertebrate density and the standing crop of fish (Gammon,1970).

Sediment reaches receiving waters from industrial and municipal waste water, particles flushed in urban runoff, and erosion from the land surface. Sediment and sediment transport have been identified by several researchers as the major NPS pollutant from the land surface and as carriers of other pollutants. (Office of Air & Water Programs EPA 1973, Donigian 1976, McElroy 1976.)

A study of sediment erosion from varying land-use types, conducted in Maryland by Yorke et al (1978), revealed cropland, urban areas, and construction sites such as site development, as sources of excessive sediment loading to receiving waters.

Levels of suspended sediment in surface waters vary extremely with flow. The highest level of suspended sediment concentration occurs at times of greatest runoff. Sediment flow has been characterized as exhibiting extreme degrees of unsteady, non-uniform flow (Linsley et al 1958). As a result of the variability in sediment concentrations due to flow, it is not presently feasible to include in-stream suspended sediment concentration as a water-quality parameter within a steady state model.

A potential alternative to in-stream suspended sediment concentration is intra-basin erosion potential. A modified form of the "Universal Soil Loss Equation" such as that developed by McElroy et al (1976) could possibly serve these ends. The functions developed by McElroy et al, enable a non-flow related calculation of sediment loading from surface erosion in tons per year. These functions and others will be the subject of further study in order to determine their applicability within the scope of the project model.

Summary -

The present water-quality problems in the M.R.B. have been identified as industrial and municipal waste water and the outfalls of combined storm and sanitary sewers. The predicted water-quality problem, given improved industrial and municipal waste-water management and separation of combined sewers, is the probable impact of NPS pollution.

In the assessment of these problems, the operating model of this study will include the following water-quality parameters:

Dissolved Oxygen - To be not less than 5 mg/l for all environmental extremes.

Dissolved Inorganic Phosphorus	- In flowing waters not more than 0.10 mg/l. In lakes, ponds and impoundments not more than 0.01 mg/l.
Sediment	- Sediment will be the subject of further investigation. A model-compatible method for determining the extent of areal erosion and its impact on water-quality will be developed.

References Cited

Ciaccio, L.L., (1971), Water and Water Pollution Handbook, Vol. 1, Marcel Dekker Inc., New York, N.Y.

Donigian, A.S. and N.H. Crawford, (1976), Modeling Nonpoint Pollution from the Land Surface, Environmental Research Series, EPA Report No. EPA-600/3-76-083.

Durodoroff, P. and Shimway, (1970), DO Requirements of Fresh Water Fish, Food and Agricultural Organization of the U.S., F.A.D. Fisheries Technical Paper No. 86-261.

Elkind, Fred A., (1977), The Perceived Signifigance of Nonpoint Sources of Pollution in New Hampshire, N.H. Water Supply Pollution Control Commission.

U.S. Environmental Protection Agency (EPA), (1973), Proposal Criteria for Water Quality, Vol. 1.

U.S. Environmental Protection Agency (EPA), (1972), Water Quality Standards Criteria Digest, A Compilation of Federal and State Criteria On - Phosphates.

Gammon, J.R., (1970), The Effect of Inorganic Sediment on Stream Biota, Water Quality Office EPA, EPA Report No. 18050 DWC, December, 1970.

Hynes, H.B., (1969), The Enrichment of Streams, In: Eutrophication - Causes, Consequences, Correctives, National Academy of Science.

Kothandaraman, V., (1972), Water Quality Characteristics of Storm Sewer Discharge and Cobined Sewer Overflows, Illinois State Water Survey, Dept. of Registration and Education, Circular 109.

Leȩ F.G., (1970), Eutrophication, Eutrophication Information Program, Univ. of Wisconsin Water Resource Center, Occasional Paper No. 2.

Linsley, Ray, K., Jr., Max A. Kohler, Joseph L. H. Pauthus, (1958), Hydrology for Engineers, Magraw Hill Book Co., New York, N. Y.

Mayo, F.T., (1975), A Welcoming Address, In: Nonpoint Source Pollution Seminar, Section 108(a), Demonstration Projects Process Reports, EPA, EPA Report No. EPA-905/9-75-007.

McElroy, A.D., S.Y. Chiu, J.W. Nebgen, A. Aleti, and F.W. Bennett, (1976), Loading Functions for Assessment of Water Pollution from Nonpoint Sources, Environmental Protection Technology Series, EPA Report No. EPA-600/2-76-151.

New Hampshire Water Supply and Pollution Control Commission, (NHWS and PCC), (1978), Summary - Merrimack River Basin Water Quality Management Plan. Staff Report No. 90a.

Office of Air and Water Programs, (1973), Methods for Identifying and Evaluating the Nature and Extent of Nonpoint Source Pollutions. EPA, EPA-430/9-7-014.

Soyer, L.N., (1966), Basic Concepts of Eutrophication, Journal of Water Pollution Control Federation, Vol. 38, pages 737-744.

Shapiro, J., (1970), A Statement on Phosphorus, Journal of Water Pollution Control Federation, Vol. 42, page 772.

Tuthill, D.F., D. Sobers, and J. Marshall, (1967), The Effect of Sediment Pollution on Water Recreation in the Upper Potomac Estuary, Univ. of Maryland, Dept. of Agricultural Economics, Misc. Pub. 653.

Turner, G.V., J.E. Dillon, T. Glidden, J.G. Cooke, and M. Lines, (1978), First Draft Merrimack River Basin Overview, New England River Basin Commission.

Vollenweider, R. A. (1975) Input/Output Models, with Special References to Phosphorus Loading Concept in Limnology. Schweizerische Zeilschrift Fur Hydrologie, Vol 37.

Vollenweider, R. A. (1969) A Manual on Methods for Measuring Primary Production in Aquatic Environments, International Biological Program, Handbook No. 12.

Yorke, Thomas, and William Hub, (1978), Effects of Urbanization on Stream Flow and Sediment Transport in the Rock Creek and Anacostia River Basin, Montgomery County, Maryland, 1962-1974. U. S. Geological Survey Professional Paper 1003. U. S. Government Printing Office, Washington, D. C.

June 1980

APPENDIX C

PLANNING-LEVEL ESTIMATES OF THE VALUE OF SURFACE-STORAGE IN NEW HAMPSHIRE

by

S. Lawrence Dingman[1)]

INTRODUCTION

The classic water-resource problem is that facing a political entity for which projected water-use exceeds the safe yield of the existing supply system. The entity must choose among several alternatives for increasing safe yield. Before that choice can be made, however, another decision is required: what basis should be used for choosing among the alternatives? The standard approach of theoretical economics is to cast the problem as a choice among the possible sequences in which the alternatives can be implemented, and to select the sequence with the least present value of cost [1]. However, there are reasons for believing that minimization of present value of cost is not the criterion that would be used by a real water-resource manager. A manager is aware of uncertainties about the future - demand projections will not be fulfilled, new sources may become apparent, old sources may become unavailable, or the economics of the alternatives may change. Just as today's manager is not bound by sequencing decisions made in the past, we cannot expect the future to be bound by today's decisions about how it should invest its money.

Thus, it is worthwhile to consider what an actual manager might use as choice criteria. As implied above, I believe (s)he would not make a sequencing decision, but only a decision about which of the alternatives to implement now. I suggest that this choice would be made largely on the basis of four considerations, and is thus a multi-objective, not a single objective, problem. One major factor would be total cost, and the objective would, of course, be to minimize this. It's also likely that one would want to minimize some measure of unit cost. The cost per volume of storage is often used for this measure, in particular in the Merrimack River Basin of New Hampshire and Massachusetts [10]. However, it appears much more reasonable to use cost per unit of yield increase as a criterion of choice. In addition, the manager would probably want the new system to exceed the projected use for at least some minimal period of time. On the other hand, (s)he might not wish

1) Associate Professor, Institute of Natural and Environmental Resources, University of New Hampshire, Durham, New Hampshire 03824

to invest money to provide for the future beyond some period of years, largely because of the uncertainties mentioned above. The final objective would be to minimize adverse environmental and social impacts.

Thus, the value of a water-supply reservoir, V, could be expressed as a vector, V, where

$$V = V(C, C/\Delta Y, T, I) \qquad (1)$$

and C is its total cost, ΔY is the increase in yield it provides, T is the period of time until the next yield increment is projected to be required, and I represents the environmental and social impacts (itself a vector). In the actual decision-making process, one might wish to recast the last two objectives as constraints. Note that the constraint for T would be expressed as $T_{min} < T < T_{max}$, where T_{min} and T_{max} are selected by the decision-maker. Then all alternative sites that satisfy those constraints could be plotted on a graph of C vs $C/\Delta Y$ so that non-inferior alternatives could be identified and trade-offs between those two objectives evaluated.

Given the multi-objective framework described above, the major question addressed herein is the estimate of the increase in yield, ΔY, that can be provided by surface-water reservoirs in the New Hampshire portion of the Merrimack River Basin. This question has two parts: 1) the yield that can be provided at the reservoir site, i.e. when the reservoir is connected to the use point via an aqueduct; and 2) the yield that can be provided at downstream locations due to the regulatory effect of reservoirs. The second of these questions is of critical importance for evaluating reservoirs with regard to instream uses of water, as well as downstream withdrawal uses.

Definition of Yield

For purposes of this paper, yield is defined as the mean daily streamflow that is exceeded on 95% of the days in a stream reach of interest, and is designated as Y_{95}. (By this definition, any limitations in availability of water imposed by the capacity of a system for distributing or treating withdrawn water are not considered.) Although this definition does not explicitly include any considerations of duration of shortage, Fig. 1 shows that with one anomaly, there is a very close relationship between $_7Q^*_{10}$ and Y^*_{95} for both regulated and unregulated streams in the Merrimack Basin, where $_7Q_{10}$ is the ten-year, seven-day low flow and the asterisk here and subsequently indicates division by the long-term mean flow.

Effects of Storage on Yield

Relations at Reservoir Sites - A point on an unregulated stream is characterized by its natural Q_{95}. In New Hampshire, this can be well estimated if the basin area and mean basin elevation above the point are known [2,3]. Thus, in the absence of regulation, $Y_{95} = Q_{95}$. At the reservoir site, Y_{95} is determined from the appropriate storage-yield curve. Table 1 summarizes methods that have been used to develop these curves in or near the Merrimack Basin. As shown in Fig. 2, these plot sufficiently close together to permit development of a composite relation that can be used for planning purposes:

$$Y^*_{95} = 1 - \frac{74.1}{78 + S^*} \tag{2}$$

where S is storage volume of the reservoir and S^* is expressed in days. Note that the average value of Q^*_{95} for the Merrimack Basin is .05.

Evaluation of a reservoir in terms of the increase in yield that could be transmitted via aqueduct to a use site involves estimation of Y_{95} only, as this is the increase in supply made available to users. For planning purposes, this can be estimated directly by eqn. 2. However, as discussed in the next section, an additional step is required when evaluating the downstream effects of reservoir regulation.

Downstream Effects of Storage - Regulation is defined as the long-term average rate of release from a reservoir. It is also represented by the area between the natural flow-duration curve for the site and the regulated curve, measured on one or the other side of the point of intersection of the two curves (Fig. 3). Fig. 3 also illustrates the relation between regulation and Δ_{95}, which is defined as

$$\Delta_{95} \equiv Y_{95} - Q_{95} \tag{3}$$

Langbein [4] made an empirical study of the relation between regulation and storage for reservoirs throughout the United States. He found that this relation could be well approximated by

$$R^* = \frac{1}{1 + (913/S^*)^{.625}} \tag{4}$$

where R^* is regulation determined from observations of fluctuations of reservoir contents. U.S. Geological Survey records permit determination of R^* for 10 non-flood-control reservoirs in New Hampshire, of which six are in the Merrimack basin. These appear to conform quite well to Langbein's curve. Thus we will henceforth assume that R^* can be estimated by eqn. 4.

Assuming reasonably consistent operating policies, one would expect a consistent relation between Δ^*_{95} and R^* at a reservoir site. This expectation was tested by calculating $\Delta^*_{95} = Y^*_{95} - Q^*_{95}$ for 11 non-flood control reservoirs in the Merrimack Basin. Y^*_{95} was calculated from eqn. 2, with $\bar{Q}$ and Q_{95} estimated by the method of Dingman [2] and S taken from U.S. Geological Survey published records. The upper and lower extremes of the relation are determined by reasoning that $\Delta^*_{95} = 0$ when $R^* = 0$ and, since the natural Q^*_{95} averages about .05 in the region, complete regulation ($R^* = 1$)

Table 1. Basis of Storage-Yield Relations for New England (See Fig. 2)

Curve	Safe-Yield Definition
NEWWA (1945) [6]	Not specified; however, based on 1911-1918 drought and approximately equal to 97% assurance (Russell et al., 1970). Curve shown is for water area = 0% of basin area. Based largely on Massachusetts.
Löf and Harison (1966) [5]	Curve is for 95% assurance, using aggregate stream-flow statistics for New England.
U.S. Army Corps of Engineers (1972) [9]	Curve based on composite synthetic stream flows at New Hampshire streams, with shortage index = .01 (see definition of index in report).
Riggs and Hardison (1973) [7]	Curve is for 95% assurance and is a composite of individual curves developed by this method applied to streams in New Hampshire portion of the Merrimack River Basin.

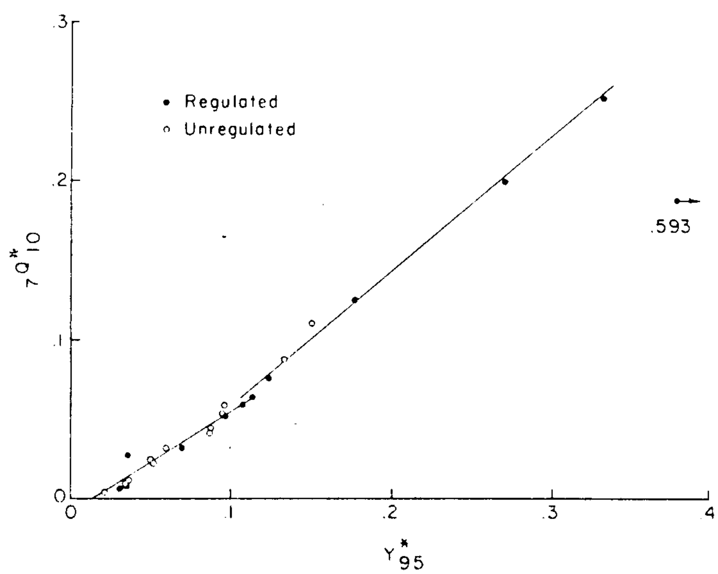

Fig. 1. Relationship between $_7Q^*_{10}$ and Y^*_{95} for regulated and unregulated streams in the Merrimack Basin, New Hampshire

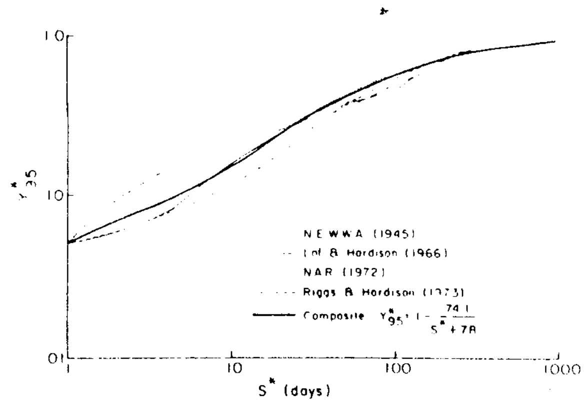

Fig. 2. Storage-yield curves applicable to the Merrimack Basin, New Hampshire

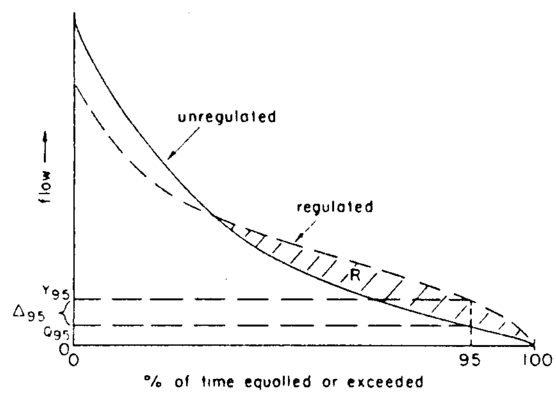

Fig. 3. Definition sketch of flow duration curves relating R, Y_{95}, Δ_{95}, and Q_{95}

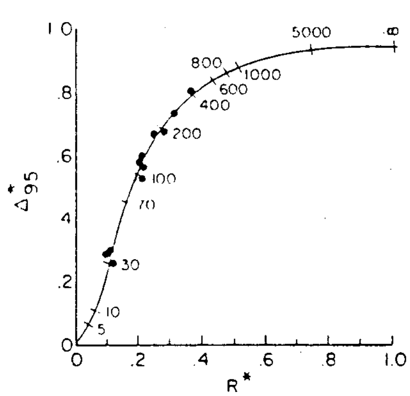

Fig. 4. "Theoretical" (curve) and actual (dots) relationship between Δ^*_{95} and R* at reservoir sites in the Merrimack Basin, New Hampshire

would achieve $\Delta^*_{95} = .95$. Fig. 4 shows the "general" curve relating Δ^*_{95} and R^*, where Δ^*_{95} is assumed equal to $Y^*_{95} - .05$, Y^*_{95} is calculated by eqn. 2, and R^* is calculated by eqn. 4. The Merrimack reservoirs fit the "theoretical" relationship quite well.

The simplest hypothesis for relating Δ_{95} to upstream reservoir storage is that

$$\Delta_{95} = f(\Sigma R) \tag{5}$$

where ΣR represents the sum of the regulation contributed by all upstream reservoirs that are not in series with another reservoir. Where there are reservoirs in series, ΣR includes only the regulation from the downstream-most reservoir.

This hypothesis can be examined using data for the reservoirs and gaging stations shown in Fig. 5. Table 2 shows the computations. Fig. 6 is a plot of Δ^*_{95} vs. $(\Sigma R)^*$, which are the actual $\Delta 95$ and ΣR values divided by the mean flow at the gaging station. Although one might be tempted to fit the data with a straight-line regression, I have shown in Fig. 6 the relationship developed earlier between R^* and Δ^*_{95} at reservoir sites (Fig. 4). Interestingly, the data for $(\Sigma R)^*$ and Δ^*_{95} at downstream sites appear to be related in the same way. Since there is conceptual as well as empirical support for this relationship, I suggest its use in estimating the downstream effects of non-flood control storage on streamflows in the New Hampshire portion of the Merrimack Basin.

The scatter of points from the "theoretical" relationship in Fig. 6 is due to uncertainties in estimating the unregulated Q_{95} at the gage sites and to deviations from eqns. 2 and 4 due to varying operating policies and local hydrologic conditions. It is unfortunate that more data are not available to provide greater confidence in the relation between regulation and downstream flows. Uncertainty is particularly high at small values of $(\Sigma R)^*$, and this is where many practical cases fall.

Summary and Conclusions

In the multi-objective context described earlier, eqn. 2 can be used to estimate ΔY for computing the unit costs (C/Y_{95}) and planning horizon (T) of reservoir sites that are to be connected to an aqueduct. Where reservoirs are contemplated to increase supplies for downstream withdrawal or instream purposes, the appropriate measure of unit cost is C/Δ_{95} rather than C/Y_{95}. Δ_{95} is computed by first estimating R for all upstream reservoirs via eqn. 4, adding these values to find ΣR, dividing ΣR by the mean flow at the downstream reach to find $(\Sigma R)^*$, and then using the curve of Fig. 6 to estimate Δ^*_{95}. Multiplication by $\bar{Q}$ for the reach then gives the estimate of Δ_{95}.

Although there are considerable uncertainties, the methods developed here appear to provide a rational and empirically-supported

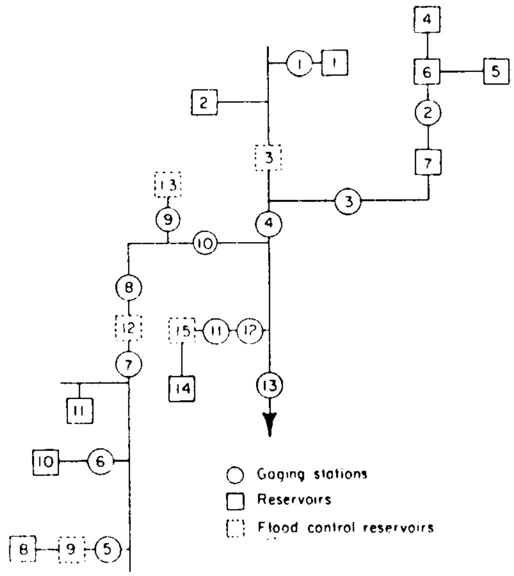

Fig. 5. Schematic diagram showing relations between reservoirs and gaging stations in the Merrimack Basin, New Hampshire

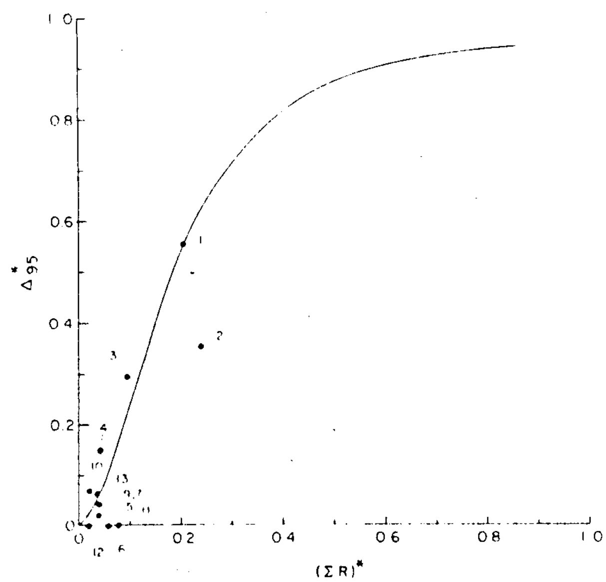

Fig. 6. "Theoretical" (curve) and actual (dots) relationship between Δ^*_{95} and $(\Sigma R)^*$ at gaging stations in the Merrimack Basins, New Hampshire. Numbers refer to stations shown in Fig. 5.

Table 2. Computations of Relations between Δ^*_{95} and $(\Sigma R)^*$, Merrimack River Basin, N. H. See text for symbol definitions. S^* in days, unstarred variables in ft^3/s. Refer to Fig. for locations of gaging stations and reservoirs.

Stream-Gaging Stations						Reservoirs						
Sta. No.	$\bar{Q}$	Y_{95}	Q_{95}	Δ_{95}	Δ^*_{95}	Res. No.	S^*	R^*	$\bar{Q}$	R	ΣR	$(\Sigma R)^*$
						1	385	.368	49.6	18.3		
1	87.7	51.7	3.0	48.7	.555						18.3	.209
						6	170	.259	491	127		
2	530	205	18.0	187	.353						127	.240
						7	31	.108	619	66.7		
3	693	228	22.9	205	.296						66.7	.096
						1	385	.368	49.6	18.3		
						2	114	.214	172	36.8		
						7	31	.108	619	66.7		
4	2755	747	328	419	.152						122	.044
						8	113	.213	14.3	3.0		
5	82.4	5.8	4.2	1.6	.019						3.0	.036
						10	37	.119	48.4	5.8		
6	99.4	3.1	3.1	0	0						5.8	.058
						8	113	.213	14.3	3.0		
						10	37	.119	48.4	5.8		
						11	36	.117	117	13.7		
7	627	66.5	38.9	27.6	.044						22.5	.036
						Same as Station 7						
8	678	63.7	40.0	23.7	.035						22.5	.033
						Same as Station 7						
10	1254	158	70.0	88.0	.070						22.5	.018
						14	67	.164	41.9	6.9		
11	91.2	3.2	3.2	0	0						6.9	.076
						Same as Station 11						
12	304	10.9	10.9	0	0						6.9	.023
						1	385	.368	49.6	18.3		
						2	114	.214	172	36.8		
						7	31	.108	619	66.7		
						8	113	.213	14.3	3.0		
						10	37	.119	48.4	5.8		
						11	36	.117	117	13.7		
						14	67	.164	41.9	6.9		
13	5223	935	606	329	.063						151	.029

basis for planning-level estimates of the value of reservoirs in increasing water supplies for instream and withdrawal uses in the Merrimack Basin.

References Cited

1. Butcher, W. S., Y. Y. Haimes, and W. A. Hall, Water Res. Rsch., v. 5, p. 1196-1204, 1969.

2. Dingman, S. L., Water Res. Bull., v. 14, p. 1481-1502, 1978.

3. Dingman, S. L., Proc. Third Conf. on Hydrometeorology; Bogotá, Colombia; Amer. Met. Soc., 20-24, Aug. 1979.

4. Langbein, W. B., U.S. Geol. Surv. Circ. 409, 1959.

5. Löf, G.O.C. and C. H. Hardison, Water Res. Rsch., v. 2, p. 323-354, 1966.

6. Committee on Rainfall and Yield of Watersheds in New England, Jour. New Engl. Waterworks Ass., v. 59, p. 314, 1945.

7. Riggs, H. C. and C. H. Hardison, Techniques of Water-Res. Invests. of the U.S. Geol. Surv., Book 4, Ch. 2, 1973.

8. Russell, C. S., D. G. Arey, R. W. Kates, Drought and water supply; Johns Hopkins Press for Resources for the Future, 1970.

9. U.S. Army Corps of Engineers, Manuals of computer programs of NAR study: Storage-yield, demand and supply models; Spec. Publ., North Atlantic Regional Water Res. Study, North Atlantic Div., U.S. Army Corps of Engrs., Oct. 1972.

10. U.S. Army Corps of Engineers, Merrimack River Water Supply Study; Northeastern U.S. Water Supply Study, Jan. 1977.

Acknowledgement

The work upon which this report is based was supported in part by funds provided by the United States Department of the Interior, Project A-050-NH, as authorized under the Water Research and Development Act of 1978, through the Water Resource Research Center of the University of New Hampshire. I thank Walter Langbein for helpful discussions of the relations among storage, yield, and regulation.

APPENDIX D

Preliminary Calculation of Yields of Isolated Aquifers in New Hampshire

Hall (1979) estimates a long-term average recharge rate to an isolated surficial aquifer in southeastern New Hampshire at about 30 cm/yr. For an aquifer of area $A_a (km^2)$, this amounts to $9.5A_a$ ℓ/s. The average volume of water in storage in the aquifer is $10^9 \; sh_{sat} \; A$ ℓ, where s is specific yield, and h_{sat} is the saturated thickness of the aquifer (m). The storage ratio of the aquifer, S_{aq} (days), is:

$$S_{aq} = \frac{10^9 \; s \; h_{sat} \; A}{(86{,}400)(9.5 \; A)} = 1{,}200 \; s \; h_{sat}$$

If s were assigned a typical value for sand-gravel aquifers of 0.3 then:

$$S_{aq} \simeq 360 \; h_{sat}$$

This indicates that there is about 1 year of storage for each meter of saturated thickness. No data are available to evaluate the relation between the yield available 95% of the time, Y_{95}, and S_{aq}. This relation depends primarily on the time distribution of recharge and the relation between storage and outflow. However, most aquifers developed for water supplies would have several meters of saturated thickness, and therefore several years of storage. As shown in Appendix A, a surface reservoir with several years of storage would have Y_{95}* nearly equal to 1. Thus, it seems reasonable to assume for planning purposes that a Y_{95} approximately equal to the long-term average rate of recharge can be obtained from an isolated aquifer.

APPENDIX E

Sample Calculation of Effects on Streamflow of Emergency Pumping from an Aquifer Connected to the Stream

For these calculations, an aquifer mapped by Cotton (1976) adjacent to the Contoocook River in Peterborough was selected. This aquifer measures about 13,000 ft by 1,600 ft and contains a well located about 1,200 ft from the river. The projected year-2000 population of Peterborough is projected to 8,880 (New Hampshire State Planning Office, 1977), which represents a domestic use rate of 0.98 million gallons/day (1.5 ft^3/s). For purposes of this example, we assume that all this water will be normally withdrawn from the Contoocook River. The drainage area above the U.S. Geological Survey gage on that river in Peterborough is 68.1 mi^2; if the 0.2 $ft^3/s\ mi^2$ minimum-flow rule is applied, Q_{min} = 13.6 ft^3/s. Again, for purposes of this example only, we assume that $Q_r = Q_{min} + W = 15.5\ ft^3/s$. Figure 13 shows streamflows recorded at the USGS gage during a 19-day period in 1977, again chosen simply as an example. In the absence of any withdrawals, the flow would have been below Q_{min} for 16 days and reached a minimum of 70% of Q_{min}. If 1.5 ft^3/s were withdrawn to satisfy municipal demands, the flow would be below Q_{min} for an additional two days and would reach a minimum of 60% of Q_{min}.

To estimate the effects on streamflow of pumping the required 1.5 ft^3/s from the aquifer via the method of Jenkins and Taylor (1974), we first compute f_q for the well. As noted, a = 1,200 ft; we select representative values of s = 0.25 and T = .080 ft^2/s and compute:

$$f_q = \frac{(1200)^2\ (.25)}{.080} = 4.5 \times 10^6\ s = 52\ \text{days}$$

Figure E1, taken from Jenkins and Taylor (1974), can be used to estimate the streamflow depletion (q) over time, where the pumping rate (Q) is 1.5 ft^3/s. The total time of pumping (t_p) for the situation shown in Figure 13 would be 16 days, so t_p/f_g = .31. Using the curve for t_p/f_g = .35 in Figure E1, we see that the peak streamflow depletion is

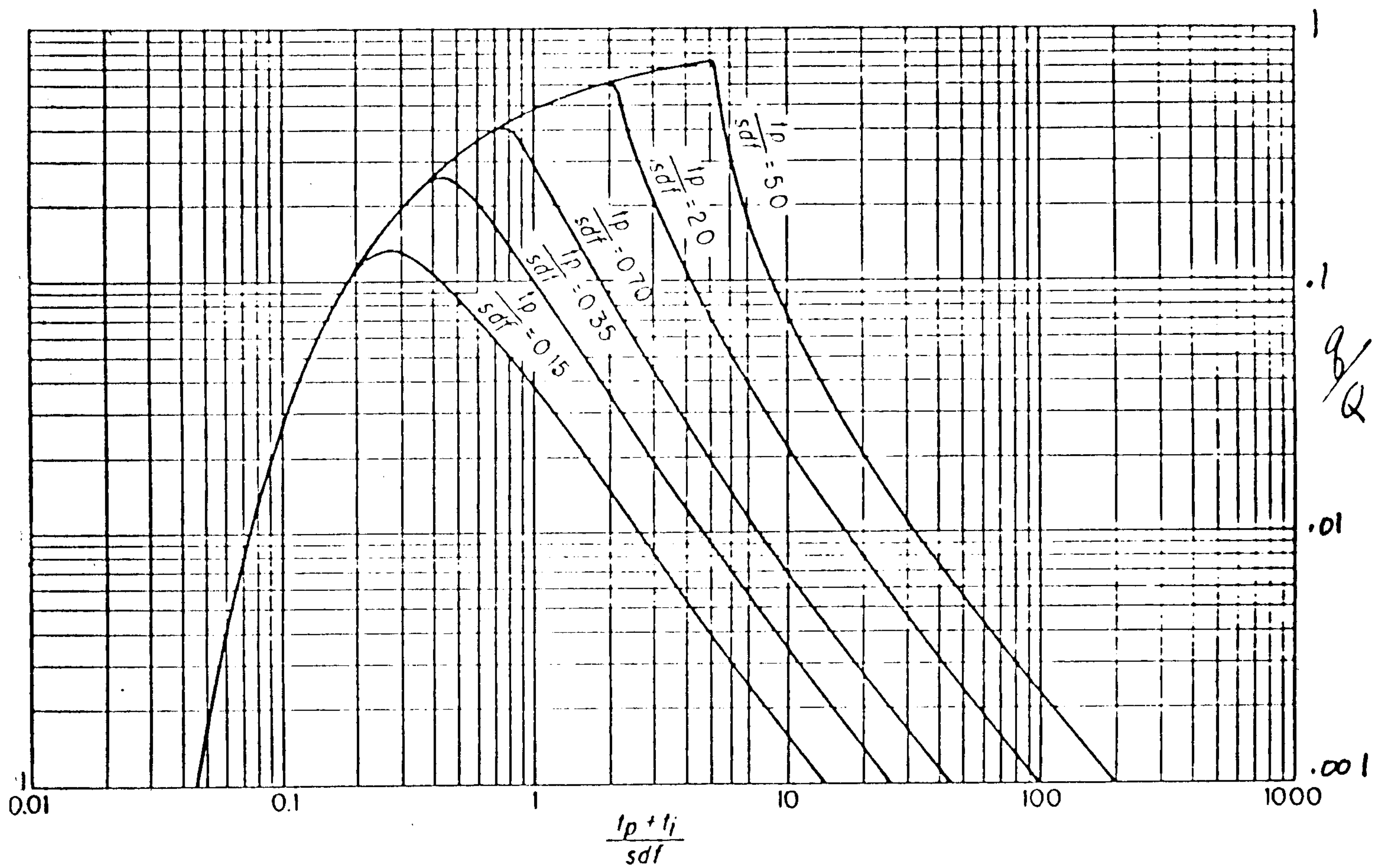

Figure E1. Dimensionless diagrams showing streamflow depletion, q, over time, $(t_p + t_i)$, as related to pumping rate, Q, and streamflow depletion factor, sdf.

$q/Q = .13$, occurring $.27\ f_g$ after pumping begins. In absolute units, this amounts to a maximum streamflow depletion of $.20\ ft^3/s$ occurring 16 days after pumping begins - clearly a very minor effect.

The flow-duration curve for the Contoocook at Peterborough shows that the assumed Q_{min} of $13.6\ ft^3/s$ is exceeded 92% of the time. Thus, pumping would be required only 8% of the time (29 days per year on the average) and there would be ample time for recharge of the aquifer during periods of higher flow.

APPENDIX F

Computations of Effect of Recycling on Chloride Concentration in Municipal Water

The situation described in Appendix A for Bow, New Hamsphire, is used in this example:

Population, $P = 5790$ persons
Water use rate, $U = 27.8$ ℓ/s

The effect of treatment on chloride content is assumed to be negligible, so $t_g = t_f = 0$, and Equations 3-48 and 3-49 apply.

Kuiper and Wechsler (1974) state that municipal use of water increases chloride concentration by about 125 mg/ℓ. Thus,

$$\frac{b_p}{a_p} = 125$$

and with $a_p = 0.0048$ ℓ/s person, b_p for chloride = 0.6 mg/s person. This gives

$$A = b_p P = (0.6 \text{ mg/s person}) \quad (5790 \text{ persons}) = 3474 \text{ mg/s}.$$

According to Hall (1975) the average concentration of chloride in the Merrimack Basin is 12 mg/ℓ, so we take $c_1 = 12$ mg/ℓ. With $k_c = .25$ (Table 3-4), Equations 3-48 and 3-49 can be solved to give the results shown in Table F-1 and plotted in Figure 17.

Table F-1

k_r	0	.1	.2	.3	.4	.5	.6	.7	.74
c_j (mg/ℓ)	16	209	245	296	378	524	881	2571	12,810
c_u (mg/ℓ)	12	32	59	97	158	268	533	1800	9,480

APPENDIX G

"Water Use Over Time" Estimated Relationships

For each of the three selected towns, projected water demand data were obtained for the years 1970, 1990, and 2020 (U.S. Army Corps of Engineers (1977)). Three functional forms were applied to this data to find which relationship yields the best statistical fit. The estimated forms notationally looked as follows:

$$\text{(1a)} \quad s_{ki} = a_i + b_i T_{ki} + u_k$$

$$\text{(1b)} \quad s_{ki} = c_i e^{d_i T_{ki}} u_k$$

$$\text{(1c)} \quad s_{ki} = f_i T^{g_{ki}} u_k$$

where, (1a), (1b), and (1c) are linear, semi-logarithmic and logarithmic, respectively; s_{ki} represents average water demand in m.g.d. for the k^{th} observation for the i^{th} town and T_{ki} denotes the trend factor for the k^{th} observation for the i^{th} town; a_i, b_i, c_i, d_i, f_i, and g_i depict the estimated parameters for the i^{th} town, and u_k is the stochastic disturbance for the k^{th} observation. The best statistical fit was attached to equation (1b) for each of the three towns. The estimated (1b) relationships for Hudson, Merrimack, and Nashua are found below.

$$\text{(2)} \quad \text{Hudson:} \quad s_1 = .4227e^{.544T}; \; R^2 = .937, \; t = 7.75$$

$$\text{Merrimack:} \quad s_2 = .6596e^{.53009T}; \; R^2 = .94, \; t = 8.03$$

$$\text{Nashua:} \quad s_3 = 6.5195e^{.12291T}; \; R^2 = .947, \; t = 8.40$$

Using (2), projected water demands can be made for the years 1990, 2000, 2010 and 2020 by designating the trend factors (3, 4, 5, 6) and calculating s_i for each of the years.

APPENDIX H

Economic Data[1/]

For our purposes, data utilized are preliminary in nature and reflect approximations where necessary. Cost data that relates to volume and distance were calculated using mileage and costs per unit per mile.

Capital Cost of the i^{th} Proposed Reservoir: C_i

	Capital Cost (1980 $)	t_0	t_1	t_2	t_3
C_1	$1,626,947	$513,877	$169,202	$53,689	$17,896
C_2	319,474	102,871	33,225	10,543	3,514
C_3	773,230	248,980	80,416	25,517	8,506
C_4	426,813	137,434	44,389	14,085	4,695
C_5	229,870	74,018	23,906	7,586	2,529
C_6	589,580	189,201	61,108	19,390	6,463
C_7	366,324	117,956	38,098	12,089	4,030

1/ Based on a 12 percent discount rate.

Unit Operation and Maintenance Cost of Proposed Reservoir i: b_i

	O & M Costs (1980 $)	t_0	t_1	t_2	t_3
b_1	$ 70/m.g.	$ 22.54	$ 7.28	$ 2.31	$ .77
b_2	320/m.g.	103.04	33.28	10.56	3.52
b_3	260/m.g.	83.72	27.04	8.58	2.86
b_4	311/m.g.	100.27	32.39	10.78	3.43
b_5	320/m.g.	103.04	33.28	10.56	3.52
b_6	270/m.g.	86.94	28.08	8.91	2.97
b_7	240/m.g.	77.28	24.96	7.92	2.64

Unit Operation and Maintenance Cost of the Proposed Pipeline from Proposed Reservoir i to Town j: e_{ij}

	O & M Costs (1980 $)	t_0	t_1	t_2	t_3
e_{11}	$11.65/m.g.	$ 3.75	$1.21	$ .38	$ .13
e_{12}	3.51/m.g.	1.13	.37	.12	.04
e_{13}	5.60/m.g.	1.80	.58	.18	.06
e_{21}	50.07/m.g.	16.12	5.21	1.65	.55
e_{22}	17.46/m.g.	5.62	1.82	.58	.19
e_{23}	52.85/m.g.	17.02	5.50	1.74	.58
e_{31}	20.66/m.g.	6.65	2.15	.68	.23
e_{32}	7.68/m.g.	2.47	.80	.25	.08
e_{33}	20.20/m.g.	6.50	2.10	.67	.22
e_{41}	13.58/m.g.	4.37	1.41	.49	.15
e_{42}	29.71/m.g.	9.57	3.09	.98	.33
e_{43}	15.84/m.g.	5.10	1.65	.52	.17
e_{51}	29.16/m.g.	9.39	3.03	.96	.32
e_{52}	12.61/m.g.	4.06	1.31	.42	.14
e_{53}	43.73/m.g.	14.08	4.55	1.44	.48
e_{61}	36.65/m.g.	11.80	3.81	1.21	.40
e_{62}	20.14/m.g.	6.49	2.09	.66	.22
e_{63}	55.90/m.g.	18.00	5.81	1.84	.61
e_{71}	17.24/m.g.	5.55	1.79	.57	.19
e_{72}	9.32/m.g.	3.00	.97	.31	.10
e_{73}	27.23/m.g.	8.77	2.83	.90	.30

Capital Cost of the Pipeline Constructed from

Proposed Reservoir i to Town j: d_{ij}

	Capital Cost (1980 \$)	t_0	t_1	t_2	t_3
d_{11}	\$1,033,842	\$ 332,897	\$107,520	\$ 34,117	\$11,372
d_{12}	3,432,353	1,105,218	356,965	113,268	37,756
d_{13}	2,150,391	692,426	223,641	70,963	23,654
d_{21}	994,137	304,012	98,190	31,157	10,386
d_{22}	2,778,715	894,746	288,986	91,698	30,566
d_{23}	899,299	289,574	93,527	29,677	9,892
d_{31}	1,338,325	430,941	139,186	44,165	14,722
d_{32}	3,714,728	1,196,142	386,332	122,586	40,862
d_{33}	1,131,809	364,443	117,708	37,350	12,450
d_{41}	1,753,760	564,711	182,391	57,874	19,291
d_{42}	809,623	260,699	84,201	26,718	8,906
d_{43}	1,498,832	482,624	155,879	49,461	16,487
d_{51}	1,076,119	346,510	111,916	35,512	11,837
d_{52}	2,488,526	801,305	258,807	82,121	27,374
d_{53}	717,414	231,007	74,611	23,675	7,892
d_{61}	1,536,619	494,791	159,808	50,708	16,903
d_{62}	2,811,259	905,225	292,371	92,771	30,924
d_{63}	896,766	288,759	93,264	29,593	9,864
d_{71}	1,498,832	482,624	155,879	49,461	16,487
d_{72}	2,811,259	905,225	292,371	92,771	30,924
d_{73}	851,927	274,320	88,600	28,114	9,371

Operation and Maintenance Cost per Unit of Current Yield from Existing Well z: g_z

O & M Cost (1980 $)		t_0	t_1	t_2	t_3
Hudson g_1	$311.41/m.g.	$100.27	$32.39	$10.28	$3.43
Merrimack g_2	288.10/m.g.	73.45	23.72	7.53	2.51
Nashua g_3	88.184/m.g.	28.40	9.17	2.91	.03

Source:

Capital Cost of Proposed Pipeline from Existing Well z to Town j: m_{zj}

	Capital Cost (1980 $)	t_0	t_1	t_2	t_3
m_{11}	$ 124,390	$ 40,054	$ 12,937	$ 4,105	$ 1,368
m_{12}	2,440,062	785,700	253,767	80,522	26,841
m_{13}	809,623	260,699	84,201	26,718	8,906
m_{21}	2,498,149	804,404	259,808	82,439	27,480
m_{22}	248,781	80,107	25,873	8,210	2,737
m_{23}	2,440,062	785,700	253,766	80,522	26,841
m_{31}	809,623	260,699	84,201	26,718	8,906
m_{32}	1,963,849	632,359	204,240	64,807	21,602
m_{33}	381,126	122,723	39,637	12,577	4,192

Unit Operation and Maintenance Cost for Proposed Pipeline from Existing Well z to Town j: k_{zj}

	O & M Cost (1980 \$)	t_0	t_1	t_2	t_3
k_{12}	\$29.20/m.g.	\$9.40	\$3.04	\$.96	\$.32
k_{13}	12.47/m.g.	4.02	1.30	.41	.14
k_{21}	29.20/m.g.	9.40	3.04	.96	.32
k_{23}	11.25/m.g.	3.62	1.17	.37	.12
k_{31}	12.47/m.g.	4.02	1.30	.41	.14
k_{32}	11.25/m.g.	3.62	1.17	.37	.12

Unit Operation and Maintenance Cost for Existing Pipeline from Existing Well z to Town j: ℓ_{zj}

	O & M Cost (1980 \$)	t_0	t_1	t_2	t_3
ℓ_{11}	\$ 1.89/m.g.	\$.609	\$.197	\$.062	\$.021
ℓ_{22}	1.25/m.g.	.403	.13	.041	.014
ℓ_{33}	.27/m.g.	.087	.028	.009	.003

Capacity of i^{th} Proposed Reservoir: R_i

	million gallons
R_1	55845
R_2	1679
R_3	8504.5
R_4	4489.5
R_5	1642.5
R_6	2628
R_7	3540.5

Capacity of z^{th} Existing Well: P_z

	million gallons
P_1	6387.5
P_2	18907
P_3	50005

APPENDIX I

The following maps can be purchased from:

The Water Resource Research Center
108 Pettee Hall
University of New Hampshire
Durham, NH 03824

Price: Overlays (8½ x 11): $1.00 ea.

Plate I. Communities in the New Hampshire portion of the Merrimack River Basin

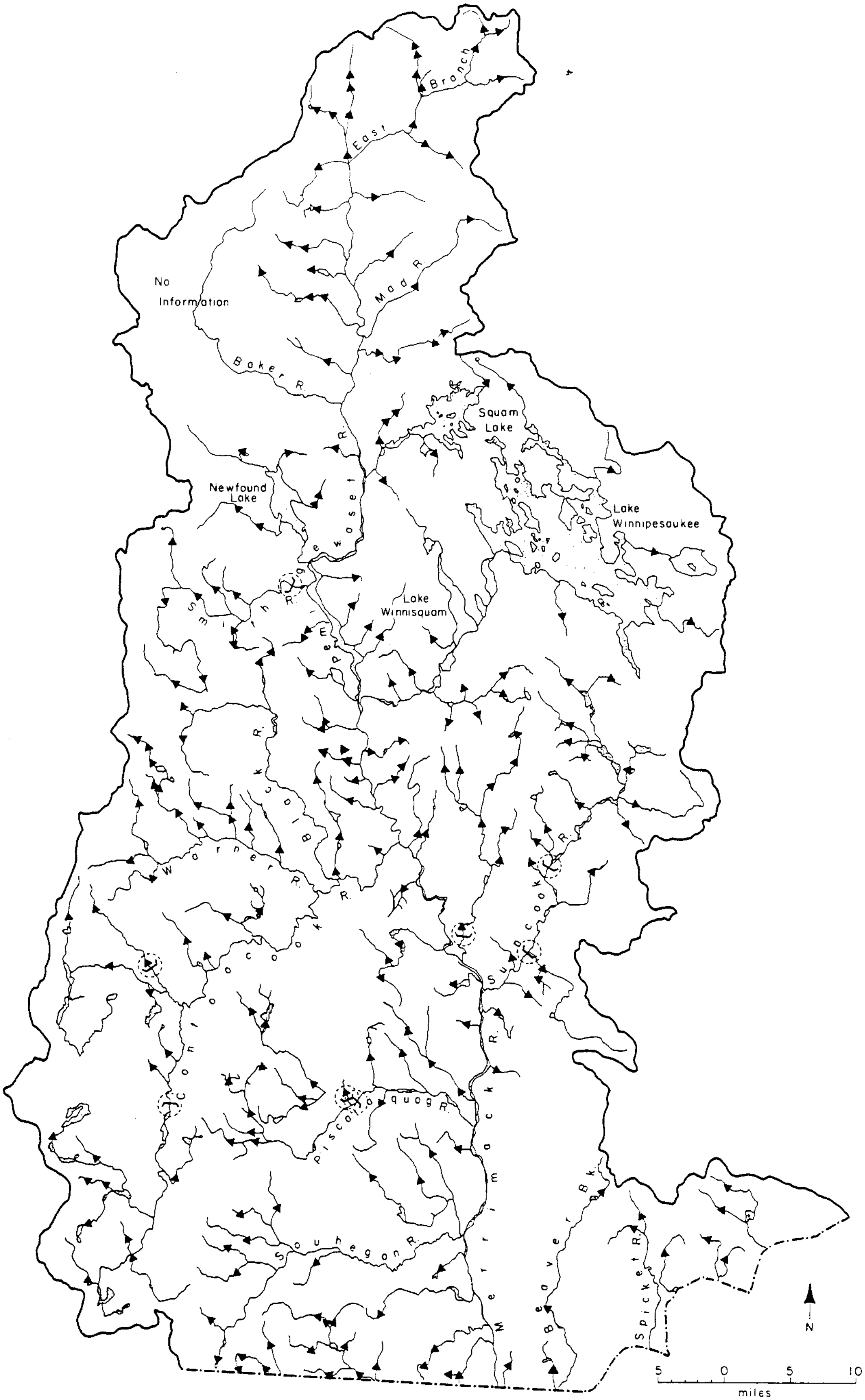

Plate II. Main streams and locations of reservoir sites identified by U. S Soil Conservation Service ▲ and U. S. Army Corps of Engineers

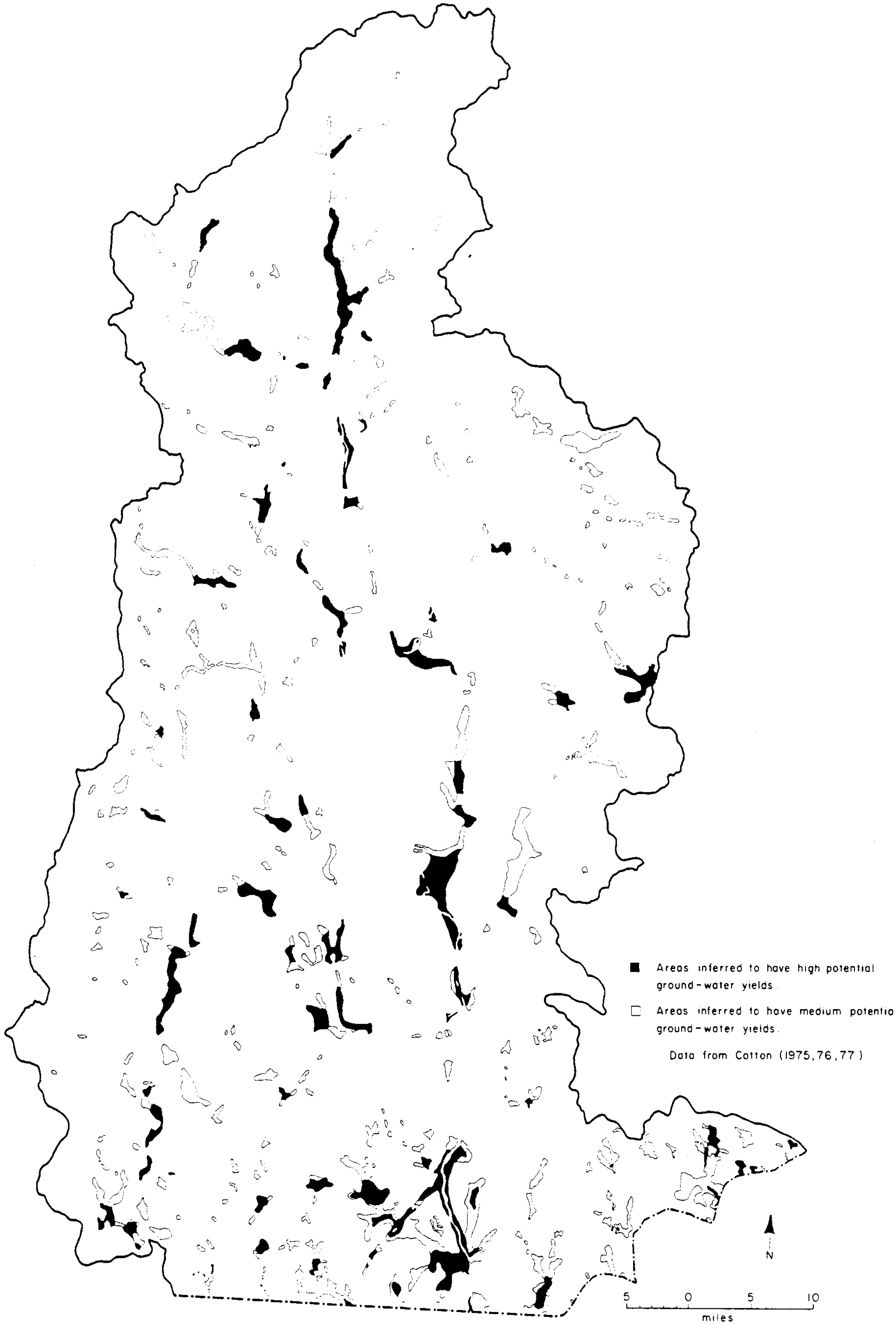

Plate III Areas with high and medium potential ground-water yields