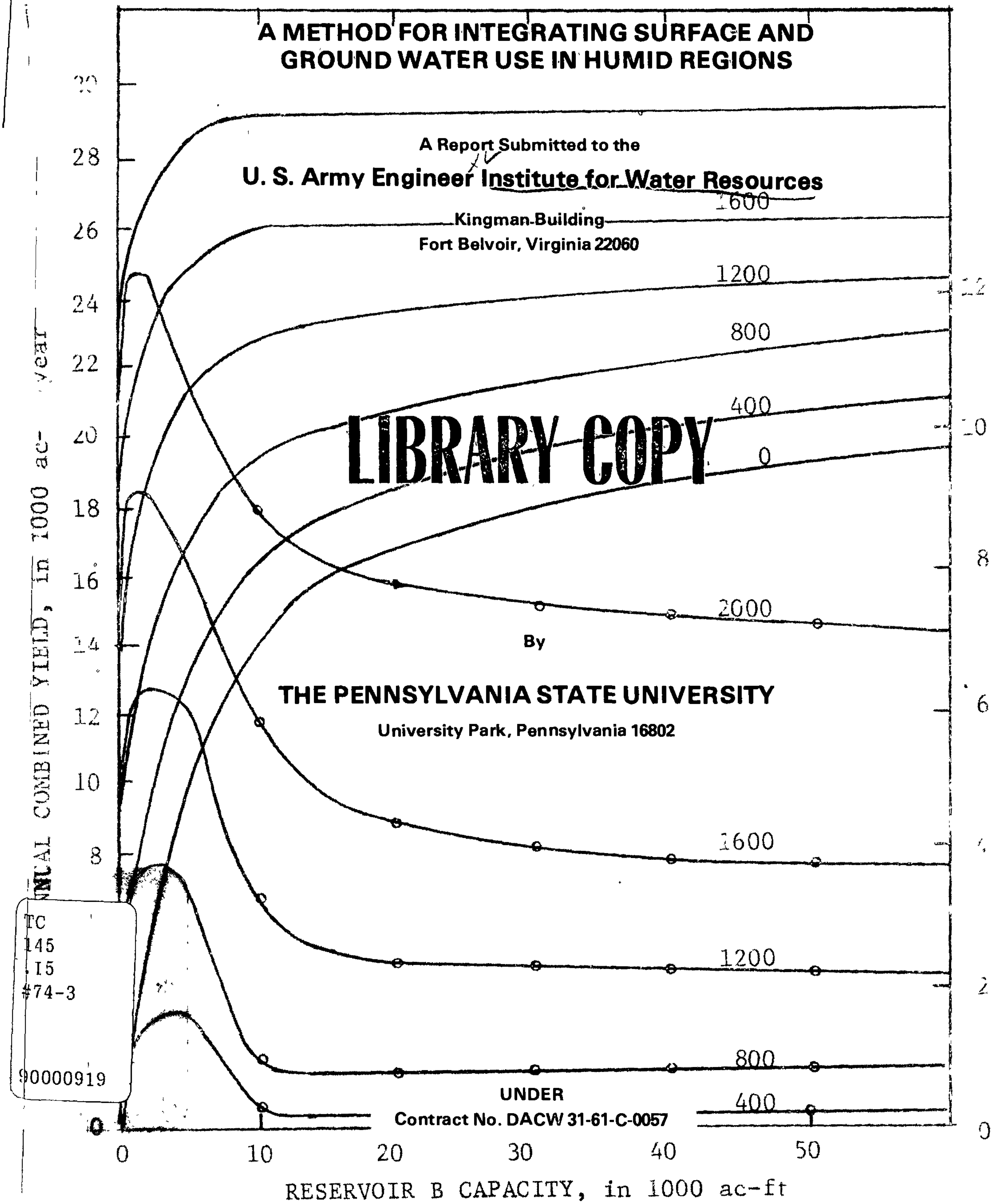

A METHOD FOR INTEGRATING SURFACE AND GROUND WATER USE IN HUMID REGIONS

A Report Submitted to the

U. S. Army Engineer Institute for Water Resources

Kingman Building
Fort Belvoir, Virginia 22060

By

THE PENNSYLVANIA STATE UNIVERSITY
University Park, Pennsylvania 16802

UNDER
Contract No. DACW 31-61-C-0057

FEBRUARY 1974

IWR CONTRACT REPORT 74-3

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Gert Aron
Thomas Rachford
John Borrelli
Walter Stottmann

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PREFACE

This project was initiated as a result of a proposal originally submitted on June 11, 1970 to the Institute for Water Resources (IWR) through The Pennsylvania State University Institute for Research on Land and Water Resources. After subsequent discussions between IWR and the project investigators, the proposal was modified and resubmitted in final form on November 25, 1970. Authorization to begin the resulting project, oficially entitled "The Integration of Ground and Surface Water Use in the Appalachian States," was communicated to the investigators on about February 16, 1971.

The primary objective of the study reported herein was to examine and to attempt to quantify the role that ground water should be accorded in future allocations of surface water storage for water supply. The major effort was directed toward identifying circumstances under which integrated use of ground and surface water sources would be economically and hydrologically desirable in humid areas like the Appalachian Region of the Eastern United States. The research involved development of a methodology and case study test, both of which are documented in this report. Several locations were considered for the case study, with the Elmira, New York water supply region finally being selected.

Numerous individuals and organizations made contributions to the study at various times. R. W. Harrison provided liaison on administrative and technical matters as IWR's representative. G. Antle and J. Tang, also of IWR, provided several good ideas which we attempted to implement. Corps of Engineers personnel in both the Baltimore and Pittsburgh Districts provided suggestions on possible case study sites and other data. Meetings were also held with H. Schwartz at the New York District Office of the Corps of Engineers and with G. K. Young of Water Resources Engineers, Inc., at Springfield, Virginia.

P. Milnes of the Pennsylvania Department of Environmental Resources provided water supply data on Hazelton, Pennsylvania while it was being evaluated as a case study site. The Elmira (New York) Water Company, and in particular, J. G. Copley, freely supplied information about the Elmira system and were very cooperative in the case study project.

This study was directed by G. Aron and T. M. Rachford of The Pennsylvania State University Department of Civil Engineering. Much of the technical work was carried out by J. Borrelli and W. Stottmann, both Graduate Assistants in the Department of Civil Engineering. Administrative support and clerical assistance in preparing this report were provided by The Pennsylvania State University's Institute for Research on Land and Water Resources.

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CHAPTER 1

INTRODUCTION

Conjunctive use of ground and surface water resources originated in the late 1940's in the arid Western United States in response to the growing water problems of that period. Today, the conjunctive use concept has been widely accepted by water resource planners and is considered to be prerequisite to optimal water utilization in arid and semi-arid regions. In contrast, water conservation *per se* has received less emphasis in humid regions because of a favorable hydrologic environment characterized by continual abundance of water and generally higher recharge rates.

Artificial recharge of ground water is essential for effective water conservation in arid regions and the term "conjunctive use" almost without exception implies the presence of artificial recharge facilities that would not be considered necessary in humid regions. To avoid misleading terminology we have adopted the term "integrated use" in lieu of the more conventional "conjunctive use." By our definition, integrated use, or alternately "coordinated use," refers to a carefully managed ground and surface water system which has neither a technological nor economic requirement of artificial recharge.

1.1 Justification for Study

The fact that water is relatively abundant does not assure a reliable water supply. Recent instances of water shortages in the humid Eastern United States have been well documented (Young et al., 1972; Anderson et al., 1972). The risk of such shortages is not dependent on the absolute quality of average annual yield in humid regions, but is more a function of hydrologic variability and the effective use of storage to dampen the consequences of this variability. In general, ground water yield is less variable than surface water yield, but it is a more difficult source to develop and often cannot alone supply the total demand to the service region. In contrast, opposition to exclusive surface water development is increasing for environmental and economic reasons. Existing surface storage reservoirs in some areas cannot anymore assure a reliable water supply for their service region without infringing on competing multipurpose storage allocations. These factors, together with further demand increases, lend considerable weight to the argument that more attention should be given to the integrated use of ground and surface water sources.

However, procedures for developing and operating ground and surface water systems together in an optimal manner have not been resolved. A recent statement by the American Water Works Association's Committee on Availability and Development of Water Supply (1969) confirms this supposition. The committee states: "We do not know how to develop conjunctively both streamflow and ground water for optimum use," and recommends that more research in the field of joint utilization of ground and surface water be undertaken.

The Water Resources Council's "Principles and Standards for Planning Water and Related Land Resources" (1971) also lend impetus to the study of joint utilization of ground and surface water supplies. Benefits and costs identified under the multiobjective accounts for Regional Development and Environmental Quality are likely to favor integrated ground and surface water use.

1.2 Objectives of Study

A water demand center confronted with the problem of having to select the most economical alternative for meeting its present and future water needs from an aquifer-reservoir system faces the following four questions:

1. Which of the available sources should be used?
2. To what degree should they be developed?
3. At which point in time should they be developed?
4. According to which broad operational scheme should both sources be managed?

The primary objective of this study was to answer the above four questions with particular regard to quantifying the role that ground water usage should be accorded in planning for present and future allocation of surface water storage.

1.3 Scope of Study

To achieve these objectives a two phase project was defined. The first phase was largely a conceptual determination of what an integrated ground and surface water system might consist of and how it might be operated. An interim report was prepared at the end of this phase (Aron et al., 1972) to summarize developments to that point. The second phase of the project emphasized analytical and numerical verification of the previously developed concepts and a case study test of their applicability.

Benefit evaluations of the economic or social worth of a given quantity of water were not attempted. The "demand" concepts of variable price-quantity relationships were also not considered to be within the scope of the study.

In short, we did not examine the worth of water in alternate uses, nor did we look at the selling price that would lead to an optimal resource allocation. The conceptual scope of the study was concentrated on the development and verification of a general methodology for determining the long-term timing, sizing and integrated operation of the components of a ground and surface water system.

1.4 Outline of the Report

The more important conclusions and recommendations of the study are summarized in the following chapter. A detailed description of the physical and hydrologic components of a ground and surface water system is given in Chapter 3 of this report. Most of the conceptual developments of the first phase of this project are assembled in Chapter 4. Their application to a prototype system is detailed in Chapters 5, 6 and 7. The basic least cost system is analyzed in Chapter 5; subsidary considerations based on the use of storage in the system are emphasized in Chapter 6; and general system sensitivities are explored in Chapter 7. Chapter 8 covers the case study results for Elmira, N.Y. and briefly describes other potential case study locations.

Additional details are contained in a set of appendices following this report. Appendix Z contains a listing of all cost equations used in this study and their sources. A general description of computational schemes for the economic analysis is given in Appendix B.

CHAPTER 2

SUMMARY AND CONCLUSIONS

This investigation was directed towards developing the concepts of integrated use of ground and surface water and the effects of these concepts on water yields and the associated costs of water supply. The major findings and conclusions of the study are summarized in this chapter.

2.1 Summary

The general framework of project development is shown in Figure 2.1. The concept of yield is treated by the use of a shortage index which quantifies the level of occasional shortages. This approach, which has been used by other previous investigators, is a refinement of the more traditional "firm yield" concepts. Surface water yield is calculated from a synthetic series of generated streamflow. Ground water yield is treated deterministically; aquifer drawdown is simulated in response to a monthly withdrawal schedule. The yield from single or multiple ground and surface water sources is compared with water usage based on projected population trends, and the shortage index calculated accordingly. This is shown schematically by the rectangular fields inside the large circle in Figure 2.1.

For a specified risk of shortage a system is defined and the costs of reservoir and conveyance, treatment of ground and surface waters of variable quality, and wellfield development and pumping are calculated. The components of the system are re-analyzed until an approximately least-cost system is determined. The definition of a near optimal water supply scheme was directed not entirely toward costs but likewise water conservation.

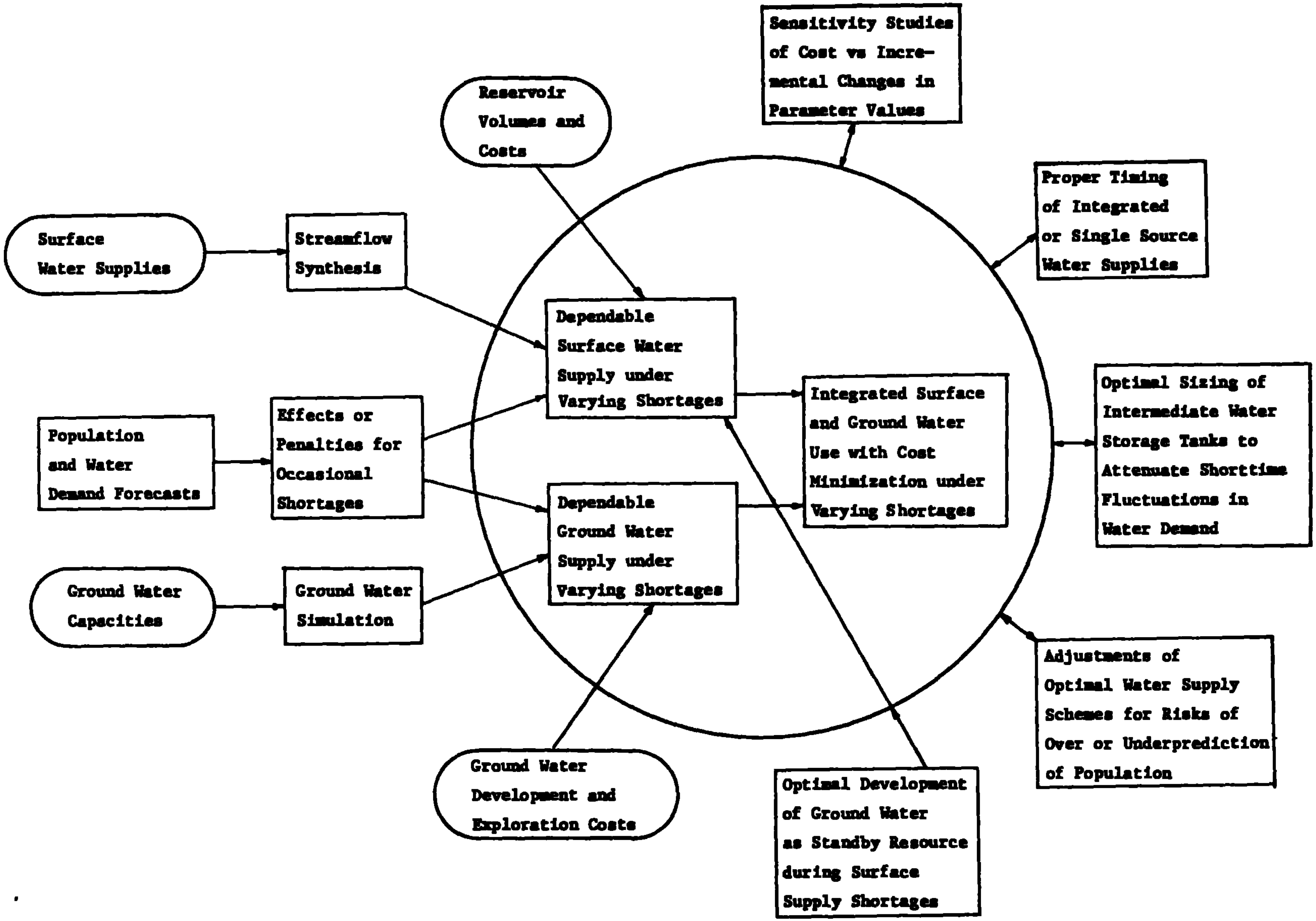

Fig. 2.1 - Flow Chart of General Project Development

A sensitivity study of influential parameters was also performed. Special attention was given to the time-capacity expansion of single source systems and the time of supplementation of a ground water source with a surface water source or vice versa. The use of intermediate storage and, in particular, its effect on system flexibility in response to over- or under-prediction of population was found to be an interesting concept. The use of ground water in a preventive pumping capacity to forestall surface reservoir shortages was also found to be a strong component of integrated use.

The basic attractiveness of integrated ground and surface water use resides in the characteristic differences between streams and aquifers as sources of water supplies as shown in Table 2.1. Through judicious coordination of these resources, the strengths of each overcome the weakness of both. The abundant but fluctuating surface water versus the lower but steady delivery of ground water; the limited but essentially free ground water storage versus the flexible but expensive surface water storage; and the differences in quality between ground and surface water are all factors to be considered in integrated use.

To provide quantitative examples of the water use integration procedures and effects, a hypothetical water supply and demand system was devised. Concepts that were found to have potential for success, based on the hypothetical system, were further explored by application to a case study situation utilizing available data.

There are two primary reasons for testing the methodologies on a hypothetical system prior to their application to a case study. First, the selection of a case study site is a slow process of deliberation;

Table 2.1. Stream and Ground Water Characteristics

CONSIDERATIONS	STREAM WATER	GROUND WATER
Water Replenished Rates and Volumes	Relatively large volumes with seasonally fluctuating flow rates often in disharmony with demand and withdrawal fluctuations.	Relatively low volumes with steady flow rates. Replenishment may increase with increasing withdrawals.
Storage Potentials	Reservoirs can provide desired storage volume, but sites may be difficult to procure and be wasteful in land use and water loss through evaporation. Easy water retrieval from storage.	Aquifers provide free storage of fixed, sometimes inadequate volume. Water retrieval requires wells, pumps, and lifting energy.
Water Quality	The water usually requires full conventional treatment (flocculation, coagulation, filtration, and disinfection), sometimes further treatment for taste and/or odor. Low CO_3 hardness.	Usually acceptable quality which does not require conventional treatment. Surface water percolating from stream to aquifer may receive effective filtering. Often high CO_3 hardness; expensive softening may be needed.

choosing and eliminating potential sites, field visits, data processing, and so on. Conceptual developments must await the selection of the case study site, which may be found inappropriate as concepts crystalize.

Secondly, and possibly more importantly, there is good reason for avoiding the constraints which any case study site will impose on the development of a general methodology. A hypothetical supply and demand system can be modified and tailored to accommodate the application of all concepts which seem of merit, and thus allows the desired generalization in the development of the methodology for integrated water use planning.

2.2 Case Study Results

The city of Elmira, New York was chosen as a case study site to demonstrate the planning methodology developed in Chapters 4 and 5 of this report and to test the cost equations which were assembled and presented in Appendix A.

Elmira proved to be a good site for the application of the methodology, because a water supply planner has the choice between a river supply of ample quantity and questionable quality, a seemingly reliable ground water supply with a slightly high $CaCO_3$ hardness, and several scattered small creeks which would require holding reservoirs to firm up their yields. This variety of alternative sources allowed the investigators to develop design yield curves and schedules of system expansion for integrated as well as separate supply systems.

In the economic analysis the integrated reservoir-aquifer system, which had appeared highly efficient from a water conservation point of view, could not compete with either river or ground water as a single supply or any combination of river and ground water. The overall least-cost supply and the chosen conditions consisted of a one-to-one mixture of treated Chemung River and untreated ground water which eliminated the need for ground water softening. The economically poor showing of the integrated reservoir-aquifer source combination was largely due to present regulations requiring full-scale treatment of all surface water supplies regardless of the degree of purity of the water source.

Despite the investigators' attempt to present the case study conditions as realistically as possible, many of the cost estimates had to be based on general equations which can only yield crude cost approximations. Thus the economic conclusions should not be taken as anything better than a demonstration of the cost accounting procedures used. A more complete cost analysis of the alternative schemes, using detailed on-site data and conditions, may well produce entirely different economic conclusions for the Elmira area.

2.3 Conclusions

This study produced two primary conclusions as follows:

1. From the viewpoint of water conservation, judiciously coordinated use of ground and surface water can be highly efficient if both sources are available in appreciable amounts.
2. From the viewpoint of economics, integrated use can produce cost savings under many conditions. In general, whenever surface reservoir storage is present in a system the opportunity exists for economic integration of ground water. The economic savings through integration are contingent on high marginal costs at the upper limits of either source development. Such conditions exist

when no single source can supply the total demand, or when a source that can satisfy the total demand is relatively expensive. Water quality considerations aside, conditions most unfavorable to integrated use occur when run-of-the river pumping without reservoir storage, or aquifer pumping without excessive drawdown can easily supply projected demand.

A factor that was found to have considerable potential for providing economic advantages to integrated use is the prospect of mixing ground and surface water of different qualities, particularly when ground water hardness is a major factor.

In addition to these primary conclusions, secondary conclusions were made as follows:

When ground water is used to supplement surface water supplies from a reservoir of limited conservation storage, consideration should be given to setting up a preventive pumping schedule. The policy of using ground water merely as a backup resource to be called on whenever surface water shortages arise was found to be highly ineffective. Under such circumstances, the maximum rate of ground water delivery, rather than the extent of aquifer storage, tends to be limiting. There is high likelihood that the ground water reserves would be subject to demands beyond their well field capacity. Ground water pumping should therefore start well ahead of and in anticipation of shortages. A preventive pumping rule was developed and seemed to work effectively on the hypothetical system.

The use of treated water storage beyond those volumes usually provided by local storage within the destribution system is also advocated. Such additional storage, termed "intermediate storage" in this report, can carry the demand center over a few days of exceptionally high water consumption and thus allow the pumping equipment, supply lines, and treatment facilities to be reduced in scale and to operate at a higher load factor.

Planners are cautioned to consider the risk of over- or under-estimates of population and water requirement trends. The effects of such mis-estimates can be costly if a system is very rigidly planned. A scheme that includes significant amounts of intermediate storage would be much more flexible to adjust to changing conditions and should therefore be given strong consideration.

CHAPTER 3

DESCRIPTION OF A GENERALIZED WATER SUPPLY SYSTEM

The results and conclusions of this study are based largely on an analysis of the hypothetical system presented in Figure 3.1. This system consists of one water use center and several potential water sources; three relatively small streams, all having the physical and hydrologic potential for reservoir storage available to the water use center; one relatively large river from which direct pumping without storage is feasible; and a nearby, exploitable aquifer.

The characteristics of the water sources and the water use center were selected so that any one of the sources would be capable of satisfying the yearly water use. In so far as possible, parameters were chosen so that no particular source is overwhelmingly more favorable than any other. To develop integrated use concepts, various combinations of two sources were tested. It was assumed that concepts developed for dual sources could later be extrapolated to multiple source combinations. Similarly, the system was limited to a single water demand center. The consideration of several scattered water use centers of differing characteristics would have necessitated a regional analysis beyond the scope of this study.

3.1 Characteristics of Water Demand Center

The water demand center could be any water user with a consumption schedule predictable over the entire span of the planning horizon. It could be an industry as well as a municipality. For the general analysis, hypothetical water consumption schedules are assumed to simulate expected

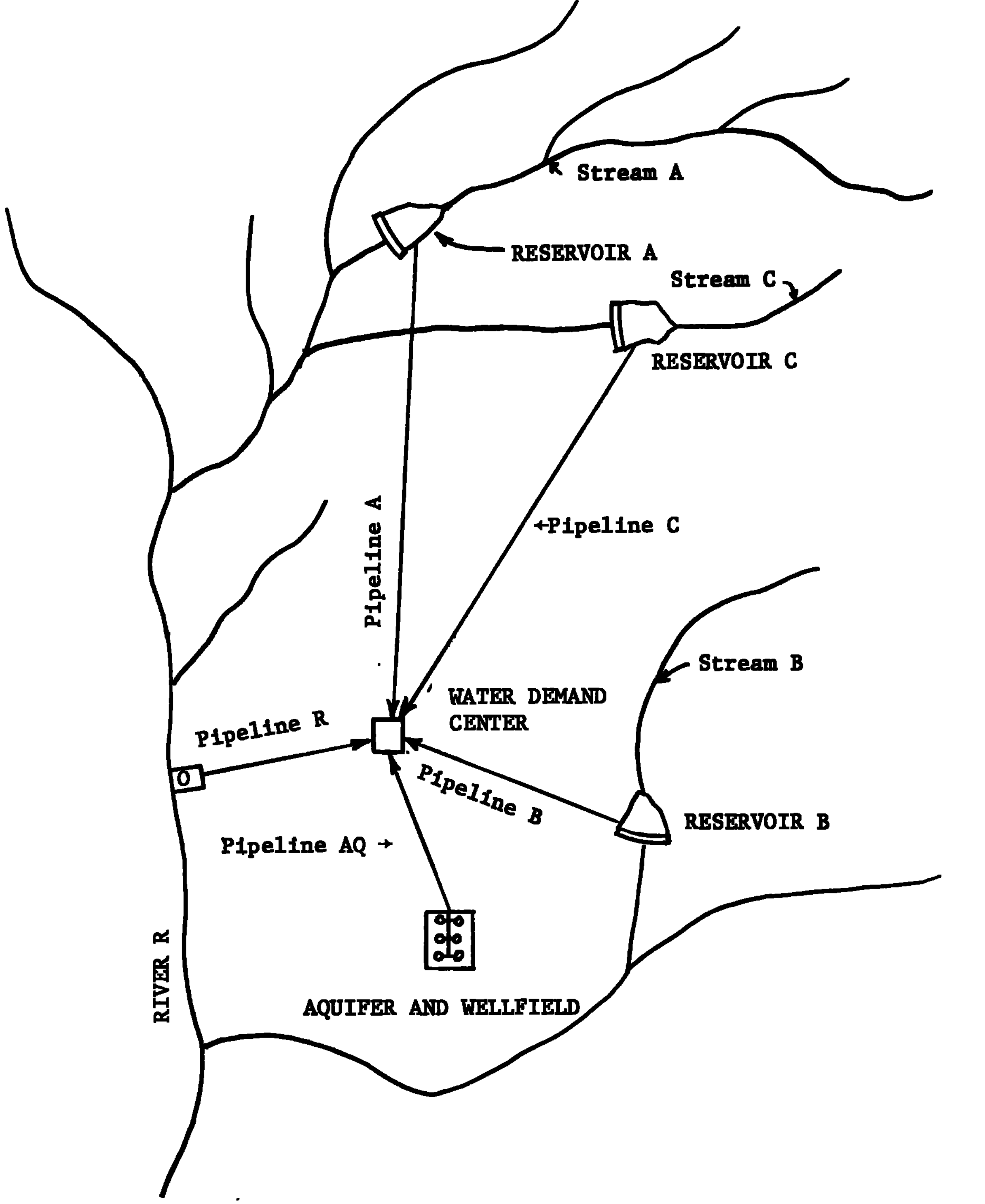

Fig. 3.1 - Sketch of Hypothetical Ground and Surface Water Supply System

water use patterns of medium to moderate size communities in the humid portion of the United States over the next few decades.

3.1.1 Size of Demand Center

Population projections are based on analyses of current populations and expected population growth rates. Table 3.1 shows the current size distribution of water use centers in the Appalachian Region of the United States. A surprisingly large percentage of all communities in this humid region fit into the population range between 10,000 and 75,000. The demand centers with base population exceeding this range are generally major metropolitan areas whose needs are supplied by a single surface water source, and were not good candidates for the integrated use study. Hence, an initial population value of 40,000 was assumed for the demand center and used throughout this study except where noted otherwise.

Appropriate growth rates were determined from demographic projections by the U. S. Corps of Engineers (1968) and by the State of New York (1968). There does not seem to be a standard pattern representative of communities in the region of the study; therefore a yearly linear growth rate of 2.5 percent of the initial population was used throughout the study except where noted.

3.1.2 Consumption Rates

Average consumption rates are calculated from annual population forecasts assuming an average daily per capita consumption factor of 150 gallons per day (gpd). The consumption factor adopted here is assumed to incorporate all types of industrial, commerical, and domestic water use. Surveys conducted by the American Water Works Association indicate that per capita consumption

Table 3.1. Distribution of Current Populations for Demand Center Locations

	Population Ranges[1]							
APPALACHIAN REGIONS	10,000 to 24,999	25,000 to 49,999	50,000 to 74,999	75,000 to 99,999	100,000 to 149,999	150,000 to 199,999	200,000 to 249,999	250,000 or more
West Virginia	8	5	2	-	-	-	-	-
Pennsylvania	64	14	6	2	3	-	-	2
Kentucky	17	4	1	-	1	-	-	1
Tennessee	21	6	-	-	1	1	-	2
Virginia	15	3	1	1	4	1	1	1
North Carolina	22	7	3	1	3	-	1	-
Georgia	23	6	2	-	2	1	-	1
Alabama	19	6	2	-	2	1	-	1
South Carolina	9	4	2	-	1	-	-	-

[1]From 1970 census data.

rates within a service area remain almost constant for all three uses (Seidel, 1966); which in total average approximately 150 gpd.

Consumption rate was not treated as a variable price-quantity dependent factor in the normal economic sense. Primarily, this study is focused on the prospect of providing a given quantity of water at least cost, or with least risk of shortage, or some equivalent constraint. This analysis would lead to the development of a supply curve, if taken to completion. This was not done, because the price of supply would include the cost of distribution to the consumer (Hirshleifer et al., 1969), which does not influence the least cost source development, and hence was not treated in the study.

When sufficient information is available, demand curves can be and are used to develop a schedule of yearly consumption rates. Young et al. (1972), in a study of the Washington, D.C. metropolitan area, developed a schedule of yearly consumption rates based on the price and elasticity of residential per capita water demands and the fixed ratios of industrial and commercial demands to the residential demand. They assumed that the average residential per capita use would increase at a rate of 0.25 percent per capita per year, but that it would be modified by price changes using the price-demand relationship. The average price elasticity of the residential demand was taken as -0.67. This elasticity was assumed not to be responsive to changes in price or increasing consumption rates. Other sources have reported that in humid regions municipal water demand, in particular, is generally inelastic (Howe and Linaweaver, 1967), and therefore moderate changes in price would not substantially alter projected consumption rates. This statement is also verified by the projects per capita consumption rates given by Todd (1970) which show the daily per capita consumption rates (on a national average) only increasing from 157 gal in 1965 to 170 gal in 2020.

3.1.3 Daily and Monthly Consumption Rate Fluctuations

Another important consideration in water usage is the fluctuation of monthly and daily water consumption rates about the yearly average. Within the range of fluctuations, the maximum day consumption rate is normally the shortest duration design flow. This parameter is generally used to determine the design capacity of a water supply system from the water treatment plant back to the source of supply (Camp and Lawler, 1969; Fair, 1971). The maximum day consumption rate is defined as being equivalent to the largest daily volume of water consumption during a given year. Various references give ratios of maximum to average day consumption rates (RMA) ranging from 1.50 to 2.90 (Babbitt, 1959; Camp and Lawler, 1969; Linaweaver et al., 1967). For the hypothetical supply systems an RMA ratio of 2.2 was determined from the daily consumption rate-duration curve shown in Figure 3.2. This curve, taken from Babbitt (1959), is representative for a typical municipality.

The ratio of the maximum hour to average daily consumption rate ranges from 2.0 to 7.0 (Fair et al., 1971). However, hourly fluctuations are of concern only in the design of the distribution system. It is standard practice to supply sufficient storage within the distribution system to attenuate all hourly fluctuations so that a constant rate of water is provided from the supply system on the day having the maximum volume of consumption during the design year.

Average monthly consumption rates, expressed as a percentage of annual consumption, were developed from the data reported by Babbitt and Doland (1955) and the California Water Resources Department (1966), and are shown in Figure 3.3. These were used in all calculations for the hypothetical system and are assumed to remain constant during the period of analysis.

3.2 Characteristics of Sources and Supply Systems

The assumed characteristics of the ground and surface water sources and their location relative to the demand center are listed in Tables 3.2 and 3.3.

The surface water sources are assumed to respond in parallel to the same microclimate environment so that individual yields are additive without any inter-source operational modifications. The hydrology for the surface water sources was based on historical records of non-related streams that do exhibit relatively similar annual regimes. This was assumed acceptable because the yield analysis in Chapter 4 is based on synthetic streamflow generated from the parameters of these historical data by the U.S. Army Corps HEC program (1966).

According to current practice in water resource development, reservoirs are designed to meet multipurpose use and storage space is provided according to priority of use. The optimal design of multipurpose reservoirs is very difficult to generalize and is highly dependent on specific local circumstances. The storage volumes given in Table 3.2 and the cost analysis later in this report are based on single purpose considerations. However, the procedures for integrated use analysis can be incorporated into a multipurpose framework if desired.

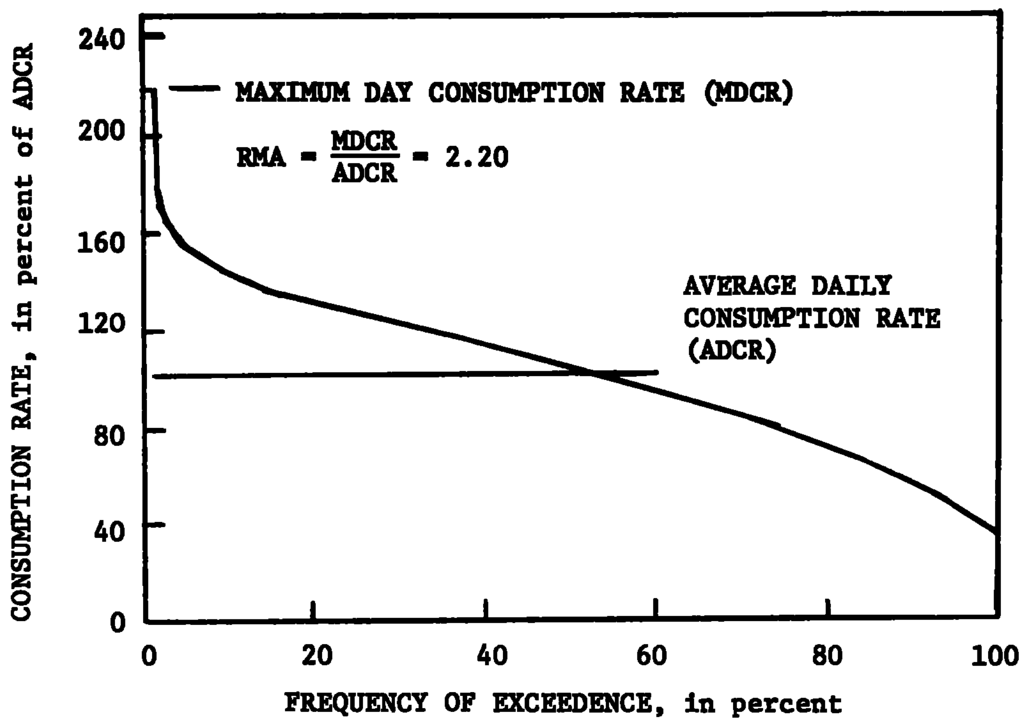

Fig. 3.2 - Daily Consumption Rate-Duration Curve (after Babbitt, 1959)

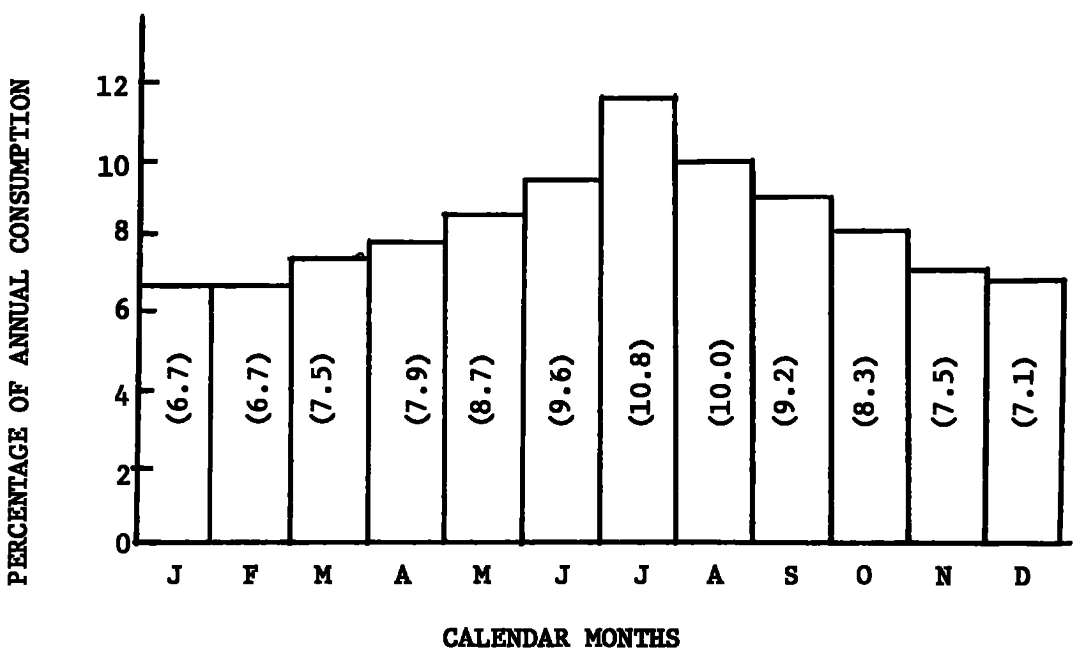

Fig. 3.3 - Monthly Consumption As a Percentage of Annual Consumption

Table 3.2. Physical Data and Dimensions of Hypothetical Surface Water System

Characteristics of Surface Water Sources	River R	Stream and Reservoir A	Stream and Reservoir B	Stream and Reservoir C
Streamflow based on	Chemung R. at Chemung, N.Y.	Muncy Cr near Stonestown, Pa.	[illegible] Elmira, N.Y.[1]	Moshannon Cr. at Osceola Mills, Pa.
Period of historical record	Sept. 1903 to present	October 1940 to present	May 1938 to present	October 1940 to present
Drainage area (mi^2)	2,506	23.8	28	68.8
Ave. annual discharge (cfs)	2,450	45	30.6	107
Ten year-1 month low flow (cfs)	50	3.0	1.0	9.0
Distance from intake to treatment plant (mi)	6	8	6	10
Increase in elevation from intake to treatment plant (ft)	+150	-100	-150	+150
Maximum reservoir storage capacity (ac-ft)	N.A.	60,000	60,000	60,000
Average yearly turbidity (ppm SiO_2)	50	50	50	50
Average yearly hardness (ppm $CaCO_3$)	40	40	40	40

[1]The drainage area at the gage is 77.5 mi^2. Drainage area and discharge were reduced by a factor of 0.36.

Table 3.3. Physical Data and Dimensions of Hypothetical Ground Water System

Characteristics of Aquifer and Wellfield	
Aquifer area (mi^2)	20
Average aquifer thickness (ft)	200
Depth to aquifer (ft)	100
Maximum aquifer storage volume (ac-ft)	179,200
Maximum allowable aquifer drawdown (ft)	133
Maximum storage depletion (ac-ft)	90,000
Average recharge rate (mgd/mi^2)	0.50
Average well discharge (gpm)	350
Permeability (gpd/ft^2)	100
Specific yield	0.07
Average distance to treatment plant (mi)	6
Increase in elevation from wellfield to treatment plant (ft)	+150
Average yearly turbidity (ppm SiO_2)	negligible
Average yearly hardness (ppm $CaCO_3$)	200

Aquifer characteristics necessary to model and simulate drawdown in response to monthly pumping are given in Table 3.3. These ground water source characteristics are all hypothetical, but are assumed to be typical of a reasonably productive ground water source in the Northeastern United States. A U.S.G.S. study has found that the average well discharge in this region is approximately 350 gpm and therefore that value was used in this study (Cederstrom et al., 1971).

In any aquifer study, the question of maximum allowable drawdown arises. This is a much discussed topic in water resource management literature (Walton, 1970). Opinions on the subject are manifold, ranging from the view that water table levels should be maintained at least at current levels to the opposite position that ground water should be mined to the limits of physical and economic feasibility. A limiting drawdown of two-thirds of aquifer depth and one-half of storage volume within the aquifer was adopted for this study.

Water quality characteristics of turbidity and hardness were assumed for both ground and surface water sources as shown in Tables 3.2 and 3.3. These are representative values of typical ground and surface water sources and are two parameters that do exhibit treatment cost sensitivities. Other parameters may be far more important in particular situations. A discussion of the types of treatment analyzed and their costs is given in Appendices A and B of this report.

The initial status of water supply systems varies greatly from one system to another, and effective generalization is difficult. Therefore, in the study that follows in Chapters 4, 5 and 6, the assumption of initially undeveloped conditions is maintained. This is a critical point in system expansion. The decision of whether to add to an existing system or begin anew is most difficult, often having political as well as economic implications. These considerations were considered to be beyond the scope of this study.

It is recognized that other options such as the use of renovated wastewater for direct recycling or ground water recharge also exist but were not considered here. These are certainly worthy of consideration in the overall water-use cycle and ultimately should be incorporated into integrated water use planning.

CHAPTER 4

METHODOLOGY FOR THE DETERMINATION OF A LEAST COST WATER SUPPLY ALTERNATIVE

The primary objective of this chapter is to formulate and demonstrate a general methodology that can be used for determining in a step-by-step procedure an approximately optimal water supply system from a set of feasible alternatives. The criterion of optimality is a system that approaches least cost while meeting a specified water use requirement.

Formal optimization techniques such as linear or dynamic programming were considered but ultimately rejected for various reasons. Mathematically, the system is not well-behaved; it is highly nonlinear and contains many interdependences. To produce a workable mathematical programming statement of this system would have required considerable oversimplification. Further, too much effort would have been devoted to the mechanics of solving the mathematical program, all at the expense of the analysis of integrated use concepts that were the main focus of the study.

As indicated in Chapter 3, five possible water supply sources were available for consideration. Allowing for all possible combinations and hierarchies in installation sequences, including the entire range from single source to five source systems, there would be 325 alternatives. However, a dual source system presents the advantages of integrated operation and yet avoids the replication of capital expenditure present in schemes drawing from three or more sources and so a least-cost system is likely to be a single or dual source system. For these reasons, the following analysis is restricted to exploring only single and dual source combinations for river R, streams A and B, and aquifer AQ. This reduces the number of alternatives to 16 as enumerated in Table 4.1. Stream C is not considered until the analysis of intermediate storage presented in Chapter 6.

4.1 General Solution Strategy

The procedure developed here follows a sequence of several, essentially independent stages, each leading to intermediate solutions serving as inputs to the succeeding stage.

In Figure 4.1, the procedure is presented in the form of a chart for the general case of one surface and one ground water source. The decision variables, which are shown for each stage of the procedure by rectangular boxes, are those quantities that are determined by the planning procedure. Oval boxes are used to denote physical paramaters, including such measurable quantities as streamflow, aquifer properties, and population data. Hexagonal boxes indicate external variables or constraints that are beyond the control of authority water supply planning study. Examples of this category are low flow release requirements, maximum aquifer drawdown, or perhaps reservoir operating rules mandated by multipurpose functions. Finally, trapezoidal boxes are used to show intermediate processing operations.

The five stages of the suggested procedure are introduced briefly below:

Stage 1 - Determination of the Combined Yield of a Surface and Ground Water System: The long-term yield from an aquifer-reservoir system is determined for a wide range of reservoir sizes and aquifer wellfield capacities. The relative contribution of each source may vary between 0 and 100 percent. Taking into consideration the stochastic nature of yield from systems containing surface water sources, the yield determination

Table 4.1. List of Possible Water Supply Alternatives

Alternative Designation	Description of Alternative
I	Alternatives involving one source only
I A	Reservoir A only
I B	Reservoir B only
I R	Direct river pumping
I AQ	Aquifer only
II	Alternatives involving two surface sources
II A-B	Reservoir A first, Reservoir B later
II B-A	Reservoir B first, Reservoir A later
II R-A	River supply first, Reservoir A later
II R-B	River supply first, Reservoir B later
II A-R	Reservoir A first, River supply later
II B-R	Reservoir B first, River supply later
III	Alternatives involving surface and ground water source
III A-AQ	Reservoir A first, Aquifer later
III B-AQ	Reservoir B first, Aquifer later
III R-AQ	River supply first, Aquifer later
III AQ-A	Aquifer first, Reservoir A later
III AQ-B	Aquifer first, Reservoir B later
III AQ-R	Aquifer first, River supply later

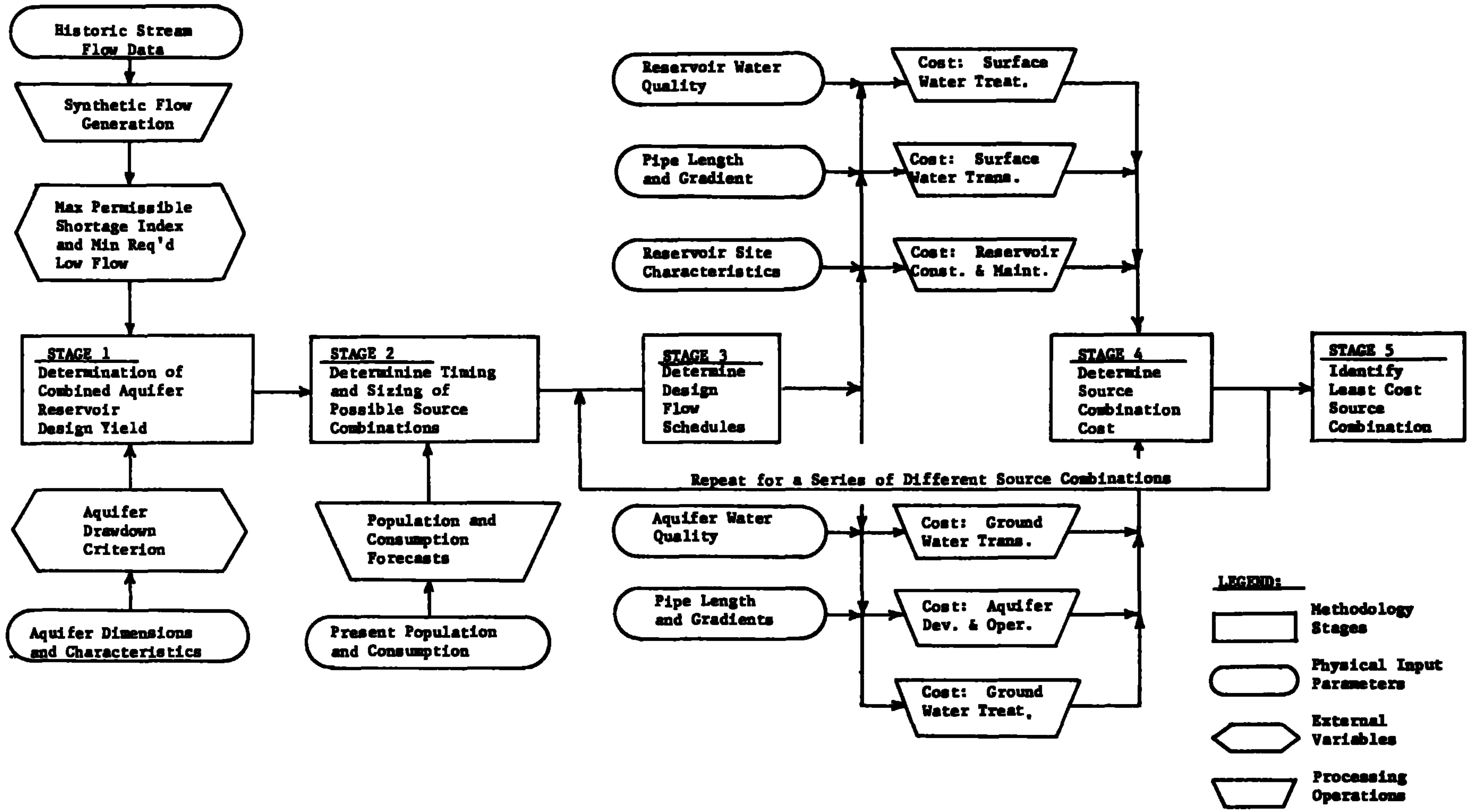

Fig. 4.1 - Flow Diagram Outlining Procedure for Determining Least Cost Source Combination for Alternatives With One Ground Water and One Surface Water Source

procedure should be based on simultaneous reservoir-aquifer routing with series of synthetic monthly streamflow sequences. The final result of the procedure will be a set of curves similar to those shown in Figure 4.2. A detailed description of a yield determination procedure is presented in section 4.3.

Stage 2 - Determination of Feasible Source Combinations: Using the set of combined yield curves from Stage 1 as well as the demand center's schedule of annual water consumption, the range of combinations of wellfield and reservoir capacities that can satisfy the demand center's water requirements in the last year of the planning horizon are identified. This information is summarized in the form of a yield isoquant, an example of which is shown in Figure 4.3. Also shown in Figure 4.3 is a time-of-supplementation curve, indicating the point in time when the initial source (reservoir A) requires supplementation from the second source (reservoir B or aquifer). For example, a reservoir capacity of R units and a monthly pump capacity of P units will be needed in year 50, the last year of the planning horizon. The reservoir is constructed first and can supply all the water until year N. Starting in year N the aquifer will be developed gradually until it reaches its final pump capacity of P in year 50. An in-depth description of yield isoquants of time-of-supplementation curves is presented in section 4.4

Stage 3 - Determination of Annual Schedules of Design Flow Rates: From the range of feasible water supply combinations developed in Stage 2 a number of combinations uniformly distributed over the entire feasible range is selected. For each of the combinations chosen, annual schedules

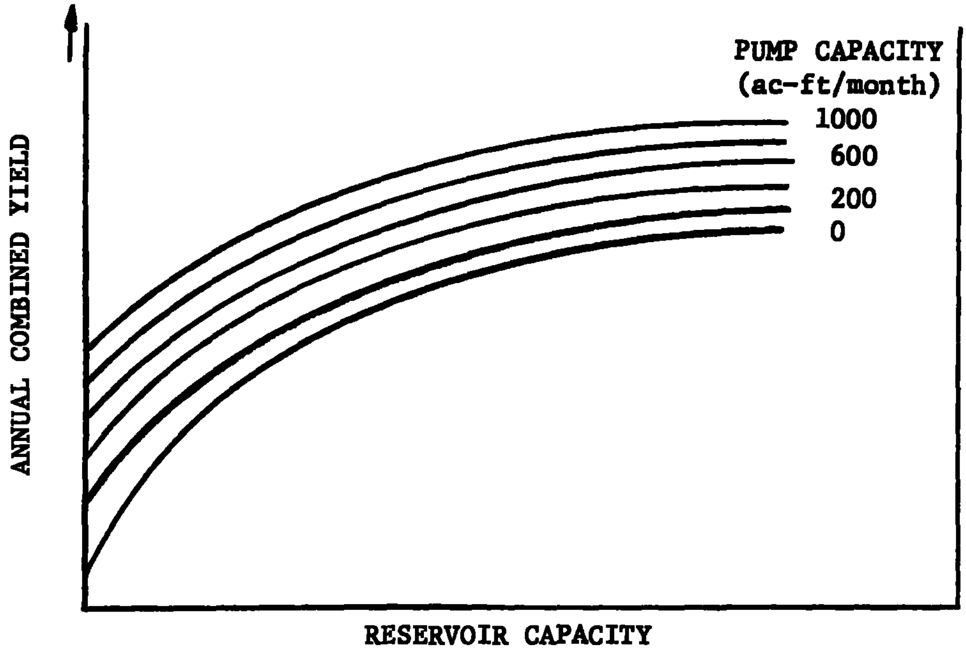

Fig. 4.2 - Combined Yield as a Function of Reservoir Size and Aquifer Pump Capacity

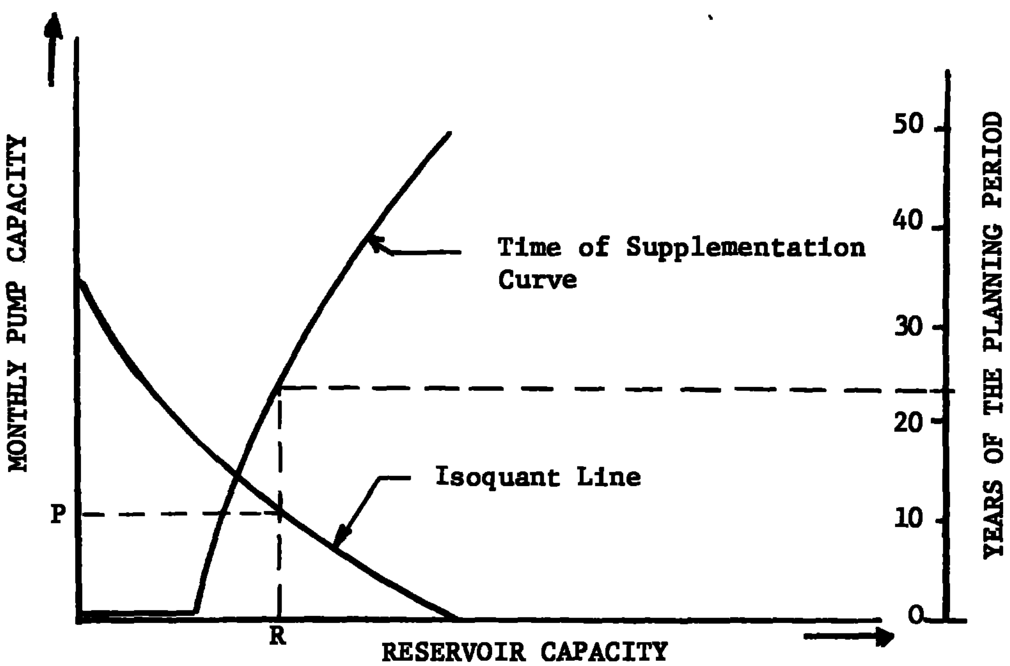

Fig. 4.3 - Time of Supplementation Curve and Isoquant Line

of maximum day flow rates and of monthly flow volumes pertaining to surface water transportation, ground water pumping and transportation, and surface and ground water treatment are determined. Design flow schedules of maximum day flow rates specify the capacities which each supply source, as well as the associated pumps, pipelines, and treatment facilities must equal or exceed during each year of the planning horizon. A schedule of consumption volumes must also be developed for each month of every year during the planning horizon. How these schedules can be calculated is explained in detail in Chapter 5.

Stage 4 - Calculation of Costs: Cost calculations can be treated as an independent stage in the analysis. These calculations require an input of primary decision variables, design flow schedules as calculated in the previous step, and physical input parameters. They produce values for secondary decision variables and individual as well as total costs. Examples of cost calculations are described in detail in Chapter 5 and Appendix A. The corresponding computational schemes and assumptions are summarized in Appendix B.

Stage 5 - Identification of Least Cost Combination: In this stage the costs of the source combinations selected at the beginning of Stage 3 should be compared. The combination with the lowest cost constitutes the "best" combination within the alternative and its cost and resulting decision variables are retained as being representative for the one alternative under consideration. A judicious examination of alternatives is required at this stage because there is no guarantee that the response surface is concave.

4.2 Design Yield Definition

The time distribution of runoff from natural watersheds is a highly stochastic phenomenon. The greatest problem in determining water yield estimates from surface streams is the uncertainty of the representativeness of historical droughts as an indicator of future drought potential (Close, Beard, and Dawdy, 1970). Yield, no matter how defined, is a probabilistic variable and should be treated as such.

In recognition of this uncertainty, the application of deterministic procedures based on critical drought periods such as the traditional mass curve analysis by Rippl (1883) was deemed too simplistic. Following current hydrologic practice, water yield problems can be solved on the basis of month-by-month simulation studies with recorded or hypothetical streamflows (Fiering and Jackson, 1971). The use of several synthetic flow sequencies, based on the mean, standard deviation, and skew of the historical sample, has the advantage that yield estimates can be based on a wide spectrum of statistical information. For the purpose of our study, 1,000 years of continuous streamflow were synthesized for each of the streams used in the hypothetical system.

4.2.1 Concept of Shortage Index

Strictly speaking, there cannot be a firm yield with absolutely no shortages whatsoever, because no matter how long a synthetic flow sequence is used, there is always a small chance of surpassing the worst drought generated. Thus, it is apparent that the risk of certain rare shortages should be incorporated into any study, perhaps in the form of a maximum allowable shortage index, which in itself is not a new idea. Beard (1964) proposed a shortage index as the sum of the squares of annual shortage ratios over a 100 year period. The shortage index developed for this study closely resembles Beard's index. The concept of the water supply loss function (Maass et al., 1962) is utilized to relate shortage index and economic losses resulting from water supply deficits. In Figure 4.4, as an example, the upper and lower limits of a water supply loss function for Lehigh, Pennsylvania are shown (Hufschmidt and Fiering, 1966). Under the assumption that the parabolic shape of the Lehigh water supply loss function is somewhat typical for small to medium size towns in the Eastern United States, the monthly shortage index for this study was defined as follows:

$$MSI_i = \left[\frac{B \cdot SH_i}{D_i}\right]^A \tag{4.1}$$

in which MSI_i = shortage index for month i, SH_i and D_i = volumes of water shortage and target draft during month i, and A and B = parameters.

The magnitude of A reflects the relative severity of rare major shortages versus more frequent minor shortages. With the value of A = 1.6, derived from the Lehigh data and adopted for this study, a 10 percent shortage occurring once in a given time span is equally severe as a 5 percent shortage occurring 3 times. The parameter B is merely a multiplier, conveniently set equal to 2 in order to give the monthly index a value of unity when the shortage equals 50 percent of the target draft. The yearly shortage index YSI is defined as the sum of the monthly shortage indices.

4.2.2 Choice of a Limiting Shortage Index

The reservoir design yield is now defined as that target output which a supply system can meet subject to a chosen maximum allowable shortage

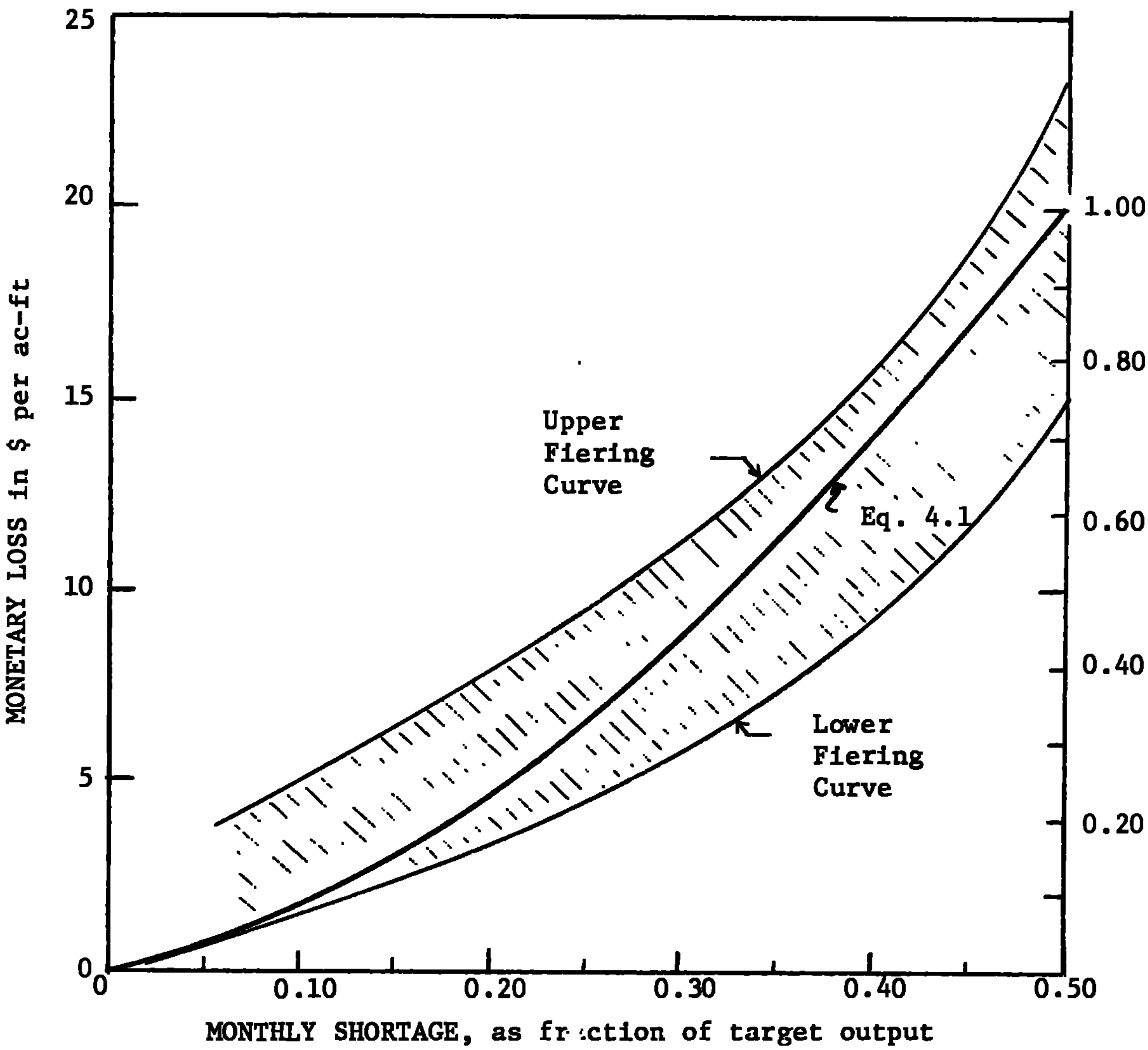

Fig. 4.4 - Water Supply Loss Function for Lehigh, Pa., and Corresponding Monthly Shortage Index

index. This maximum index was considered in this study as an external decision variable to be set by the decision-making agency in charge.

In theory, it should be possible to find an optimal shortage index by balancing the marginal costs of the increased system capacity due to lowering the limiting shortage index against the marginal losses incurred by increasing the shortage index limitation. However, not enough data on the costs of water shortages were available to warrant such a side study at this time. Recently, a study by Young et al. was completed on the effects of water shortages in York, Pennsylvania. The results from this and similar studies could be used at a future time to determine an optimal shortage index. In the application to the hypothetical system and the case study, an arbitrary value of 0.05 was chosen as the maximum allowable average annual shortage index.

4.3 Design Yield Determination

Design yield, as defined in section 4.2, is computed in an iterative procedure, routing inflows and outflows through the reservoirs and the aquifer and registering spills and shortages as outlined in Figure 4.5.

In the procedure described in this section, ground water plays the role of supplementing surface water in varying degrees as needed. Therefore, the detailed routing description will begin by setting ground water pumping to zero, present examples of yield values for two reservoirs, and finally deal with the procedure chosen to supply ground water.

4.3.1 Reservoir Routing Procedure

For an assumed annual target output, month-by-month reservoir routing through N years is performed according to the following algorithm:

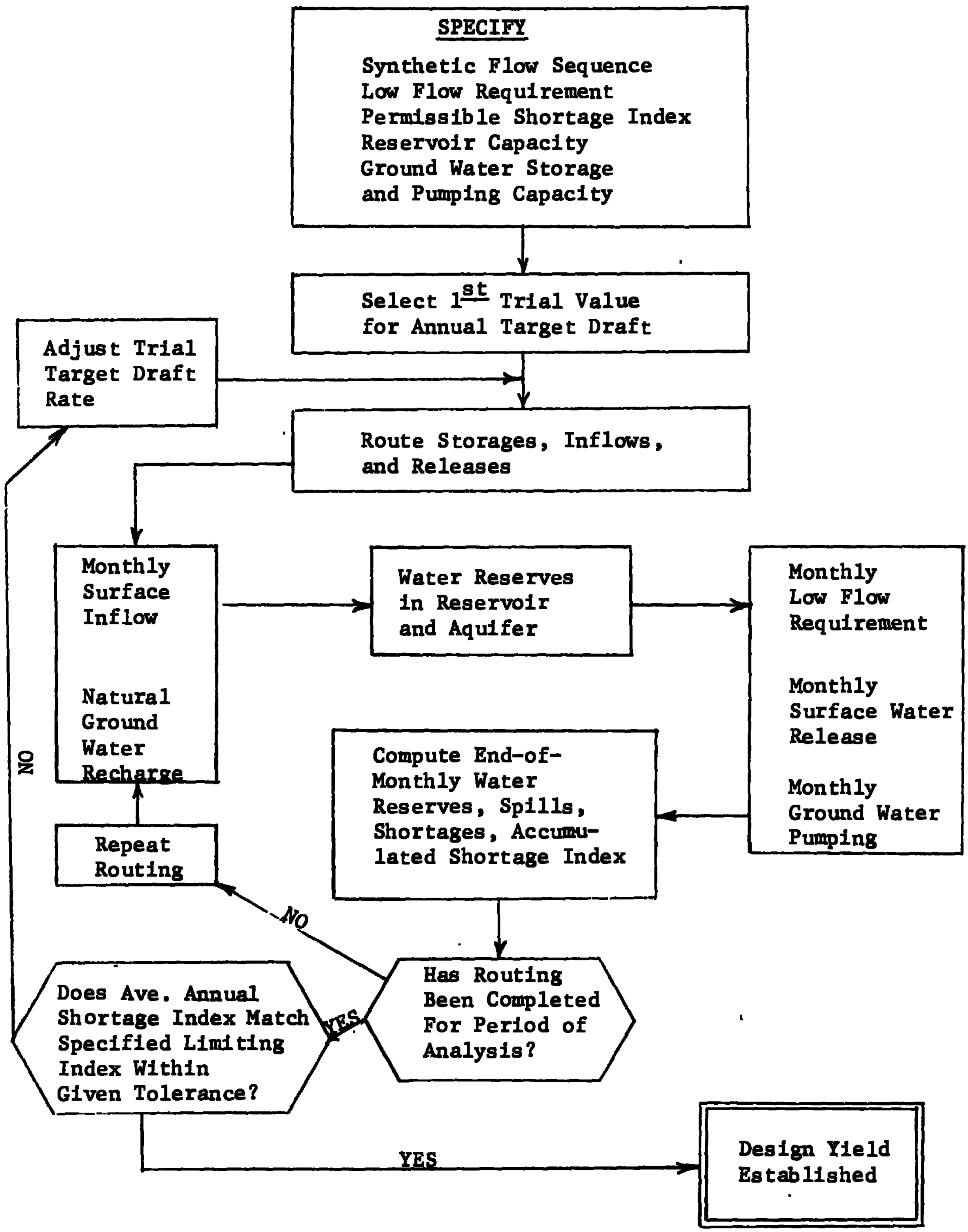

Fig. 4.5 Flow Chart for Design Yield Determination.

$$S_i = S_{i-1} + Q_i - LF_i - D_i \tag{4.2}$$

subject to the constraint

$$0 < S_i < S_{max} \tag{4.2a}$$

in which S_i = reservoir storage at the end of month i; Q_i, D_i and LF_i = volumes of reservoir inflow, target draft and required low flow release during month i, respectively, and S_{max} = storage capacity of the reservoir.

Monthly water supply target outputs, D_i, are fractions of the total annual target output, determined according to the typical relative demand schedule presented in Chapter 3.

Depending on the end-of-month storages in the reservoir, as determined by the routing equations, the following operational adjustments must be made. If at the end of a month the volume of water in the reservoir exceeds the maximum reservoir capacity, S_{max}, the excess must be spilled, and the storage volume is set equal to S_{max}. If the end-of-month reservoir storage volume is negative it represents a supply shortage which is recorded, and S_i is reset to zero.

Some assumptions directly pertaining to the reservoir routing were made, as follows:

1. The highest runoff volume in the northeastern part of the United States occurs during late winter and early spring. This seasonal distribution of runoff suggests that the 12-month period from May 1 to April 30 constitutes the most advantageous

routing year because the reservoir will be filled or at least nearly filled at the end of each routing year, and a quasi-independence of shortage amounts in successive routing years is assured.

2. The evaporation from the reservoir surface is assumed to be balanced by direct precipitation on the lake surface. This assumption seems justifiable in humid areas and for reservoirs with small surface areas, as pointed out by Frederick (1969).
3. Only during times when the reservoir inflow exceeds the predetermined 10-year frequency 1-month low flow limit can inflows be retained in the reservoir.
4. During the period of analysis the conservation storage space in a reservoir is time-invariant and not affected by reservoir sedimentation or reallocation according to multipurpose uses.

Routing a sequence of synthetic inflows and assumed target drafts through the reservoir over a span of N years results in a series of monthly shortages, from which the monthly shortage index can be calculated and totaled to produce a yearly shortage index value. Finally an annual average shortage index is calculated by dividing the sum of yearly shortage indices by the sequence length N.

Routing the same flow sequence with several different assumed annual target rates establishes a relationship between annual average shortage index and annual target draft. That target draft which corresponds to the chosen shortage index is consequently the corresponding design yield. As an example, such a relationship is presented in Figure 4.6 for hypothetical reservoirs A and B on the basis of a 50-year synthetic flow sequence. Both reservoirs have a capacity of 20,000 ac-ft but stream A (Muncy Creek) has a considerably larger sustained flow, and thus a higher design yield for any value of the shortage index. For the chosen yearly index limit of 0.05 the yields from reservoirs A and B are 23,500 and 19,200 ac-ft per year, respectively.

4.3.2 Choice of Synthetic Flow Sequence for Design Yield Determination

The generation of 1,000 years of synthetic stream flow, while reducing the uncertainty of drought prediction, introduced an unexpected dilemma. The consumption rate in a growing town will obviously increase with time. Routing this time-increasing demand schedule through a chosen sequence of synthetic flows would place an unduly high weight on the last years of this flow sequence, because shortages are much more likely to occur during these later years when the demand for water is highest. Therefore, a number of time-invariant target outputs were routed through various synthetic stream flow subsequences of different length.

The questions which the investigators faced were two: 1) How long a synthetic flow sequence should be used for the design yield determination, and 2) could a particular subsequence be chosen as representative of the entire 1,000-year flow sequence generated? In the interest of forecast reliability, it would have been best to use the entire 1,000-year sequence in each design yield determination; however, this would have resulted in excessive computer time use. The 1,000 years of synthetic flow were therefore divided successively into two 500-year, four 250-year, ten 100-year, twenty 50-year, and fifty 20-year subsequences. The design yields of reservoirs A and B with 5,000 ac-ft capacity were computed for all of these subsequences, and the results were plotted in Figure 4.7 for Reservoir B. The coefficients of variation of the yield (standard deviation divided by mean yield) exhibit a continuous decrease with increasing sequence length, as expected, but more significantly, the mean yield exhibits a pronounced decrease as the sequence length is increased from 20 years to 50 and 100 years, but levels off markedly between 50 and 100 years duration.

In the interest of conserving computer time, it was deemed adequate to use 50-year sequences for the yield determinations of the hypothetical system examined here. The relationship between yield and length of the hypothetical flow sequence might be quite different for other watersheds. These sequences should not be confused with the period of analysis of 50 years chosen. Only under the assumption of a time-invariant population can the yield determination sequences and the period of analysis be considered equivalent.

The individual design yields of the twenty 50-year sequences were compared for the purpose of choosing one sequence which would most closely represent the entire sample. The yields were plotted on Figure 4.8 and exhibit an essentially normal distribution which would be expected of yields based on subsequences of randomly generated stream flows.

After constructing curves similar to Figure 4.8 for reservoir sizes of 5,000, 10,000 and 40,000 ac-ft and comparing the deviations from the mean of the yields obtained for the twenty sequences, sequences 14 and 20 were chosen as being closest to the mean yield in reservoirs A and B, respectively. These 50-year sequences were then used exclusively in the yield determination of all the single and multisource water use combinations considered.

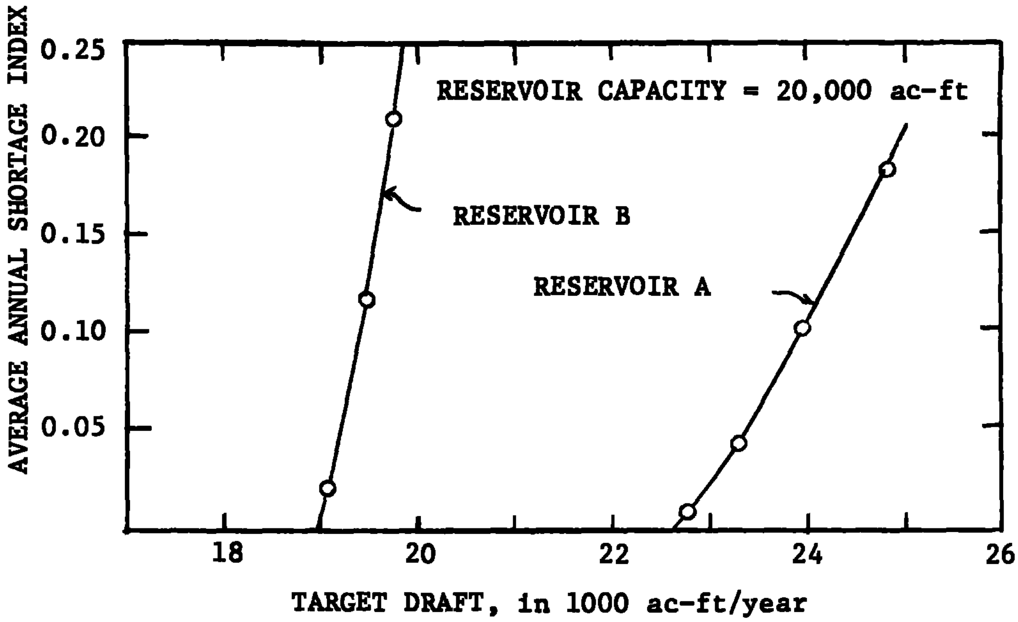

Fig. 4.6 - Example Relationships Between Annual Target Draft and Shortage Index, for two Reservoirs

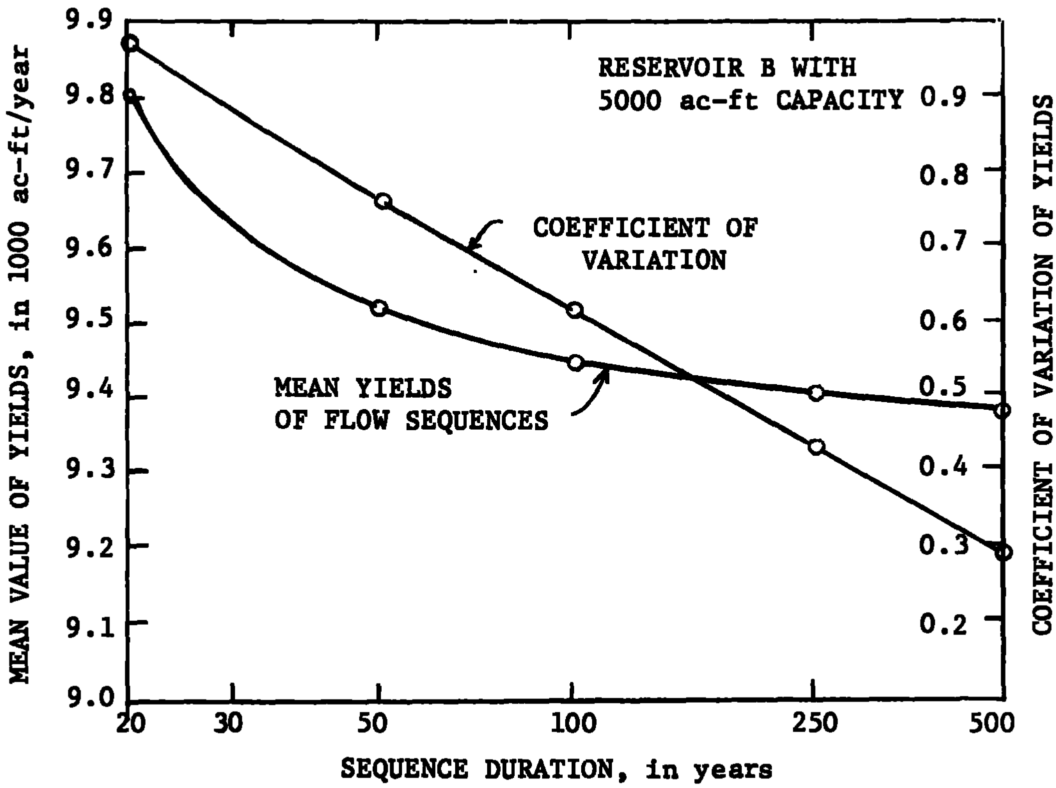

Fig. 4.7 - Effect of Sequence Duration on Design Yield

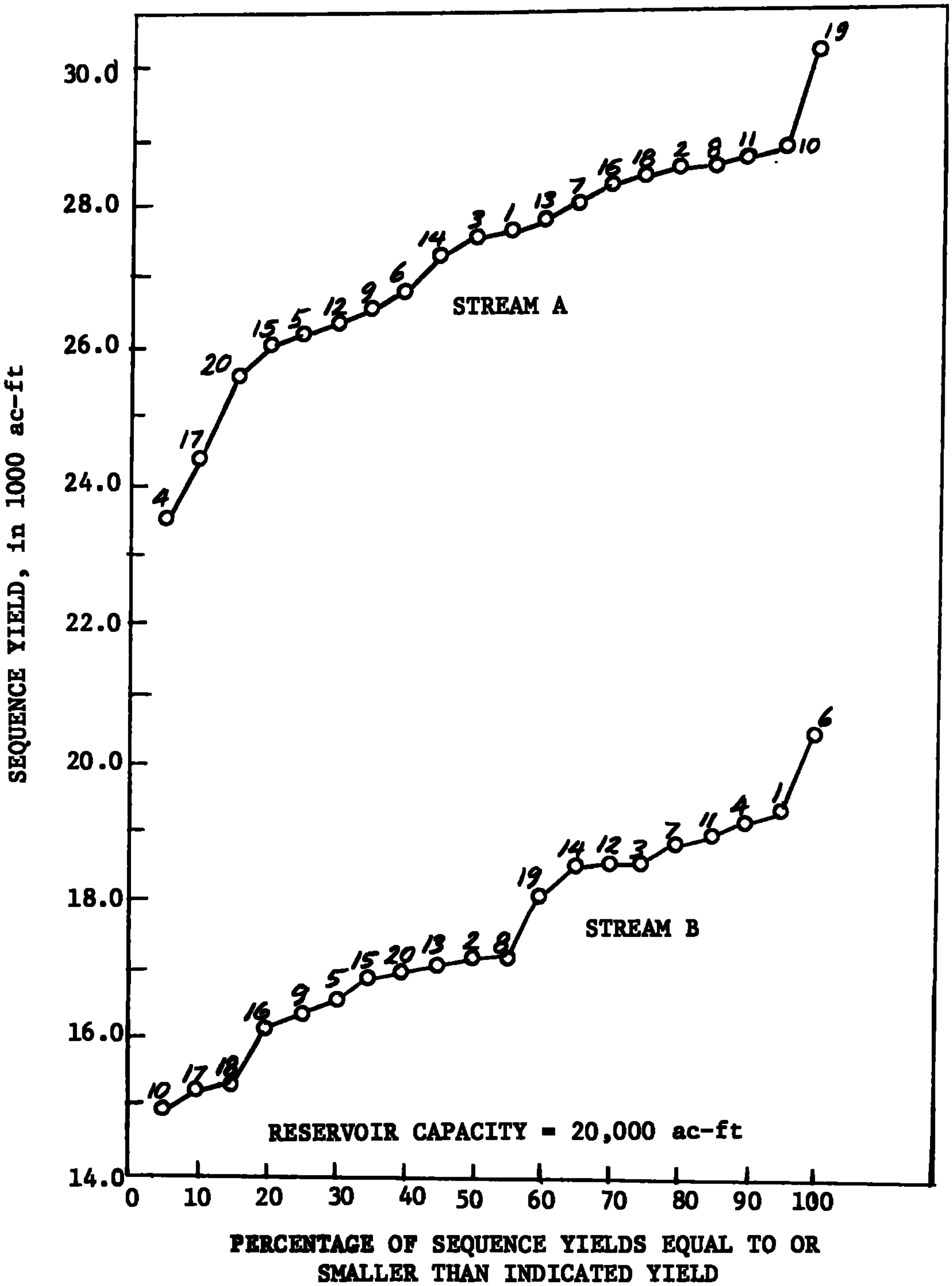

Fig. 4.8 - Sequence Yield Frequency Curve

4.3.3 Design Yields from Surface Reservoirs Without Ground Water Supplementation

Using the iterative routing technique outlined in Figure 4.5 with the selected 50-year synthetic flow sequences and the 0.05 average annual shortage index, design yield versus reservoir capacity relationships were found for the two reservoirs as plotted in Figure 4.9. The combined yield from the two reservoirs would simply be the sum of their individual yields for any combination of given reservoir capacities, because with the two reservoirs located relatively close together but on different streams, seasonal inflow fluctuations would have to be considered essentially equivalent.

4.3.4 Ground Water Pumping as Emergency Backup

The use of ground water merely to fill shortages in surface water supply when they arise could be termed the lowest degree of ground-surface water use integration. Referring to the flow chart in Figure 4.5, the use of ground water would be called for only when the reservoir does not have sufficient reserves to supply the consumer during the coming month. Such a backup system is easy to program but understandably ineffective unless the pump capacity is rather large relative to the projected consumption rate. Figure 4.10 shows how small wellfield pumping capacities increase the design yield effectively for very small reservoirs which have low design yields, as exclusive sources. For reservoir capacities larger than 10 times the monthly wellfield capacity, however, the yield increase due to the backup source becomes relatively minor, because the ground water reserve is called on too late to relieve the emergency effectively. A schedule of <u>preventive pumping</u> is therefore suggested as outlined below.

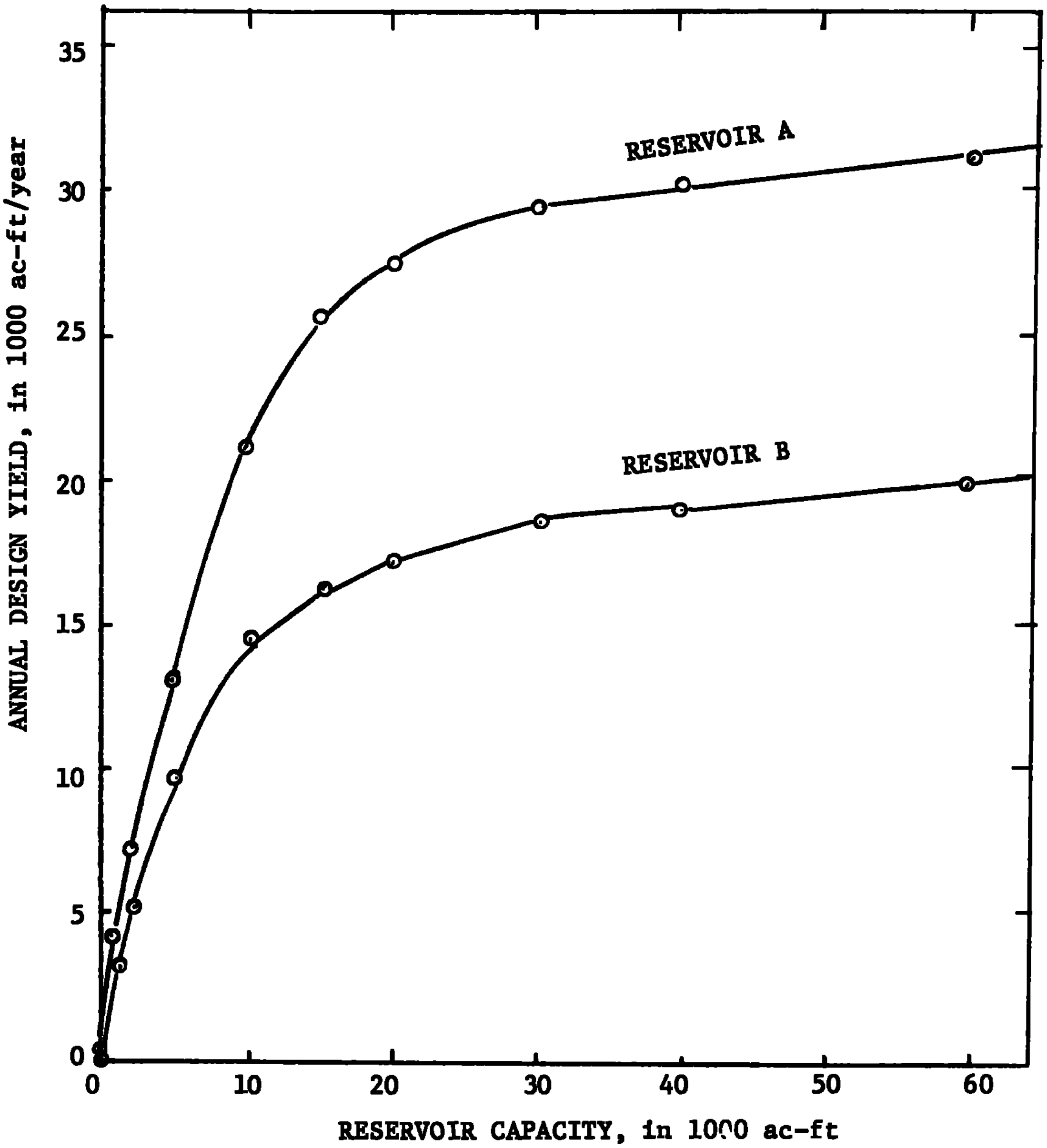

Fig. 4.9 - Relationship between Design Yield and Reservoir Capacity Without Ground Water Supplementation

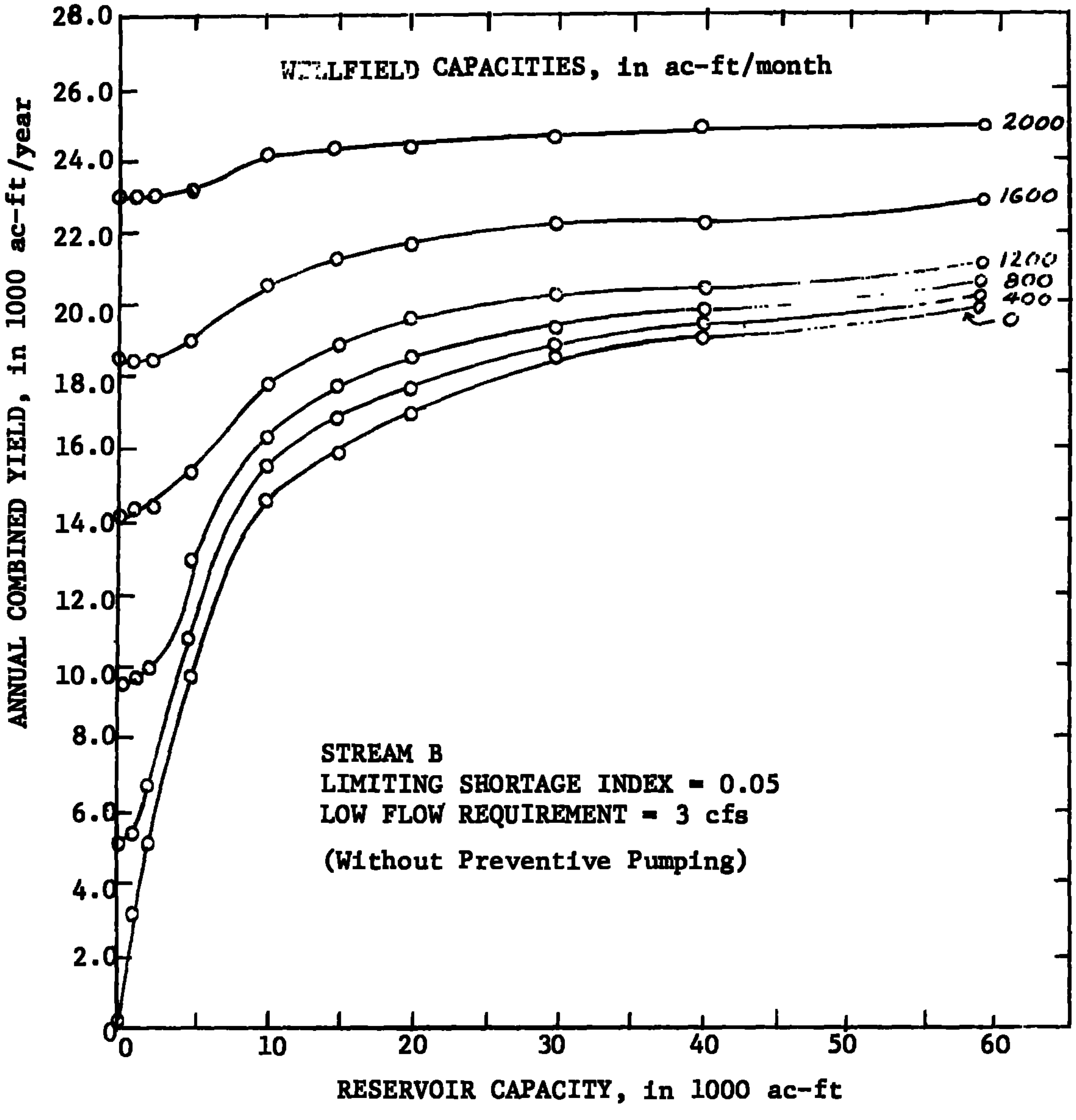

Fig. 4.10 - Annual Combined Yield for Stream B and Aquifer as Backup Source

4.3.5 The Concept of Preventive Pumping to Forestall Water Supply Shortages

As explained above, the policy of using ground water merely as backup supply whenever the surface reservoir is about to run out of water, is doomed to be an inefficient management procedure. Prudent water supply management would suggest beginning to pump ground water several months ahead of potential supply shortages and thus to provide for a carry-over storage in the reservoirs which, in conjunction with a moderate but steady ground water supply, will carry the demand center over the drought period.

Working with the "routing year" in which May is labeled as month No. 1 and April as month No. 12, a target carry-over storage can be computed for every month of the relatively dry season between June and December, or months Nos. 2 to 8, by the equation

$$CS_i = \sum_{n=i}^{8} (D_n - EXQ_n) - (8 - i)\ PMPC \qquad (4.3)$$

in which CS_i is the target carry-over storage at the beginning of month i, D_n is the target draft during month n, including mandated low flow releases, EXQ_n is the expected reservoir inflow during month n, and PMPC is the monthly wellfield pump capacity.

This target carry-over storage can be computed for a given reservoir, once the target releases, pump capacity, and expected inflows are decided upon. It can then serve as an operator's guide on a month-to-month basis. Whenever the water volume in a reservoir at the beginning of a month is less than the target storage, the decision would be made to supply the difference by ground water pumping, up to the limiting pump capacity, PMPC.

The factor EXQ, or expected reservoir inflow, is the variable which determines the degree of backup service the ground water source will provide. Thus the term degree of preventive pumping was conceived, defined as the ratio between long-term average to expected reservoir inflows. The higher the degree of preventive pumping, the more conservative is the operator's rule, namely the lower the expected inflows. The choice of a low degree of preventive pumping will reduce the role of ground water as a supply source and increase the risk of shortages during unexpected droughts. A very high degree of preventive pumping, on the other hand, may lead to excessive pumping of ground water and encroachment on the aquifer reserves while not fully utilizing the available storage in the reservoir, except in extreme drought years.

In Figure 4.11 the target carry-over storages are plotted for the projected water use of 14,950 ac-ft/year by the hypothetical demand center from reservoir B, using different degrees of preventive pumping. Understandably, the target carry-over storage is highest in the first potential drought month, June, and decreases to zero in January.

The operator is of course free to adjust his pumping schedule in the middle of a month if a major storm or severe drought changes the storage situation relative to beginning-of-month conditions.

To demonstrate the effectiveness of preventive pumping in firming up the system's reliable supply, design yields based on the 0.05 shortage index limitation were computed for a sample reservoir of 5,000 ac-ft capacity and pumping capacities of 200, 500 and 1,000 ac-ft/month. The resulting combined yields, as shown in Figure 4.12, increased drastically as the degree of preventive pumping was increased from 1 to 5, beyond

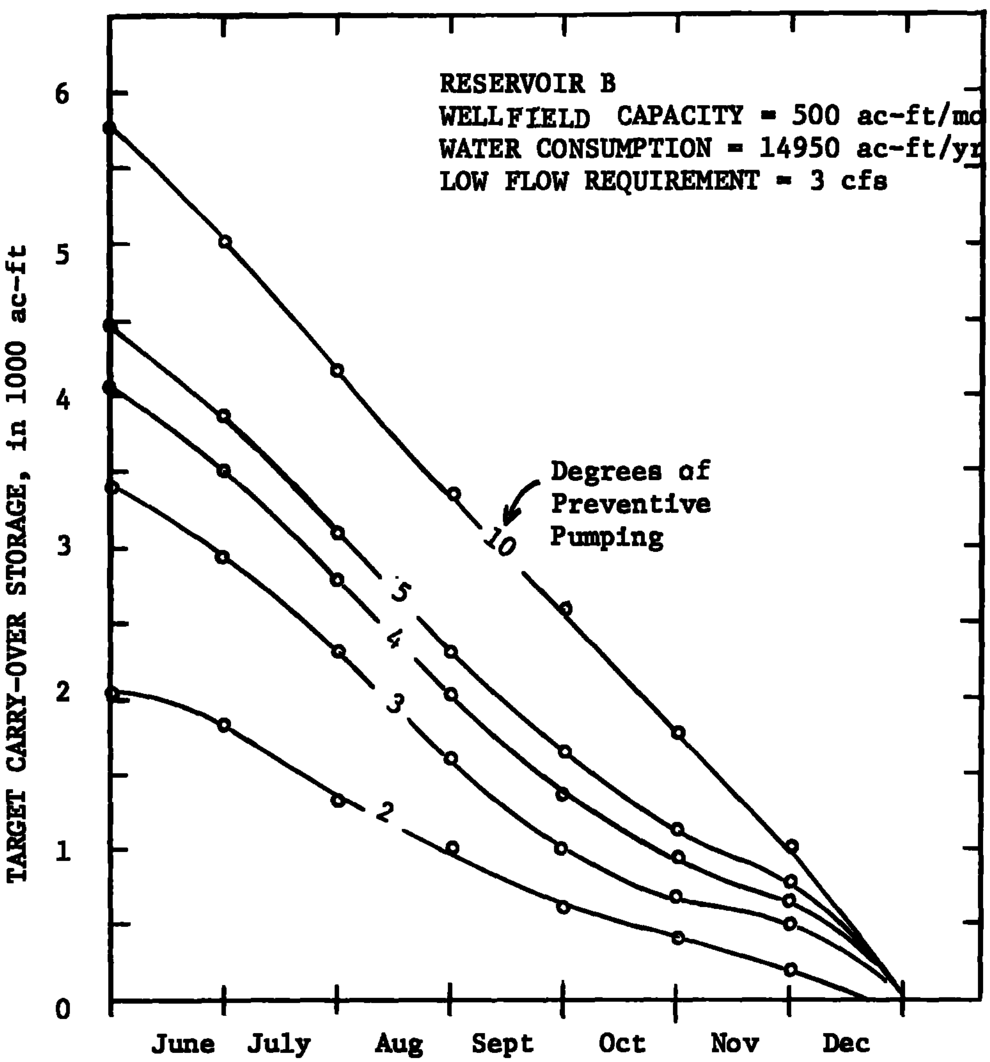

Fig. 4.11 - Target Carry-Over Storage Schedule as a Function of Degree of Preventive Pumping

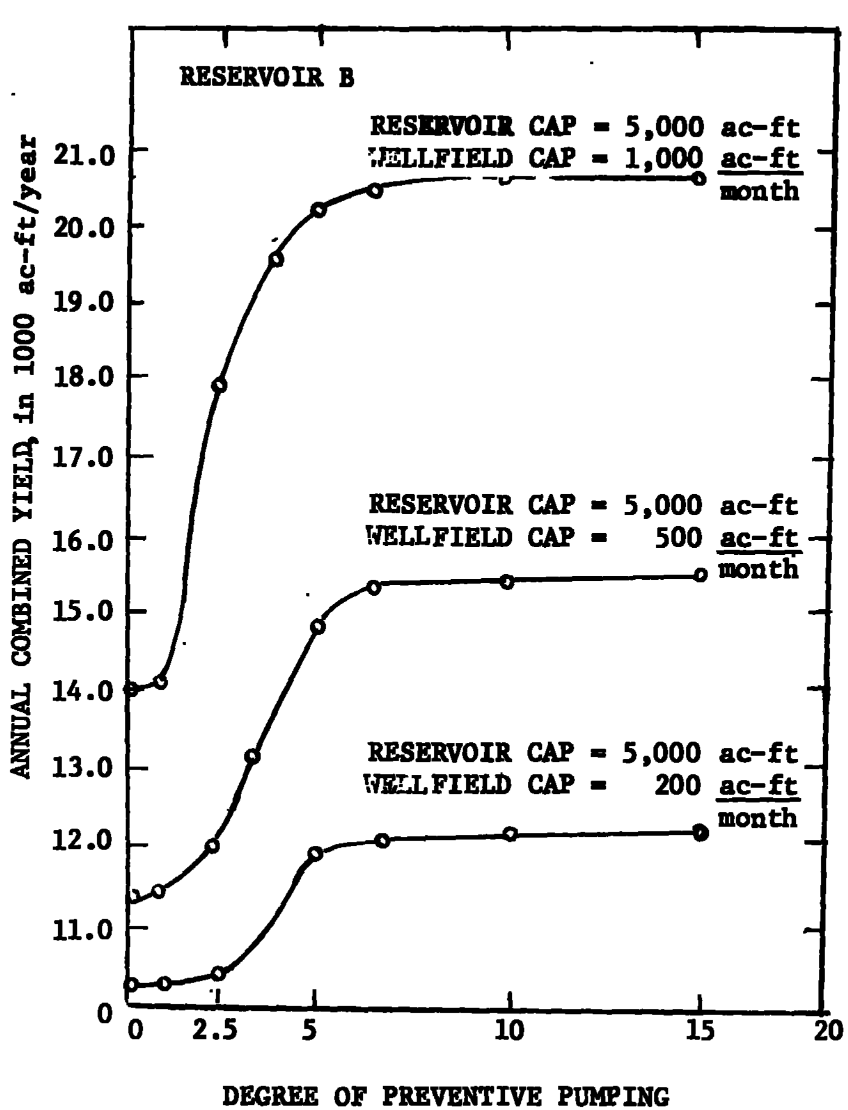

Fig. 4.12 Relationship Between Annual Combined Yield and Degree of Preventive Pumping

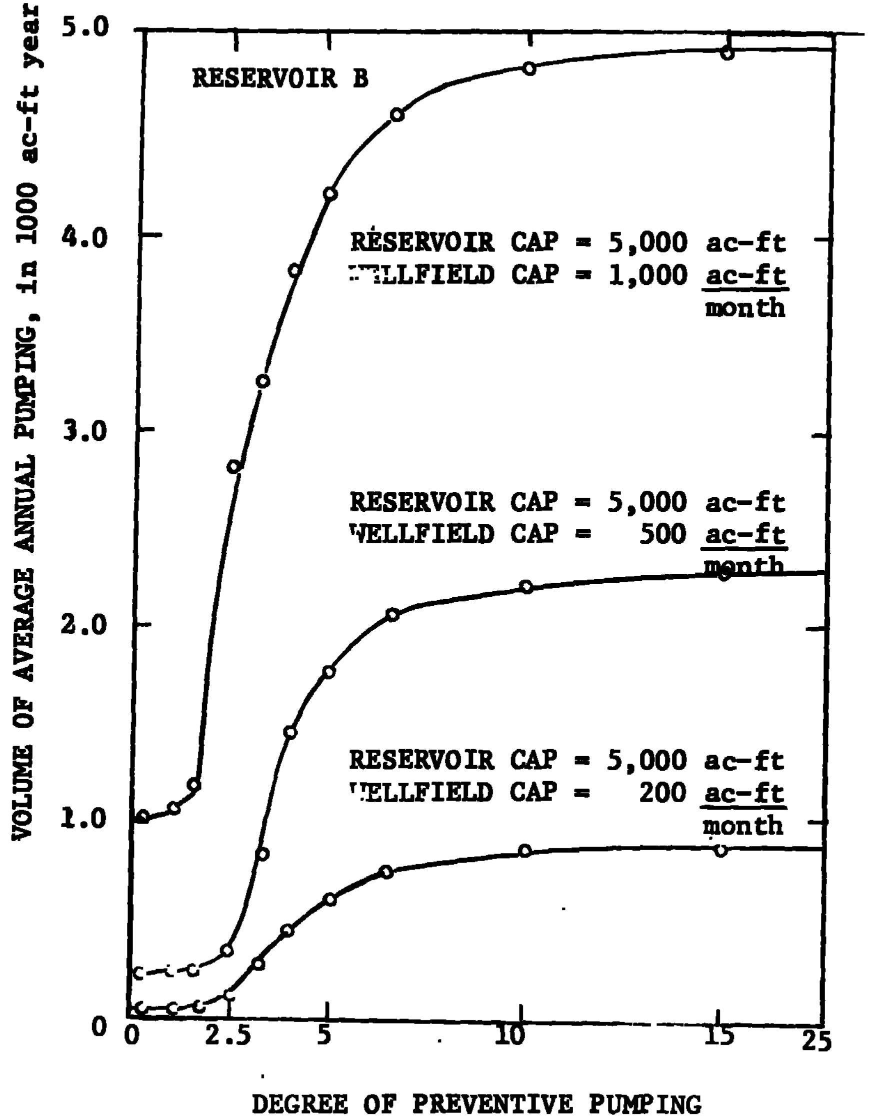

Fig. 4.13 Relationship Between Volume of Average Pumping and Degree of Preventive Pumping

which the effect began to level off. Very similar in shape are the curves of average annual ground water volumes pumped, which are shown in Figure 4.13. In the cost calculations, which are described in Chapter 5, the optimal value of the degree of preventive pumping, based on the criterion of minimal total cost, was roughly 5 for both reservoirs A and B.

On Figure 4.14, the combined annual yields from reservoir B and the aquifer operating under the preventive pumping rule are plotted as a function of reservoir and wellfield pump capacities. A second set of curves shows the average annual ground water volume pumped under the preventive pumping rule.

In comparison to Figure 4.10 a considerable increase in combined yield due to preventive pumping can be observed.

4.3.6 Yield from Aquifer as Exclusive Source

In determining the design yield from an aquifer as an exclusive or independent water source, month-by-month routing of stochastic inflows and scheduled releases is not deemed necessary as long as the annual withdrawals do not exceed the average annual replenishment. Under these conditions, the aquifer's annual design yield is merely proportional to the monthly wellfield pumping capacity PMPC.

The wellfield capacity must be sized to supply the maximum day demand, which in turn is equal to the average daily demand over the year times a ratio RMA of maximum to average day consumption, which in Chapter 3 was given a value of 2.2. Combining these relationships, the annual aquifer design yield can be expressed as

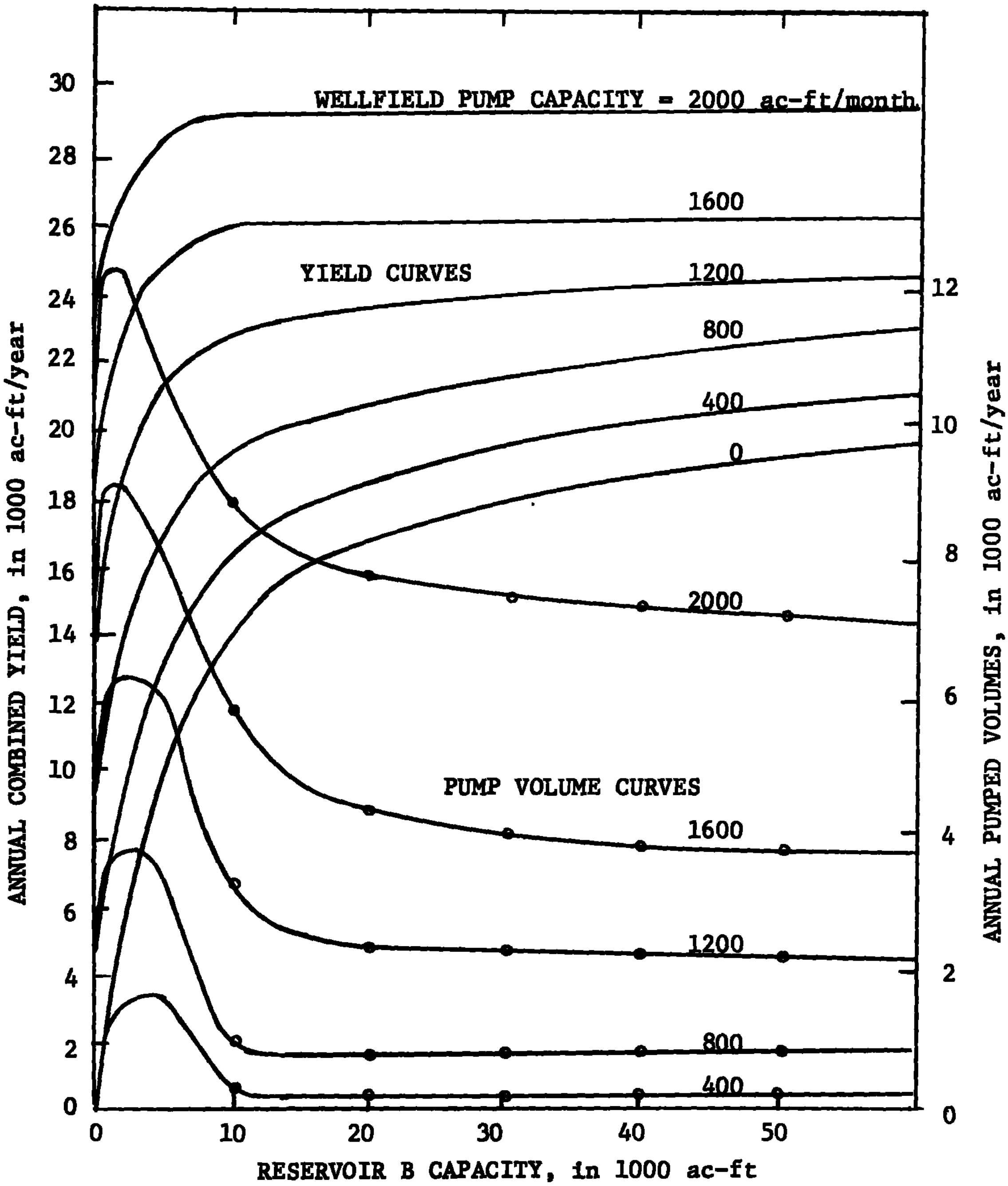

Fig. 4.14 Annual Combined Yield and Ground Water Volume Pumped for Reservoir B and Aquifer under Preventive Pumping Rule

<u>Conditions</u> Limiting Shortage Index = 0.05
Low Flow Requirement = 3 cfs
Degree of Preventive Pumping = 5.0

$$AY = \frac{12 \text{ PMPC}}{\text{RMA}} \tag{4.4}$$

or 5.45 PMPC, if the 2.2 value is introduced.

If the annual consumption rate approaches the average annual replenishment rate, the routing procedure outlined in Figure 4.5 should be used with all surface components set to zero. Under these critical conditions where a depletion of the ground water reserves is possible, it should also be attempted to determine a relationship between replenishment and annual rainfall as well as average water table elevation. This functionality is, however, very difficult to estimate without an excellent data base. The use of the average annual replenishment plus the safety factor associated with the limitation of a maximum permissible drawdown should be adequate if these detailed aquifer recharge relationships cannot be obtained.

4.3.7 Effectiveness of Surface and Ground Water Supply Integration

After a method for determining individual and combined design yields was developed and yield curves plotted for a hypothetical water supply system, the effectiveness of integrated use was investigated by comparing the combined yields from integrated use with the sum of the yields from individual systems of equal capacities but operating separately.

Figure 4.15 shows the gain in yield of reservoir B through the ground water backup supply, or the difference between combined and individual reservoir yields. The straight line for nonintegrated aquifer operation is a plot of Equation 4.4. The four curves for reservoir sized of 5,000 to 30,000 ac-ft capacity demonstrate that for any given pump capacity

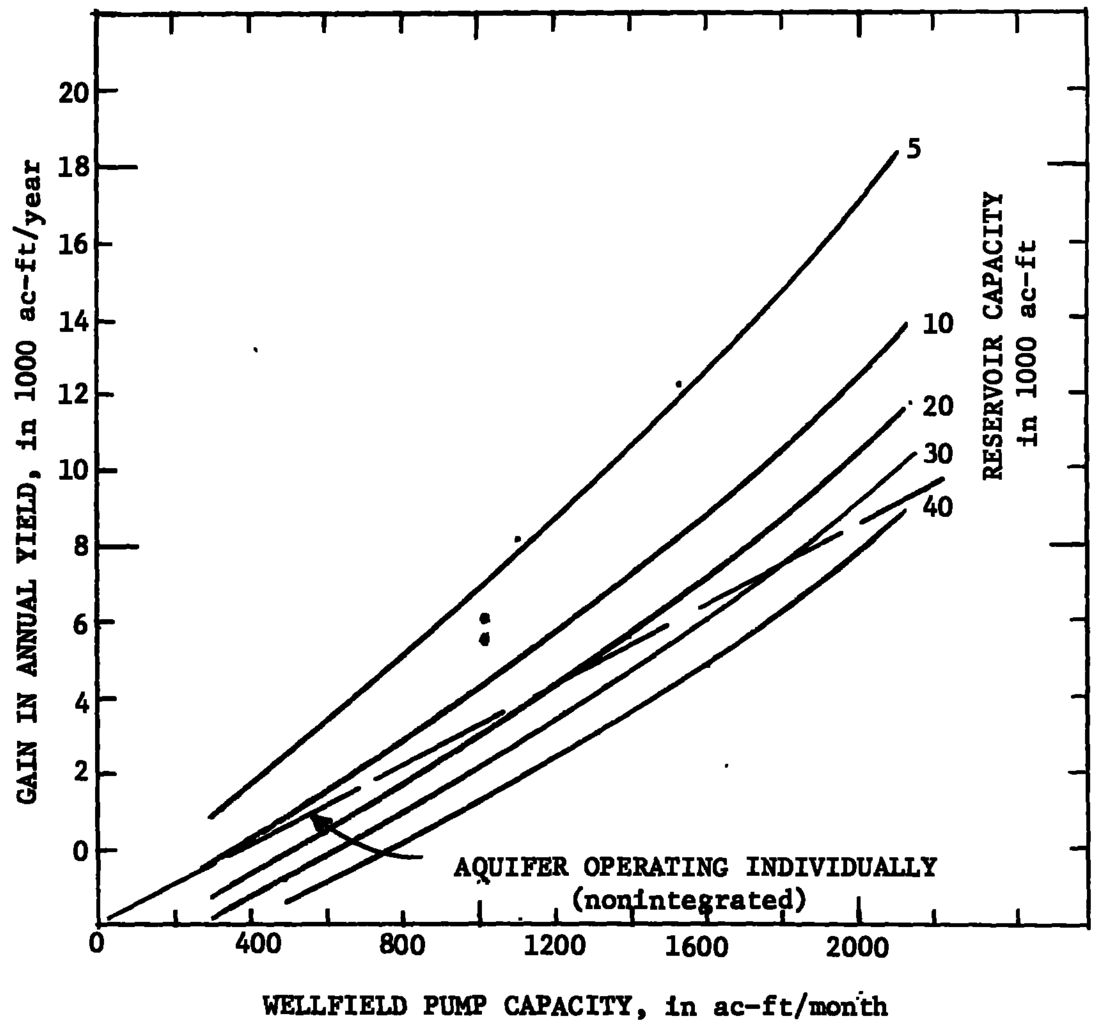

Fig. 4.15 Effect of System Integration on Annual Yields

integration is most effective if the reservoir conservation storage is small. This conclusion is logical because a larger surface storage has the capacity to capture the seasonally fluctuating reservoir inflows and release the water according to the demand center's consumption schedule, without requiring much supplementation from the aquifer.

It may seem strange that, in general, for reservoir storage volumes larger than 15 times the monthly wellfield pump capacity, the individually operating systems can provide a higher design yield than the same systems under integrated operation. This situation arises because under the integrated rule the aquifer is merely a stand-by source, thus operating under a much lower load factor than the ratio of average to maximum day water demand which is the load factor of the separately operated aquifer. A different picture is obtained if the same gain-in-yield comparison is made for equivalent volumes pumped. This comparison illustrated in Figure 4.16, shows that no matter how large the reservoir, integrated operation provides a larger yield gain than separate operation, and conversely, given a certain required yield, the integrated system conserves water more effectively.

Which of the two gain comparisons is more relevant depends on whether the main problem is one of economics or water shortage. If the capital costs of system installation seem predominant, the gain comparison on the basis of equal system capacity is appropriate. If on the other hand, the operating costs of pumping the ground water are a major factor and/or overall water sources are scarce, it would be wise to make use of the abundance of surface waters in the spring and let the aquifer recover for a few months, even if this scheme would require larger investment costs.

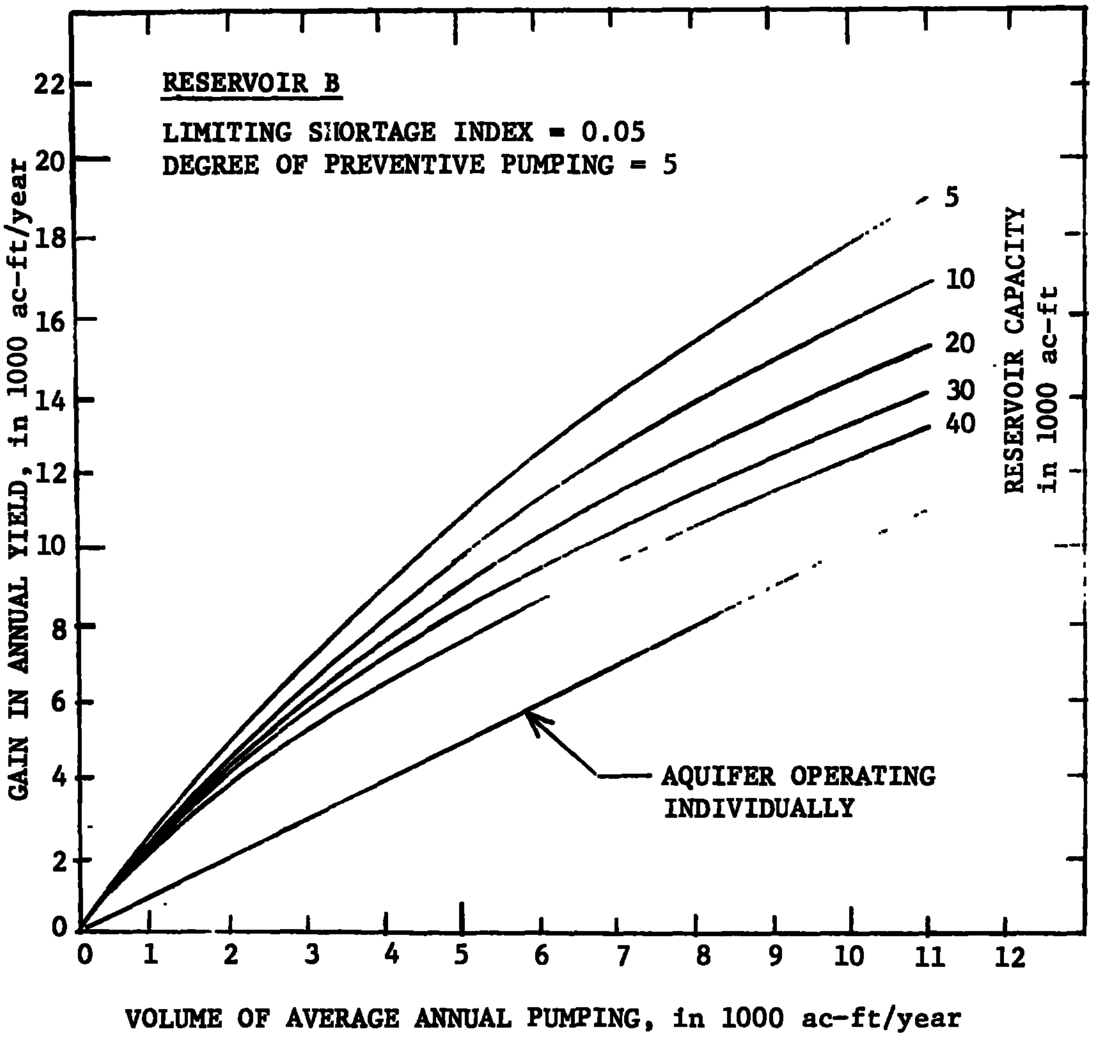

Fig. 4.16 Gain in Annual Yield Due to Volume of Ground Water Pumped

Although it is now obvious that an aquifer operated in coordination with a reservoir can produce higher yields than an aquifer operated as a single source, the conclusion that an integrated use scheme is necessarily also economically superior to single source development is premature. Whether it is economically advantageous or not remains to be tested by least-cost analysis, which will be treated in Chapter 5.

4.4 Determination of Ultimate Capacities for Several Supply Alternatives

In Stage 2 of the general methodology the yield curves developed in the previous section were used to determine the ultimate capacities needed for the single and dual source combinations listed at the beginning of this chapter. These are the capacities needed to satisfy the demand center's water requirements during the last year of the planning horizon. Time-of-supplementation curves were also developed. All numerical results were derived from the hypothetical example system defined in Chapter 3.

A prerequisite for all of the following calculations is the knowledge of the water demand center's schedule of average annual and maximum day water consumption rates. For the example demand center (initial population = 40,000, annual growth rate = 2.5 percent of the initial population), the population will have grown to 89,000 after 50 years. At that time the annual volume of water consumption will be 14,950 ac-ft, and the maximum daily volume of consumption will be 90 ac-ft. The annual distribution of population, total water consumption, and maximum day consumption are shown in Figure 4.17.

4.4.1 Alternatives Involving a Single Source

For alternatives relying only on one source the ultimate size of the supply system is easy to determine. The ultimate capacity must be designed

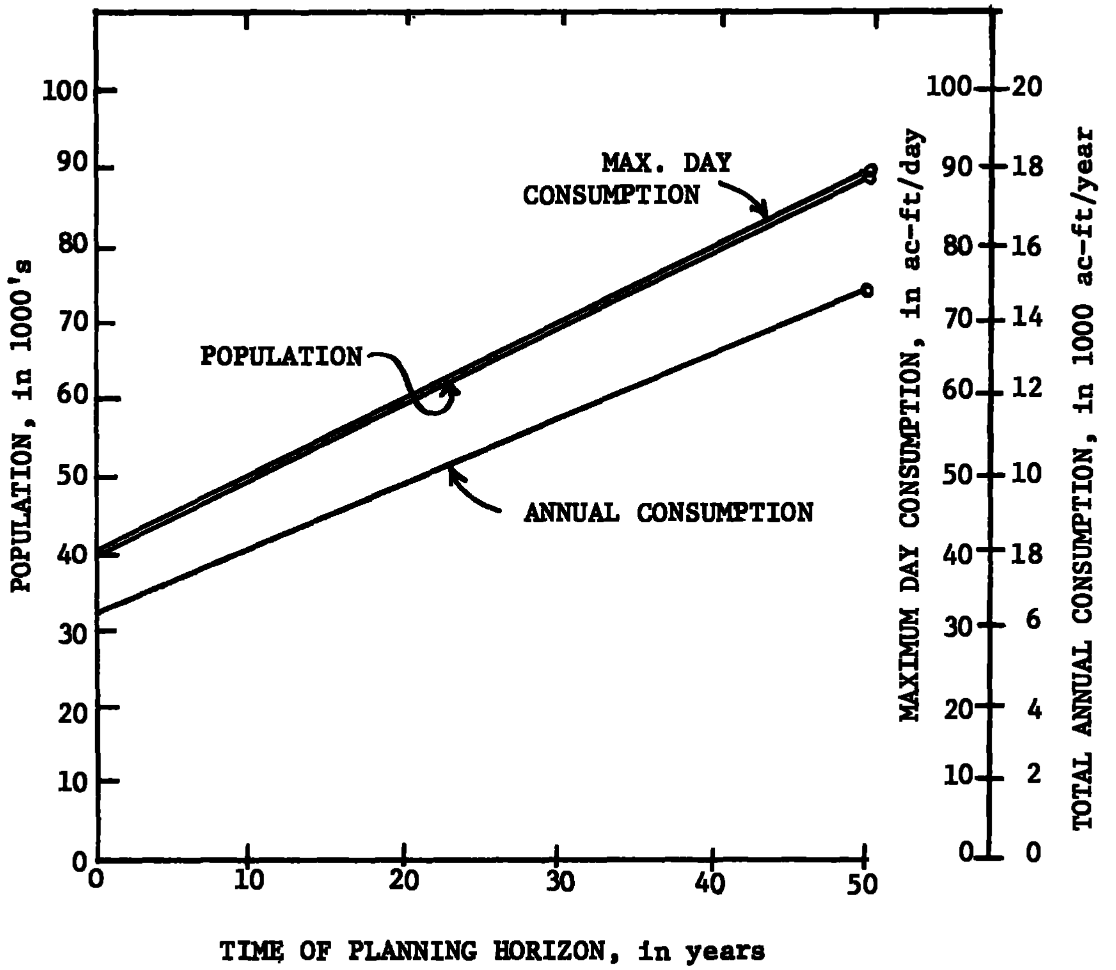

Fig. 4.17 Distribution of Population, Maximum Day Consumption, and Total Annual Consumption of Example Demand Center

such that it can supply the demand center's water consumption in the last year of the planning period.

For alternatives IA and IB as defined in Table 4.1, the necessary reservoir capacities are determined from Figure 4.9. Reservoir A must be built to 6,050 ac-ft, whereas reservoir B should have a size of 10,090 ac-ft to guarantee the water consumption of the hypothetical demand center throughout the time of the planning horizon.

Alternative IR, direct supply from river without storage, is also feasible since the river design yield of 30,500 ac-ft per year exceeds the demand center's water requirements at all times.

According to Equation 4.4 the aquifer pumping capacity needs to be developed up to a maximum wellfield pump capacity of 2,740 ac-ft/month in order to supply the ultimate consumption under alternative I AQ (aquifer supply only). The ultimate water consumption of 14,950 ac-ft per year, however, exceeds the assumed average annual replenishment of 11,200 ac-ft. The resulting deficit pumping of 3,750 ac-ft in the 50th year is equivalent to a water table drawdown of about 4 ft per year in the latter years of the planning horizon, which approaches the drawdown limitation. An individual analysis should be made in this case to decide whether such a mining strategy is prudent in the long run.

The procedures for determining possible source combinations and time of supplementation curves are different for alternatives involving two surface water sources and for alternatives involving one ground water and one surface water source.

4.4.2 Supply from Two Surface Reservoirs

A yield isoquant is shown in Figure 4.18 for supplying the ultimate demand of 14,950 ac-ft per year from the two surface reservoirs A and B

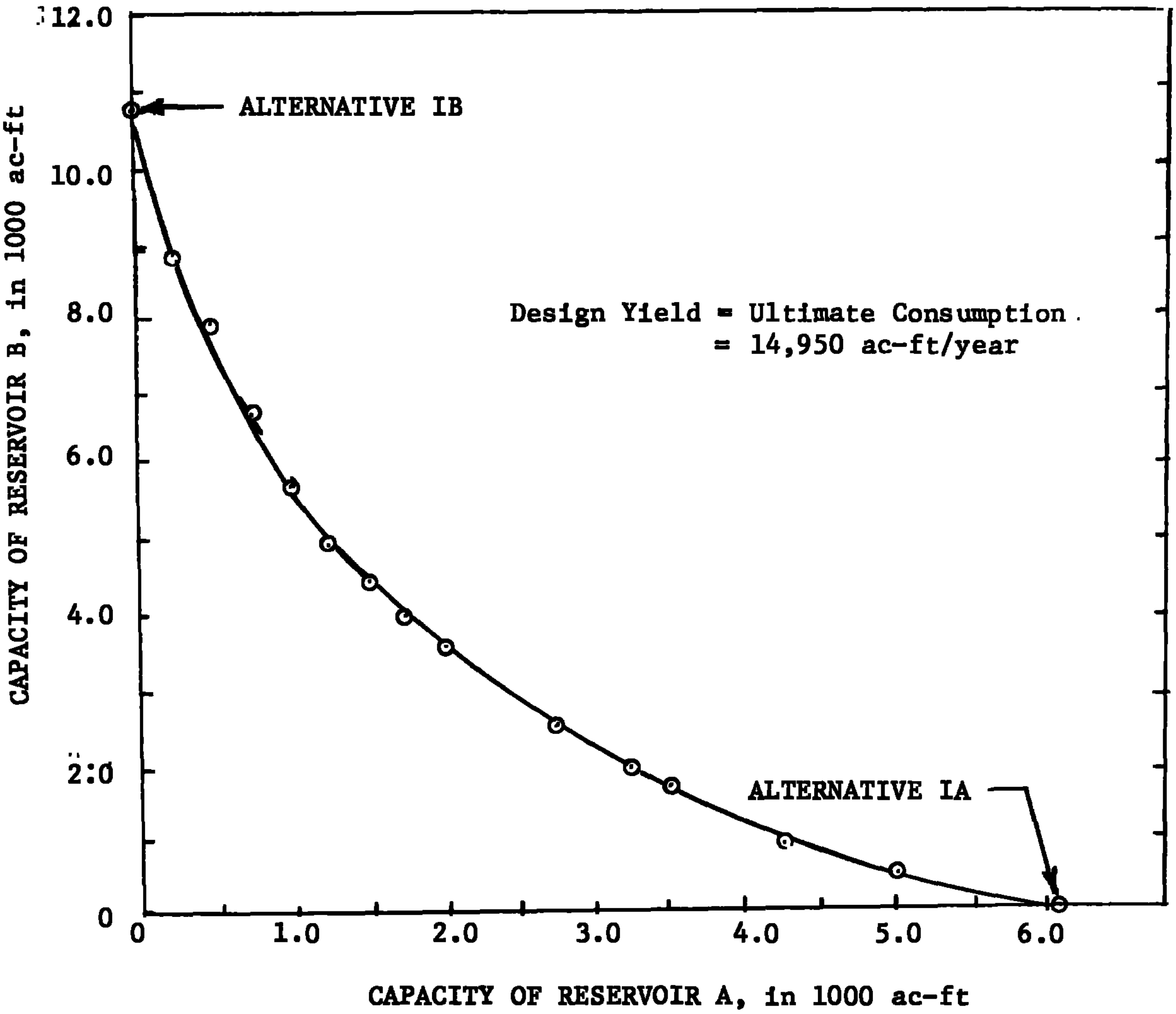

Fig. 4.18 Isoquant Line Describing Feasible Size Combinations for Two Reservoirs

was constructed directly from the individual yield curves in Figure 4.9. As mentioned earlier, the two reservoirs are located relatively close to each other, and their inflows were assumed to be subject to the same seasonal and annual fluctuations. Thus, the conjunctive operation of the two reservoirs does not offer any water conserving advantages and the combined yield equals merely the sum of the individual design yields. The isoquant line for the two reservoirs is thus constructed as a range of reservoir sizes whose yields, as determined from Figure 4.9, add up to 14,950 ac-ft per year.

In conjunction with the yield isoquant for ultimate target draft a set of curves for the time of supplementation is provided in Figure 4.19. These curves indicate the year in the planning period when the reservoir built first needs to be supplemented by the second reservoir, by river water, or by ground water. The time of supplementation for a combination within alternative IIA-B (reservoir A built first and reservoir B second) is found as follows: The yield from a selected capacity of reservoir A is sufficient as long as it is larger than the annual volume of water consumption of the demand center. The year in which the projected annual consumption exceeds the design yield of reservoir A is equivalent to the time of supplementation or the year when reservoir B must be completed. For example, if reservoir A is built to a capacity of 4,000 ac-ft, reservoir B would have to be built by year 30. The time of supplementation for alternative II B-A is determined according to the same principles. However, the curves II A-B and II B-A are different. Because of the larger natural inflow into reservoir A, this reservoir can assure a larger yield than reservoir B for equal capacities.

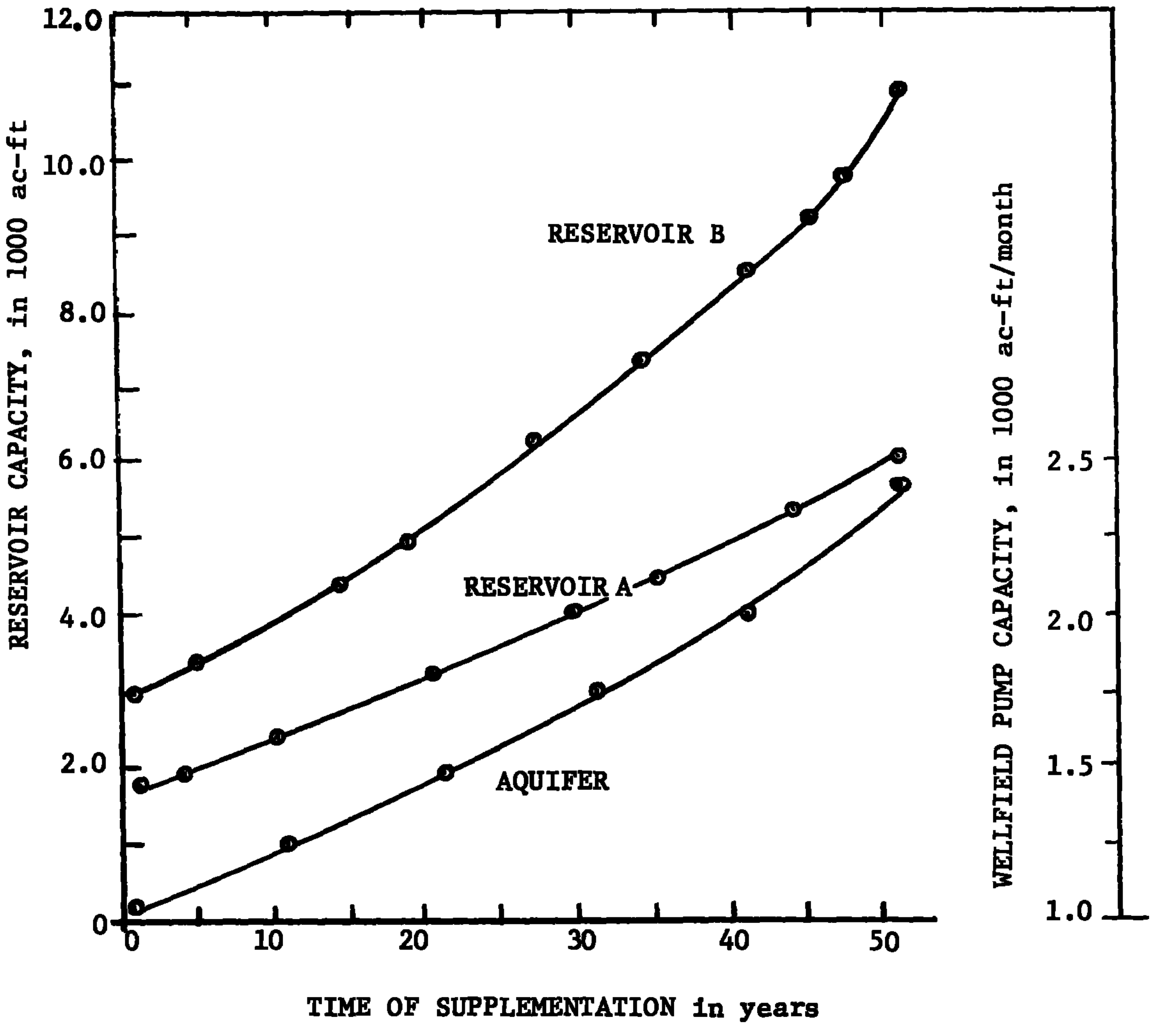

Fig. 4.19 Time of Supplementation for Reservoirs and Aquifer

4.4.3 Alternatives Involving One Surface and One Ground Water Source

Isoquant lines for combined surface and ground water supply are shown in Figure 4.20. These are constructed from a combined yield relationship such as shown in Figure 4.14 and the demand center's ultimate water requirements, as follows: Select a reservoir size and enter Figure 4.14 with both the reservoir size and ultimate water consumption to find the corresponding monthly wellfield capacity by interpolation. Note that in Figure 4.10 there are two combinations shown for zero reservoir capacity. The point falling in the isoquant line involves pumping from the aquifer and pumping directly from the undammed stream. The single point at 2,740 ac-ft monthly wellfield capacity represents the case of exclusive ground water supply (alternative I A-Q).

The time-of-supplementation curves will vary depending on whether the reservoir or the aquifer is developed first. For the case of starting with the reservoir as initial source, time-of-supplementation curves are determined in exactly the same way as for the two-reservoir case. Time of supplementation curves for reservoirs A and B are shown in Figure 4.19.

For the case of developing ground water first, a monthly pump capacity is selected and the corresponding yield is computed by Equation 4.4. Comparison of this yield with the schedule of annual water consumption in Figure 4.17 gives the time of supplementation by any one of the surface sources, which is likewise plotted in Figure 4.19.

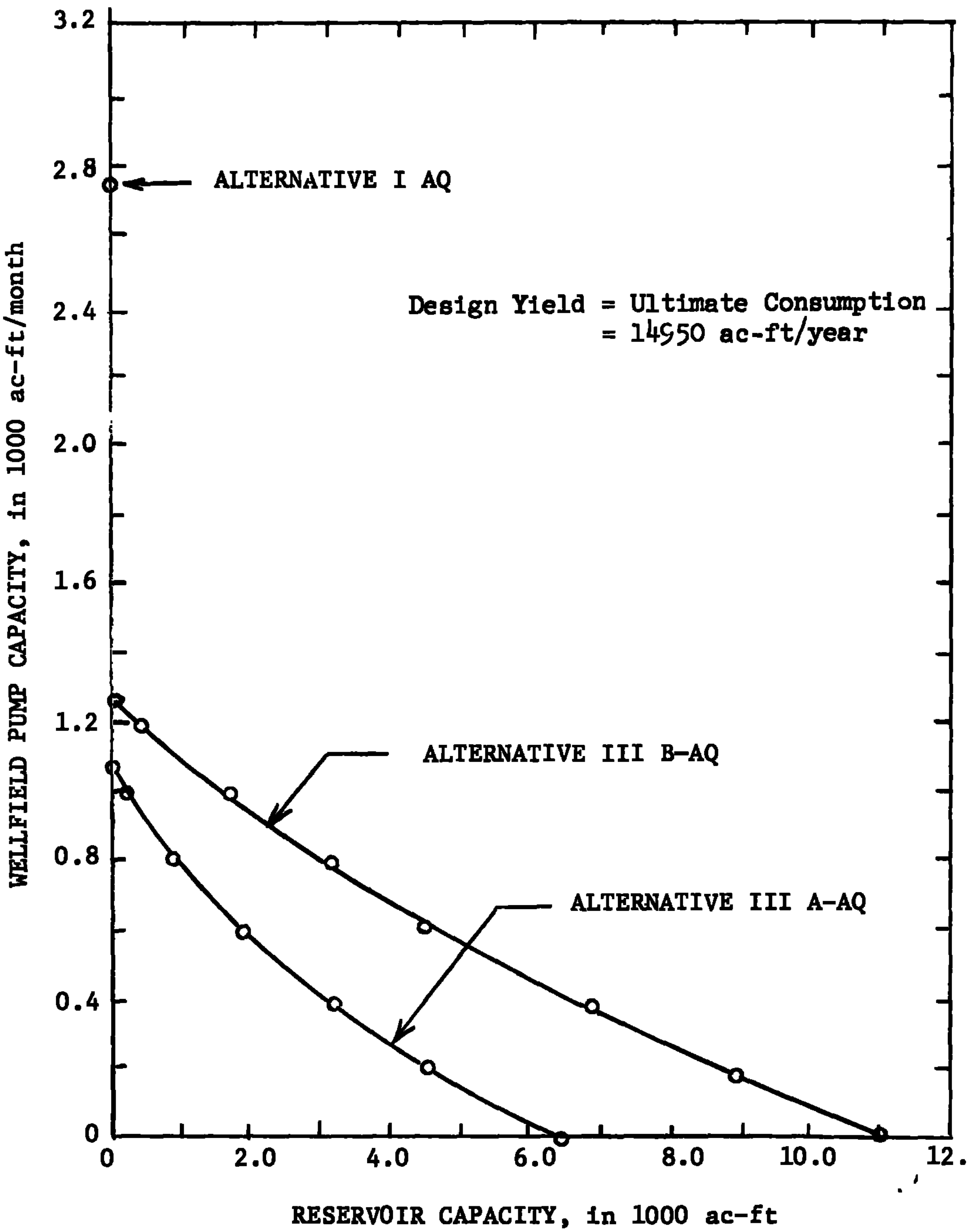

Fig. 4.20 Isoquant Lines Describing Feasible Source Combinations for Surface-Ground Water Systems

4.5 Summary

In this chapter the procedures to determine yield and the required sizing of alternative system combinations were described. The system capacities and flow volumes needed to determine the design flow schedules can be developed from the type of curves illustrated in Figures 4.9 to 4.20.

CHAPTER 5

DESIGN FLOW SCHEDULES AND COST DETERMINATIONS

The product of the procedures described as stages 1 and 2 in Chapter 4 is a set of annual yields for all single or dual source supply systems chosen for consideration, as well as system capacity combinations that will satisfy the delivery requirements in the last year of the planning period, and the times of supplementation for those dual systems in which one source is installed to satisfy only the relatively low demands during the early years of the period. For the purpose of eventually arriving at a least-cost system, a range of feasible alternatives developed in stage 2 is chosen in stage 3, and the decision variables needed for the cost determination in stage 4 are quantified.

The output of stage 3 is a series of monthly and annual schedules of these variables, including peak delivery requirements, flow volumes to be conveyed, boosted, and treated to specified levels, and energy requirements for the corresponding pumping boosting activities.

5.1 General Description of Supply Component Quantification and Cost Calculation Procedures

The design flow schedules developed in stage 3 are specific for each alternative or source combination and consist of two components: year-by-year schedules of maximum day consumptions and annual schedules of supply volumes.

Design schedules of maximum day flow specify the maximum daily capacity a particular water supply unit must have in any one year during the planning horizon in order to fulfill its role in an alternative source combination.

This information is needed to design for peak capacities and must be available for every year since it provides the basis for proper stage construction. Design schedules of supply volumes specify the flow quantities passing through each water supply unit during each year of the planning horizon. These schedules are needed for calculation of operation and maintenance costs. Annual flow volumes should suffice for a preliminary cost estimate and the selection of the economically most promising systems; for a refined cost estimation, however, supply schedules might need to be computed on a monthly basis.

The total cost for each water supply alternative is the sum of the costs associated with several cost centers. In general, these can be defined for a basic system to include:

1. Reservoir
2. Water treatment
3. Surface water transportation
4. Aquifer development, pumping, and ground water transportation.

The boundaries of each cost center and the components within those boundaries should be defined so that design and economic considerations for each cost center are as independent as possible from every other cost center. With the system decomposed in this manner, each cost center can be optimized independently without the fear that the ultimate result will be a suboptimal solution. This approach is easier computationally and also allows the planner to focus on well-defined portions of the total problem. For example, the reservoir storage requirements are determined according to the shortage index and synthetic hydrology discussed previously. The cost of reservoir storage can be calculated directly as a function of this storage requirement.

The next cost center would logically be surface water transportation which is, in essence, independent of the considerations that dictate the capacity and cost of reservoir storage. However, the components in this surface water transportation cost center are not independent of each other. For example, the design capacity and cost of a pumping station are very much related to the diameter of the pipelines through which the water is to be transported. Therefore, these two components (i.e. pumps and pipelines) must be within the same cost centers, and the least-cost subsystem within the cost center will involve tradeoffs between at least these two components.

The cost centers identified for the hypothetical systems analyzed here might need to be redefined for other systems with different physical characteristics but, nonetheless, do illustrate the advantage of this approach. All system costs presented in this chapter are based on generalized planning equations listed in Appendix A. A detailed discussion of computational procedures and assumptions for design calculations and cost analyses for each cost center is given in Appendix B. All calculations were performed by specially written computer programs which are not included with this report because of their length and because of their special purpose nature. The primary motivation for computerizing the analysis was to make possible the extensive sensitivity analysis described in Chapter 6 which follows. The calculations for each cost center were directed by a main program according to the shcematic outline in Figure 5.1.

5.2 Design Flow Quantities and Costs by Alternative

The combined delivery capacity of the supply system in any year must at least equal the maximum day consumption rate as plotted in Figure 4.17.

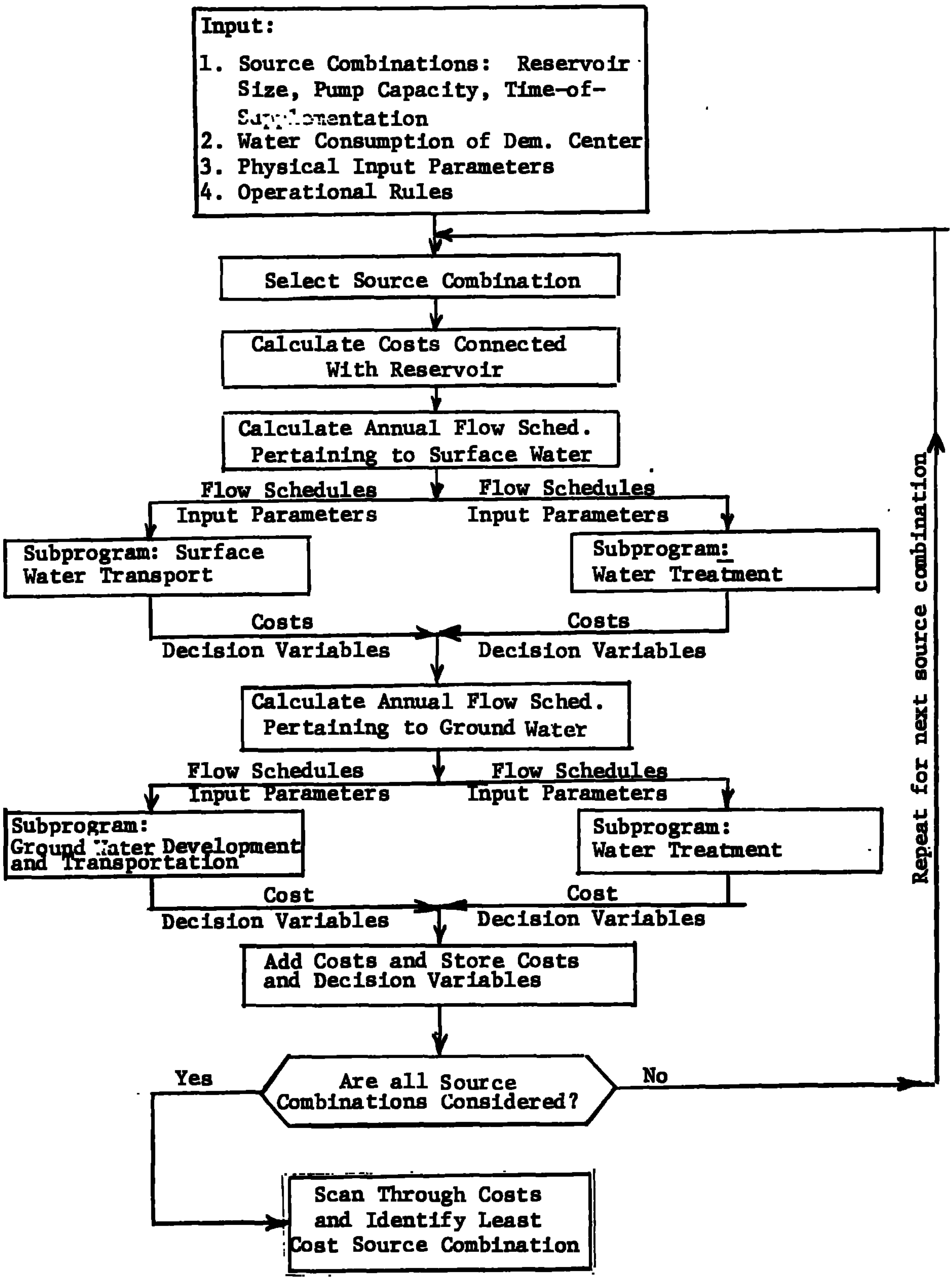

Fig. 5.1 - Outline of Cost Determination Procedure.

This rule holds whether the supply is provided by a single or dual source. The determination of monthly delivery volumes, however, requires slightly different procedures for alternatives involving a different number of sources.

5.2.1 Alternatives Involving Single Sources

When only one source is used to supply the water demand, monthly delivery volumes are taken directly from consumption schedules. Except in the case of ground water supply, in which the drawdown determined by a month-by-month pumping and replenishment balance affects pumping costs, the schedule of supply peaks and volumes and a cost calculation as described in Appendix B will provide the capital and O&M costs for the system. For the hypothetical water demand and supply system used in this study, a single set of system capacities, supply volumes, pipe sizes, and costs are produced and presented in Table 5.1 at the end of this chapter.

5.2.2 Alternatives Involving Two Surface Water Sources

When two reservoirs are being considered, the cost of several possible reservoir combinations have to be computed to find the least cost reservoir combination. As a sample computation for Alternative IIA-B, let the size of reservoir A be 2,750 ac-ft. According to the yield isoquant in Figure 4.18 the corresponding size of reservoir B is 2,593 ac-ft. The time of supplementation, i.e. the year when reservoir B must be in operation, is read from Figure 4.19 as year 15. This analysis is repeated for a feasible range of size for reservoirs A and B; and from this a least cost solution is found.

For the cost of water treatment, design schedules are the same as those for a single source system if both reservoirs deliver to the same treatment plant and are assumed to have the same water quality. If not, the procedure can be modified to handle "blending" of water of different quality and delivered to different treatment plants.

For the costs of water transportation, however, design quantity schedules are different, because different quantities of water may be produced by each of the reservoirs. For example, up to year 14, the year before expansion is needed, all water comes from reservoir A. Hence, the peak daily water conveyance capacity of pipeline A must at least be equal to the demand center's maximum day water requirements for year 14, which according to Figure 4.17 is 53.5 ac-ft/day. Once reservoir B is put in operation, its conveyance capacity should be made sufficient to supply the increment in peak consumption rate between years 14 and 50, namely 90.0 - 53.5 = 36.5 ac-ft/day. After year 14 it would be possible to let reservoir A continue to provide water at its peak capacity, as in year 14, while letting the deliveries from reservoir B build up gradually from almost zero in year 15 to their maximum in year 50. It is, however, more economical to cut back deliveries from reservoir A in year 15 and let reservoirs A and B supply water in proportion to their ultimate capacities, namely 53.5/90.0 = 60 percent by reservoir A and 40 percent by reservoir B. The maximum day and annual delivery volumes for this example are plotted in Figure 5.2 and 5.3.

The above outlined procedure was repeated for various size combinations of reservoirs A and B. The corresponding reservoir, conveyance and boosting, and treatment costs were computed concurrently and plotted in Figure 5.4. The low point on the total cost curve defines the least cost source combination within example alternative II A-B. Because the size of

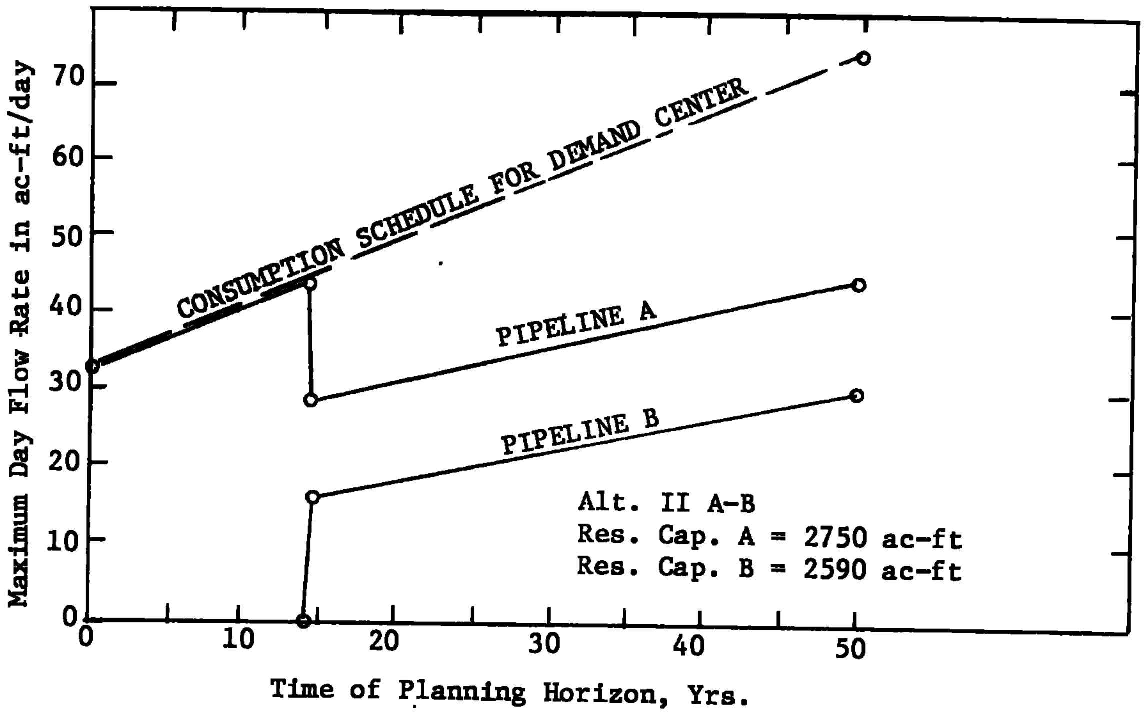

Fig. 5.2 Annual Schedules of Maximum Day Flow Rates for Alternative II A-B

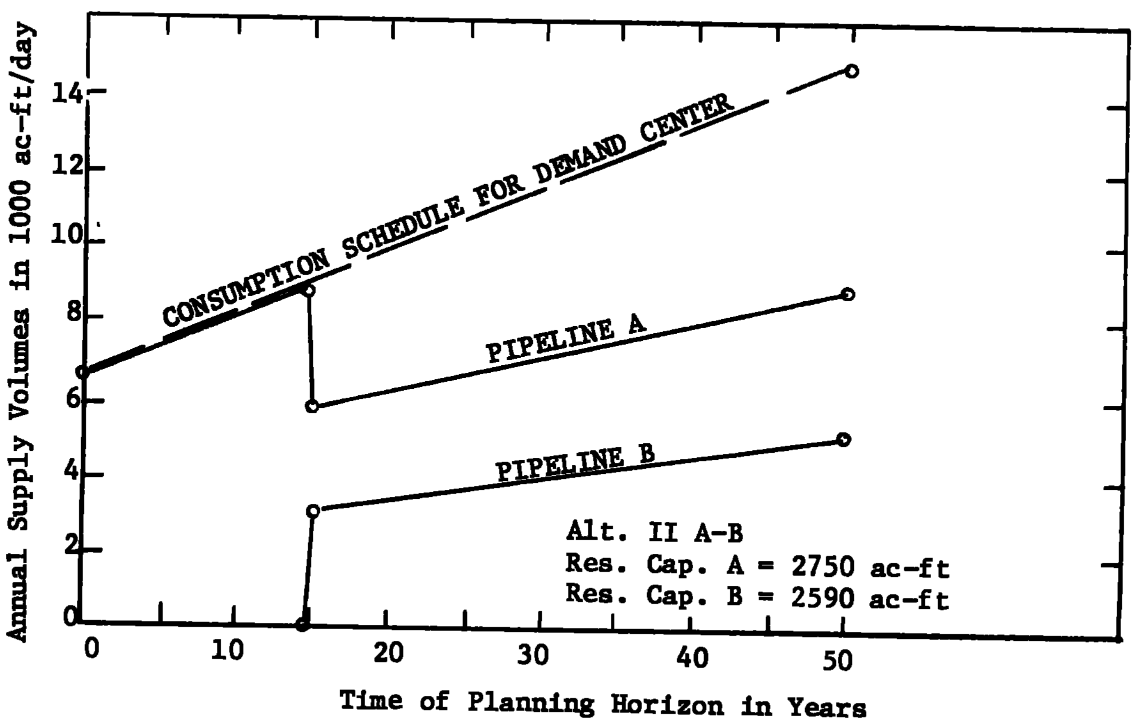

Fig. 5.3 Annual Schedule of Supply Volumes for Alternative II A-B

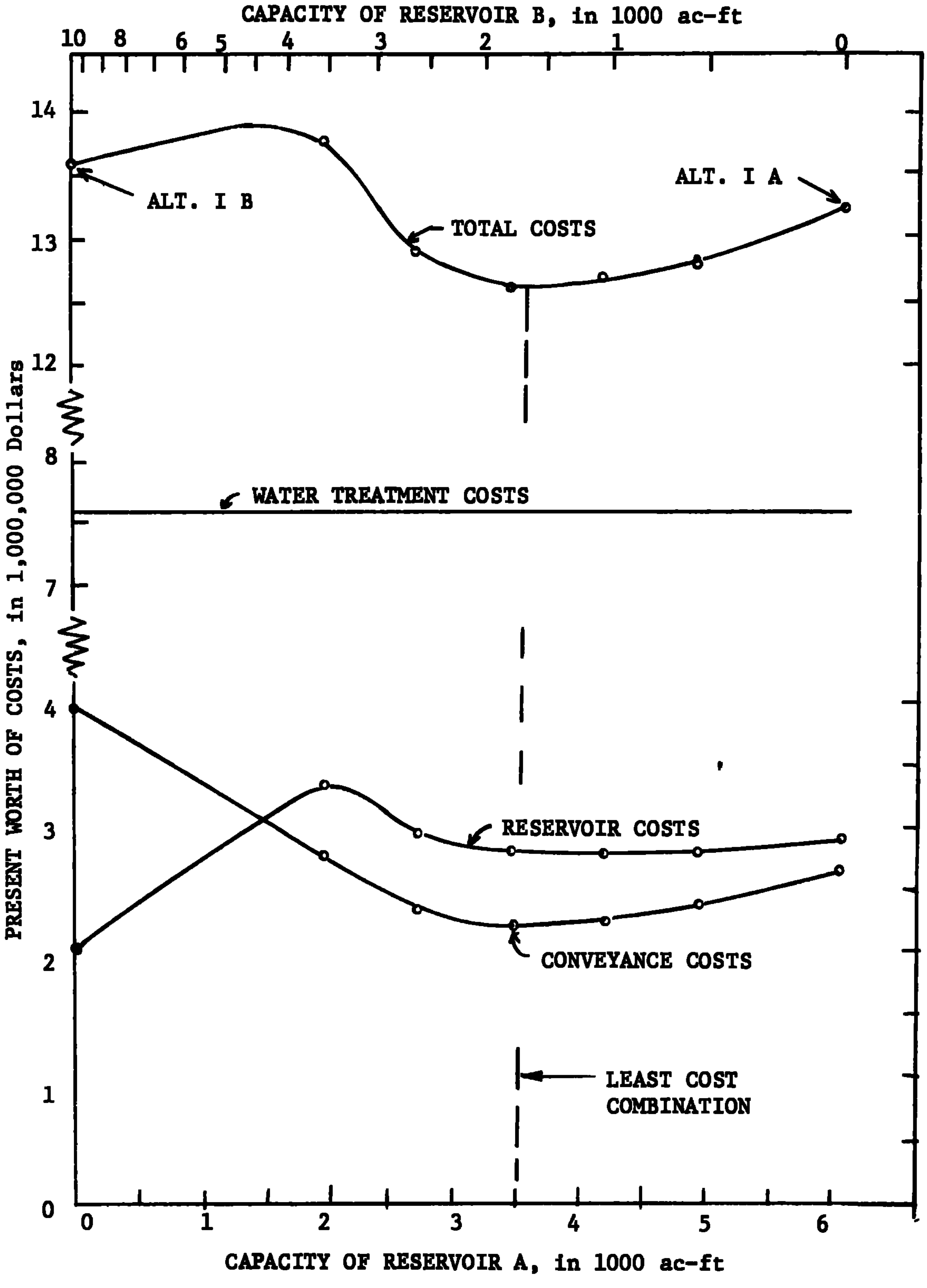

Fig. 5.4 - Identification of Least-Cost Source Combination for Alternative II A-B

reservoir A is 3,500 ac-ft, reservoir B should have a capacity of 1,720 ac-ft and be in operation by year 23, according to Figures 4.18 and 4.19.

Least-cost solutions can be determined in the same manner for all other alternatives involving two surface water sources. Costs and decision variables are summarized in Table 5.1

5.2.3 Alternatives Involving Surface and Ground Water Sources

In this category three different operational schemes for combining ground and surface water are distinguished.

1. Scheme 1 deals with alternatives III A-AQ, III B-AQ, and III R-AQ. The combined operation of ground and surface water follows the rules developed in Chapter 4 for the combined yield determination procedure. The main characteristic of this scheme is that ground water is perceived as a supplemental source for times of surface water shortages, operating under the rule of preventive pumping.

2. Scheme 2 deals with the same alternatives as Scheme 1. However, in contrast to Scheme 1, ground water, once it is developed, is used at full wellfield capacity during the entire time of the planning horizon. In order to distinguish alternatives of this scheme from those of Scheme 1, the AQ is marked with a bar, as for instance: alternative IIIA-$\overline{\text{AQ}}$.

3. Scheme 3 covers alternatives III AQ-A, III AQ-B and III AQ-R. Ground water is the initial and main source, supplemented by surface water when necessary.

5.2.3.1 Scheme 1: Design flow schedules are derived from yield isoquants and time-of-supplementation curves developed in the previous chapter (Figures 4.10 and 4.21). The determination of design flow schedules is demonstrated for the following example source combination: reservoir capacity = 4,400 ac-ft; ultimate monthly wellfield capacity = 600 ac-ft; and time of supplementation = year 14. The surface water source contributes the full supply whenever possible. Therefore, the yearly schedule of maximum day flow rates for surface water conveyance and treatment facilities is equal to the yearly schedule of maximum day consumption of the water demand center. Starting in year 14, the time of supplementation, the wellfield capacity should be developed gradually until it reaches the ultimate value of 600 ac-ft/month, in year 50. The wellfield capacities required for any intermediate year can be found with the aid of Figures 4.14 and 4.17. For example, the required combined yield in year 30 is, from Figure 4.17, equal to 11,700 ac-ft/year. Entering Figure 4.14 with a 4,400 ac-ft reservoir size and 11,700 ac-ft combined capacity, a required wellfield capacity of roughly 300 ac-ft per month is obtained. The resulting schedules of required capacity are plotted in Figure 5.5.

Design flow schedules of annual ground water delivery volumes are obtained from the curves of average annual ground water volumes versus reservoir and wellfield capacity. For the example chosen above, the installed capacities in year 30 are 4,400 ac-ft of reservoir B storage volume and 300 ac-ft/month of wellfield capacity. Interpolation in Figure 4.14 results in an average annual ground water requirement of 1,200 ac-ft. Annual delivery volumes of ground and surface water over the planning period were plotted in Figure 5.6 for the chosen example.

Flow schedules and physical input parameters can then be used to determine the least cost source combination and corresponding decision variables. In Figure 5.7, total costs and partial costs for each cost center

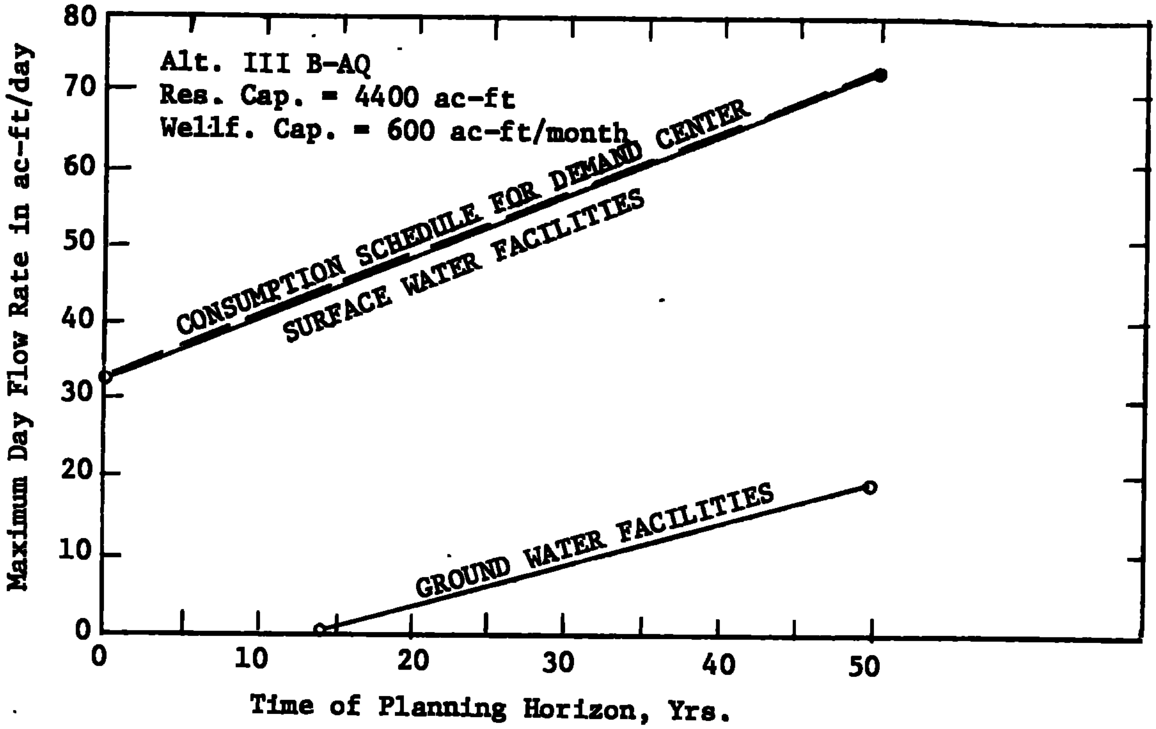

Fig. 5.5 Annual Schedules of Maximum Day Flow Rates for Alternative III B-AQ

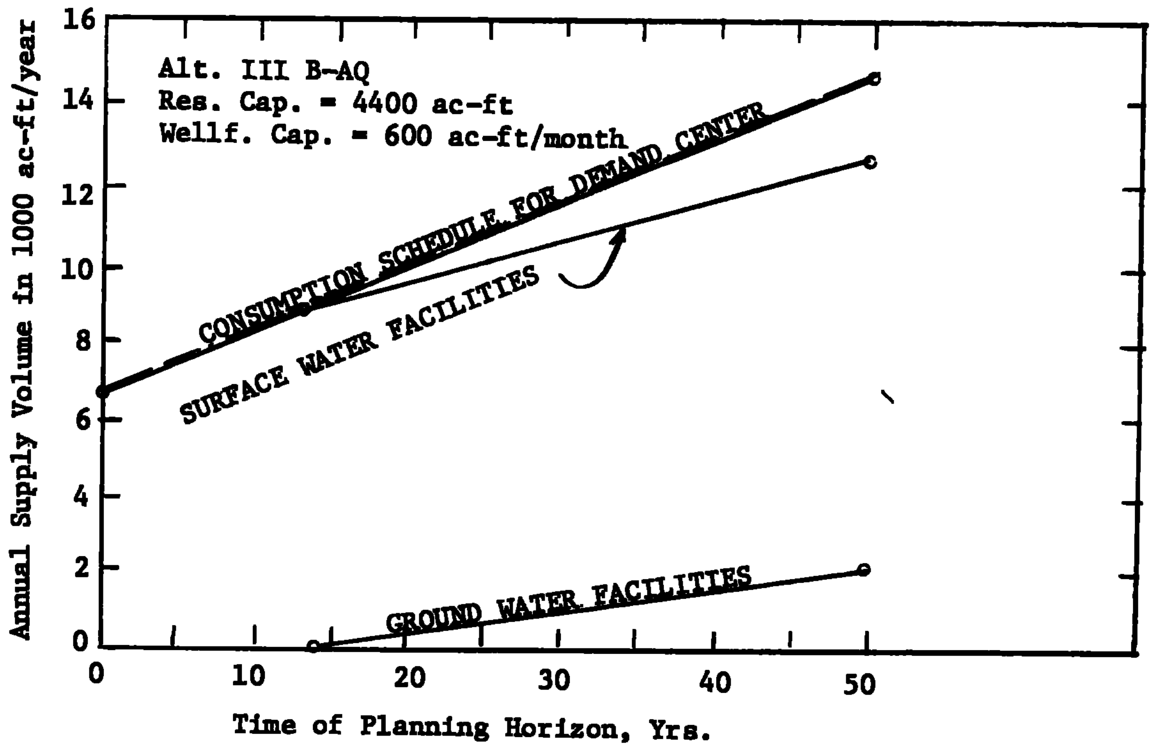

Fig. 5.6 Annual Schedules of Supply Volumes for Indicated Alternative III B-AQ

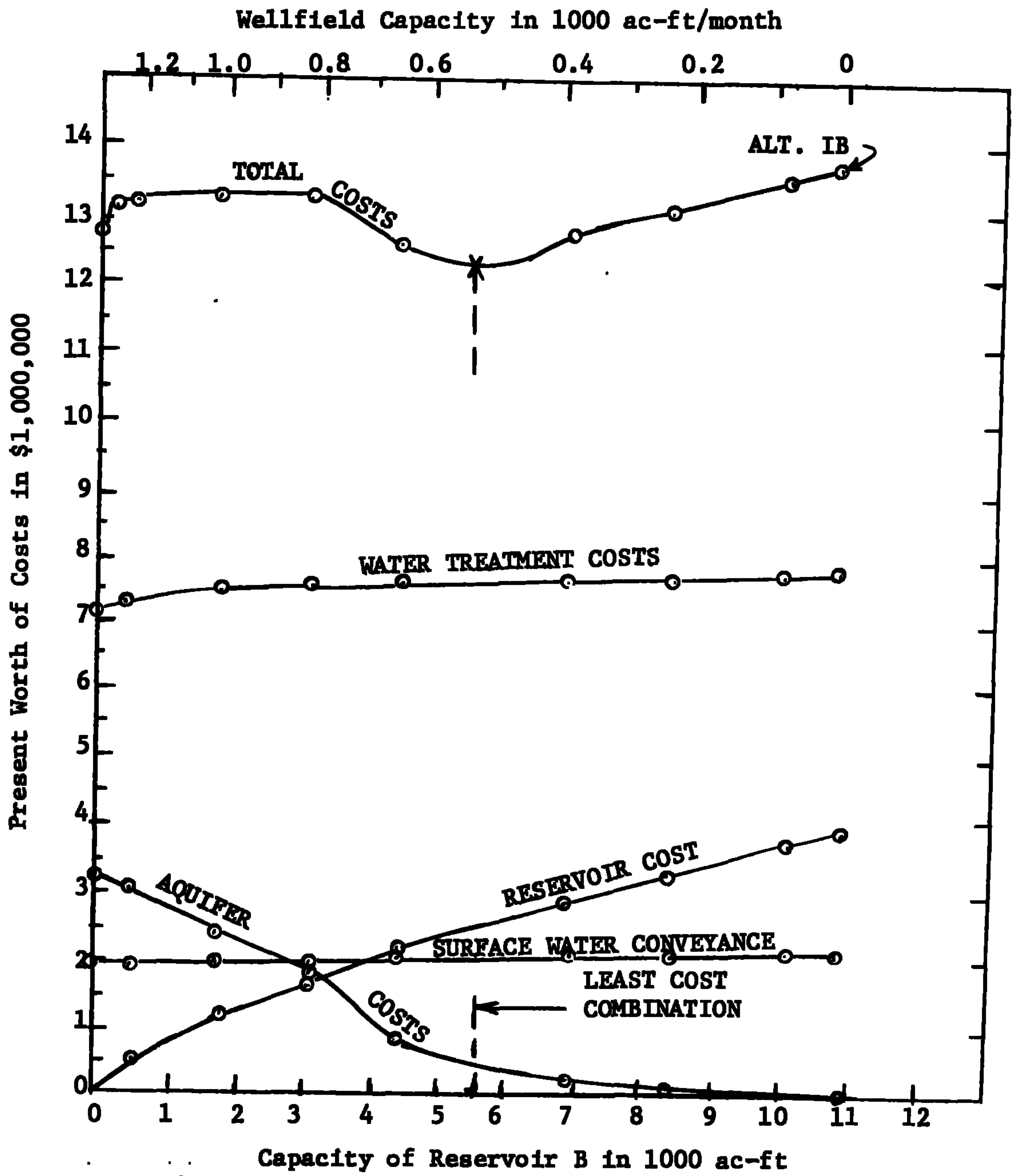

Fig. 5.7 Identification of Least-Cost Source Combination for Alternative III B-AQ

are plotted against reservoir capacity for alternative III B-AQ. It will be noted that reservoir costs and aquifer costs vary significantly with source combination as expected. Costs for surface water conveyance and water treatment show little change since, according to the operational scheme, surface water contributes the great bulk of water over the entire range of source combinations. For alternative III B-AQ applied to the hypothetical system investigated, the lowest cost will be found for a reservoir size of 5,500 ac-ft. Figures 4.19 and 4.10 indicate the corresponding time of supplementation as year 23 and a required ultimate wellfield capacity of 520 ac-ft/month.

The above computations are based on a preventive pumping ratio of average to expected monthly reservoir inflows of 5. The claim made in Chapter 4 that this degree of preventive pumping was economically most efficient is substantiated in Figure 5.8 in which the present worth of total costs is plotted against the degree of preventive pumping.

The least cost combinations and associated design flow schedules were also computed for alternatives III A-AQ and III R-AQ. Results are summarized in Table 5.1 at the end of this chapter.

5.2.3.2 Scheme 2: The basic difference between this and Scheme 1 is that full use of the installed wellfield capacity is made. In such an operational scheme preventive pumping is achieved to its highest possible degree. For this reason design flow schedules cannot be derived from the yield isoquants and combined yield curves of Figures 4.20 and 4.14, respectively. Instead, combined yield curves were used, based on a very large degree (Z100) of preventive pumping, corresponding in effect to the

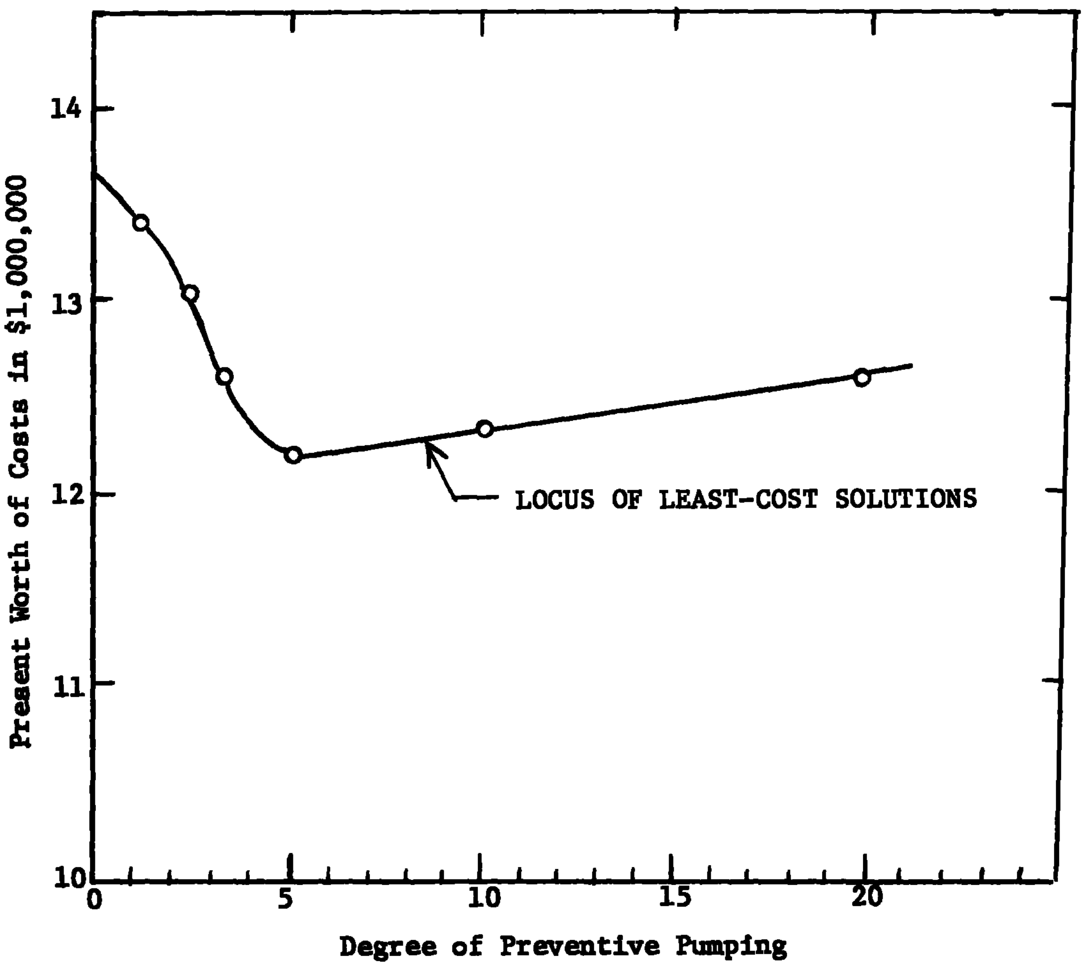

Fig. 5.8 – Relationship between Degree of Preventive Pumping and the Least-Cost for Alternative III B-AQ

assumption of zero expected reservoir inflows in the determination of target carry-over storages. Taking for an example the previous ultimate wellfield capacity of 600 ac-ft/month used in Scheme 1, the required reservoir size dropped from 4,400 to 3,930 ac-ft, and according to Figure 4.19, the required time of supplementation dropped from year 14 to 10. Consequently, the annual design schedule of maximum daily flow rates through ground water facilities starts with a value of 0 in year 9 and grows to a value of 600/30.4 = 19.6 in year 50 as illustrated in Figure 5.9. The maximum day design schedules for surface water conveyance and treatment are the differences between the annual schedule of the demand center's peak daily consumption rates and the schedules for ground water facilities. The design flow volumes are directly proportional to maximum day design capacities and are shown for the example in Figure 5.10.

The source combination - cost relationships are shown for alternative III B-AQ in Figure 5.11. These curves indicate for this alternative that source combinations become less expensive as the relative contribution of ground water increases. The least-cost source combination occurs for the case of zero reservoir capacity (pumping from aquifer and without surface storage). Similar results were obtained from alternative III A-$\overline{AQ}$. For alternative III R-$\overline{AQ}$ supply from the river only was the preferable combination. Results of alternatives within Scheme 2 are summarized in Table 5.1.

5.2.3.3 <u>Scheme 3</u>: This scheme covers alternatives III AQ-A, III AQ-B, and III AQ-R. The aquifer is the initial and main source of supply. Flow schedules again were based on combined yield relationships resulting from a high degree of preventive pumping as illustrated for the following

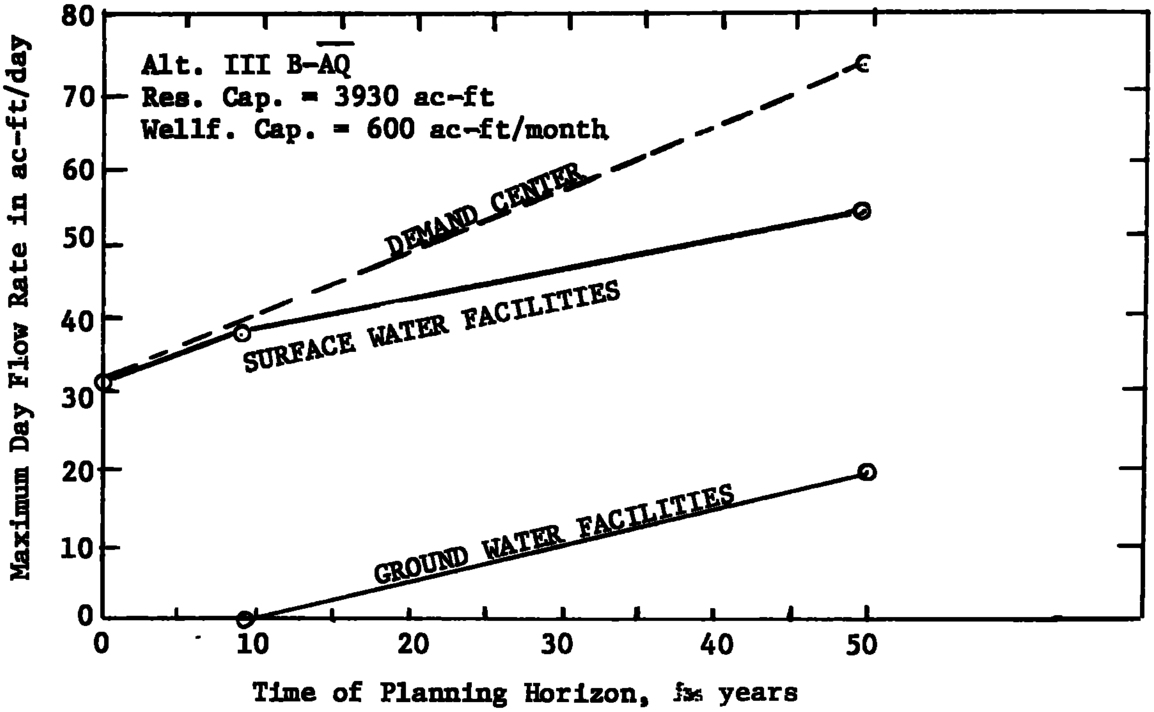

Fig. 5-9 Annual Schedules of Maximum Day Flows Rates for Alternative III B-AQ

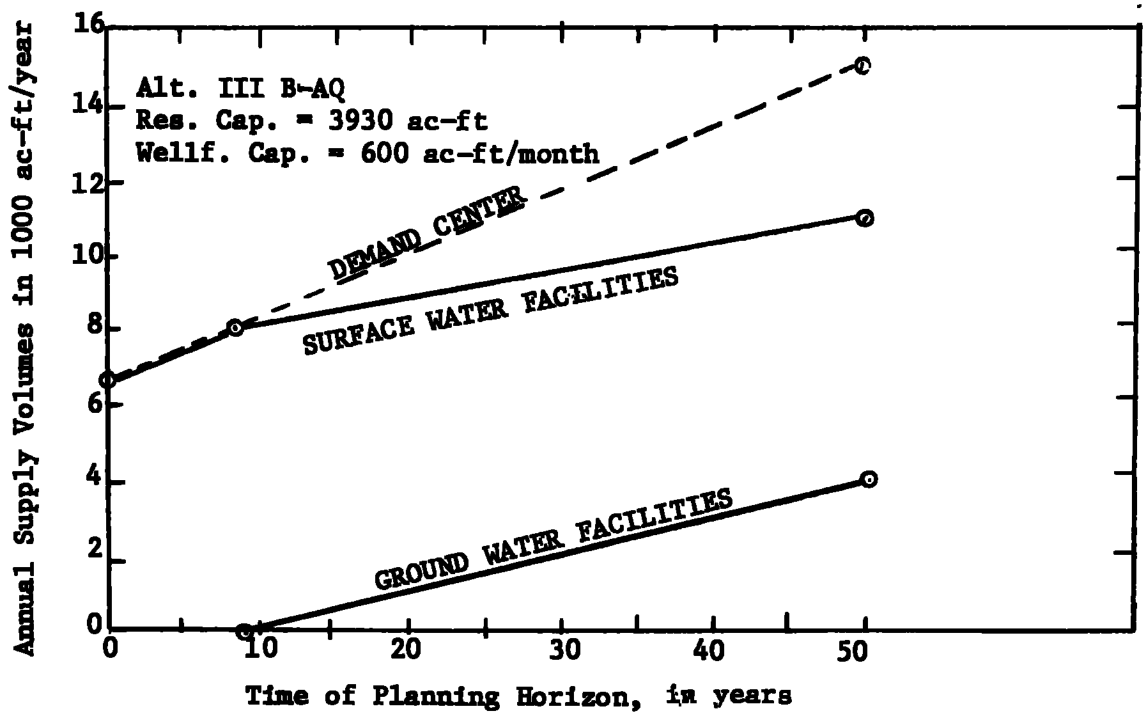

Fig. 5.10 Annual Schedules of Volumes Supplied by Alternative III B-AQ

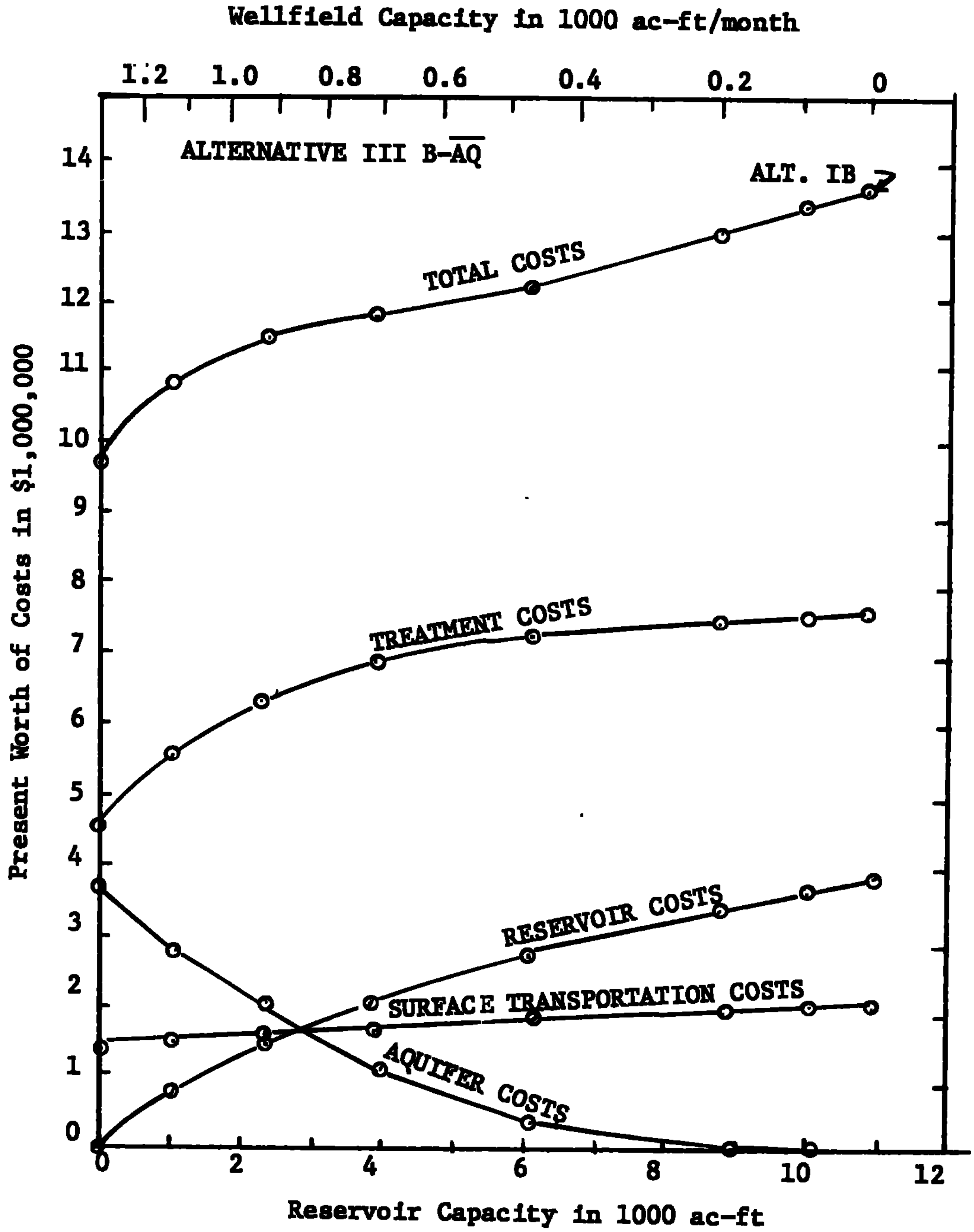

Fig. 5.11 Identification of Least-Cost Source Combination for Alternative III B-$\overline{AQ}$

example: reservoir size = 564 ac-ft, monthly wellfield capacity = 1,120 ac-ft, and time of supplementation = year 6.

Up to year 6 all water supply comes from the aquifer. Hence, ground water flow schedules follow demand center schedules for the first 6 years. For the remainder of the time of the planning horizon, maximum day capacity and volume of flow through ground water supply units remain constant at the six-year values. Starting with the year of supplementation, surface water provides the difference between demand center schedules and ground water supply schedules. Schedules for the example source combination are presented graphically in Figures 5.12 and 5.13.

Costs as a function of source combinations are plotted in Figure 5.14. Total costs decrease as the magnitude of surface water contribution reduces. The least-cost combination was in this case the one without any surface water contribution, which is equivalent to the single-source aquifer alternative I-AQ. However, as stated earlier, the possibility of aquifer depletion under this alternative should be given serious consideration.

Similar results were obtained for alternatives III AQ-A and III AQ-B, as summarized in Table 5.1.

5.3 Discussion of Results

The cost determination procedures employed in this study are adequate for a preliminary scanning of a number of alternatives in order to identify those which deserve more detailed attention. Before any final decisions are made, the costs of the more promising alternatives identified in the scanning procedure might be recalculated in more detail. Minor cost differences between alternatives (for instance, the total costs of

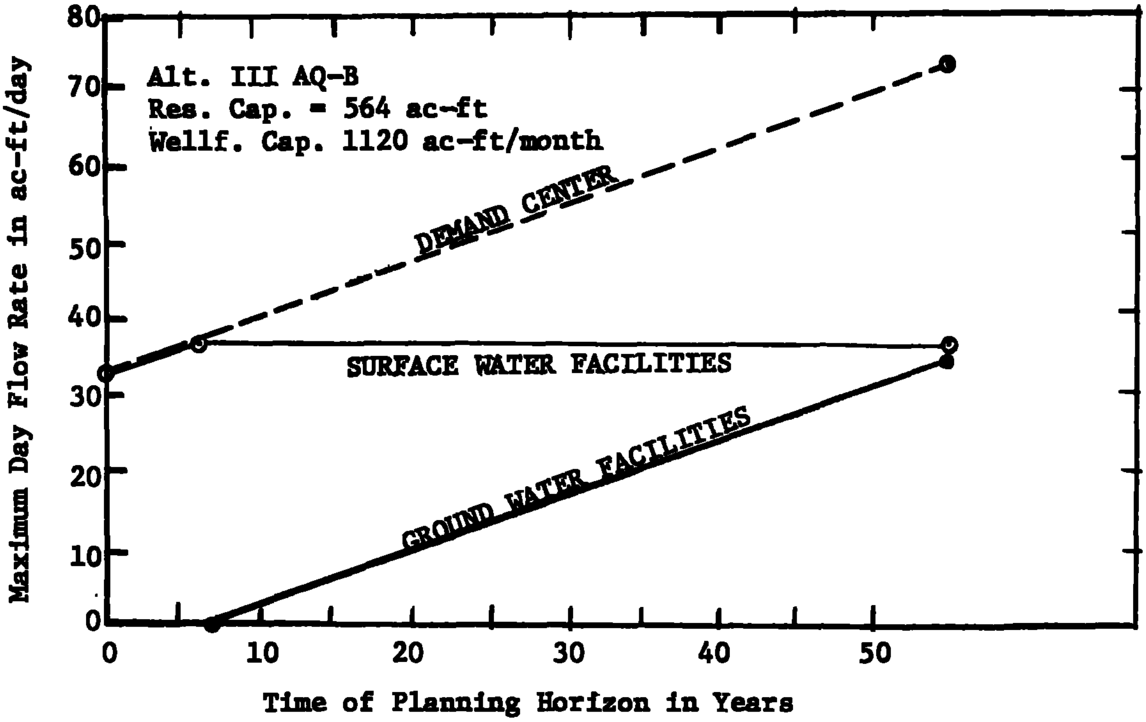

Fig. 5.12 Annual Schedules of Maximum Day Flow Rates for Alternative III AQ-B

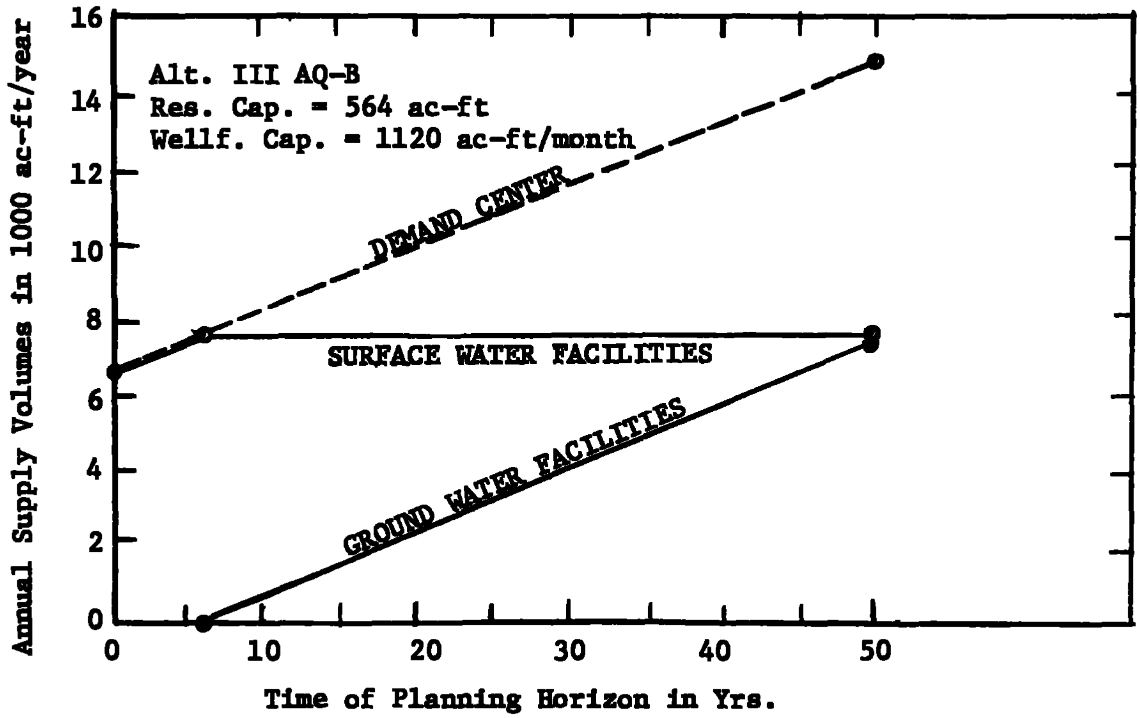

Fig. 5.13 Annual Schedules of Supply Volumes for Alternative III AQ-B

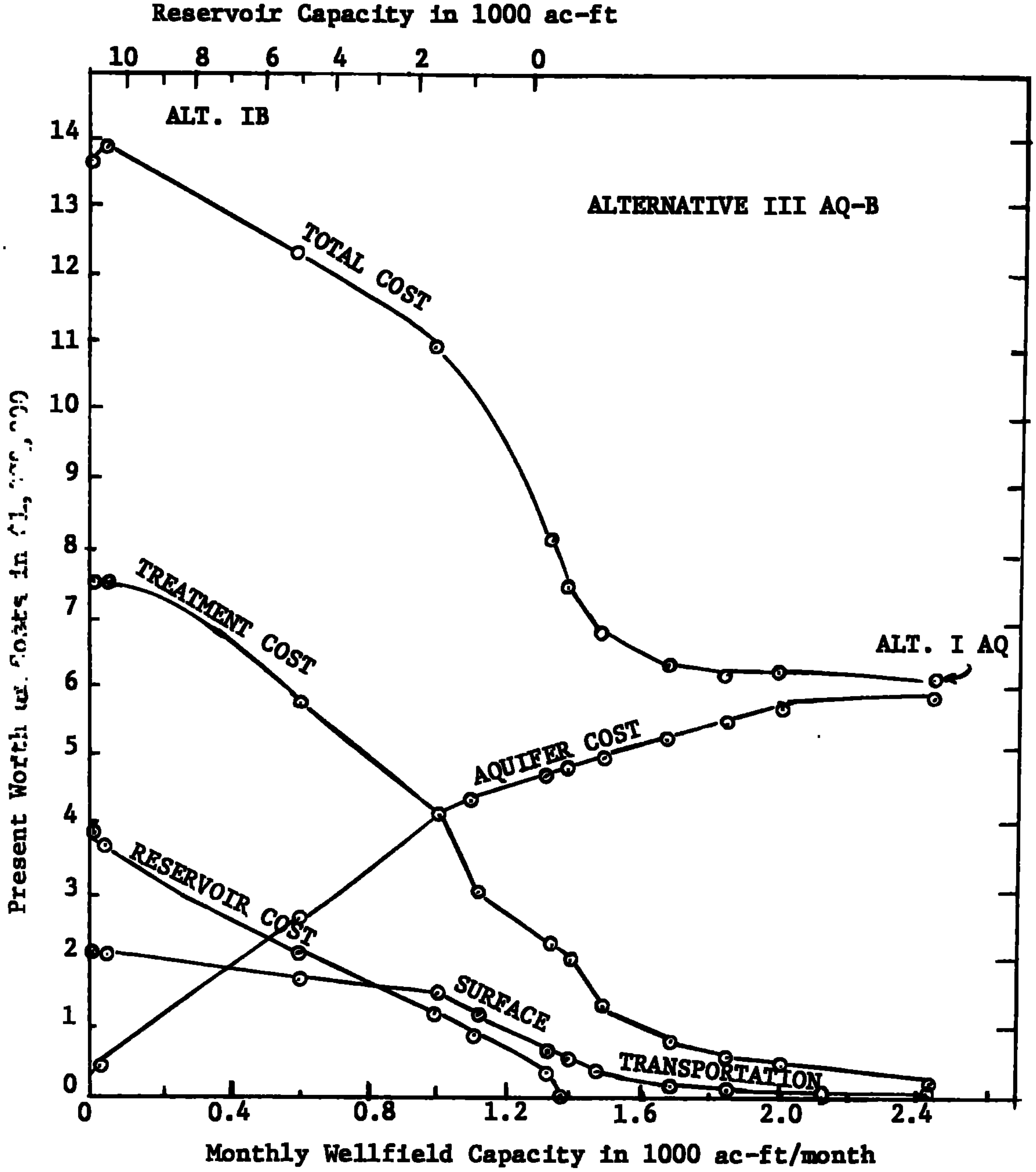

Fig. 5.14 Identification of Least-Cost Combination of Sources for Alternative III AQ-B

alternative II B-A differ from these of alternative II A-B by only $40,000 or 0.3 percent) should not be used to declare one alternative superior to another. Considerations other than costs may in this case be used to decide between alternatives. For example, the ultimate success of any alternative is largely influenced by the uncertainty of water consumption projections. Therefore, it should be a general rule that the alternative which provides more flexibility with respect to system expansion and stage construction should be preferred if costs seem to be relatively similar for the alternatives.

Comparing the total costs listed in Table 5.1 for the alternatives considered in this study, the use of ground water as a single source was far less expensive than any other alternative, with the single-source river water use a distant second choice. Obviously, these conclusions cannot be interpreted as a universal rule, but are highly dependent on the assumptions made in setting up the hypothetical system. The main reason for the low cost of the exclusive ground water supply was the assumption that chlorination would be the only treatment needed for ground water, whereas surface was was assumed to require conventional settling, filtration, and chlorination. This difference in treatment requirements is, however, standard in many places, to the extent that some communities like San Jose, California, prefer to use surface water for aquifer recharge and subsequently pump from wells for direct use after chlorination (Aron, 1969).

The reason for the lower cost of direct run-of-river pumping compared to the use of surface reservoirs lies obviously in the savings achieved by not having to build a reservoir. This conclusion is, however, based

Table 5.1. Summary of Costs and Decision Variables

Alternative	Primary Decision Variables: Capacity Res. A ac-ft	Capacity Res. B ac-ft	Wellfield Capacity ac-ft/mon.	Time of Expansion	Secondary Decision Variables: Pipe Diameter inches	Number of Wells
I A	6,050	-	-	-	36	-
I B	-	10,923	-	-	36	-
I AQ	-	-	2,740	-	36	52
I R	-	-	-	-	36	-
II A-B	3,500	1,720	-	23	30/20	-
II B-A	1,150	5,500	-	24	30/20	-
II R-A	-	-	-	51	36	-
II R-B	-	-	-	51	36	-
II B-R	-	-	-	1	36	-
II A-R	-	-	-	1	36	-
III A-AQ	4,000	-	280	27	36/14	8
III B-AQ	-	5,500	500	25	36/20	13
III R-AQ	-	-	-	1	36	-
III A-AQ	-	-	1,059	1	30/24	25
III B-AQ	-	-	1,272	1	24/30	30
III R-AQ	-	-	-	51	36	-
III AQ-A	-	-	2,740	51	36	52
III AQ-B	-	-	2,740	51	36	52
III AQ-R	-	-	2,790	51	36	52

Table 5.1. (Continued)

Costs in $1,000's

Alternative	Reservoir	Aquifer	Surface Water Transport	Source Development	Treatment	Total
I A	$2,640	$ -	$2,871	$5,511	$7,627	$13,138
I B	3,990	-	2,038	6,028	7,627	13,655
I AQ	-	5,820	-	5,820	374	6,194
I R	-	-	2,596	2,596	7,677	10,223
II A-B	2,240	-	2,805	5,045	7,627	12,672
II B-A	2,793	-	2,167	4,960	7,627	12,587
II R-A	-	-	2,596	2,596	7,627	10,223
II R-B	-	-	2,596	2,596	7,627	10,223
II B-R	-	-	2,596	2,596	7,627	10,223
II A-R	-	-	2,596	2,596	7,627	10,223
III A-AQ	2,000	150	2,850	5,000	7,600	12,600
III B-AQ	2,300	350	2,050	4,700	7,500	12,200
III R-AQ	-	-	2,596	2,596	7,627	10,223
III A-AQ	-	3,105	2,175	5,280	5,289	10,569
III B-AQ	-	3,701	1,442	5,143	4,642	9,785
III R-AQ	-	-	2,596	2,596	7,627	10,223
III AQ-A	-	5,820	-	5,820	374	6,194
III AQ-B	-	5,820	-	5,820	374	6,194
III AQ-R	-	5,820	-	5,820	374	6,194

again on the assumption that river water is available in the desired quantity and does not require relatively more extensive treatment.

A more general conclusion which the investigators believe can be drawn from the cost summary in Table 5.1 is that single source systems tend to be preferable to multisource systems, particularly if the primary source is an aquifer and the secondary source a reservoir with high investment costs. This conclusion could be used to caution planners against integrated use just for the sake of integration. If an aquifer has the capacity to provide a reliable water supply of good quality to a community at a reasonable cost, it is highly unlikely that supplementation of the ground water with surface water will yield any advantages to the community. The most common shortcomings of many aquifers lies in their relatively small natural replenishment and thus their vulnerability to depletion by sustained pumping. In that case some surface water supply, either from direct pumping or from a reservoir, is unavoidable, and the most logical comparison of planning efficiencies is between independently operated surface and ground water systems and an integrated system in which the scarce ground water reserves are called upon to supplement the surface water supply during months of high consumption and low inflows. The curves in Figures 4.15 and 4.16 exhibit a <u>clear gain in yield</u> through integrated rather than independent operation of the two sources.

CHAPTER 6

SUBSIDIARY SYSTEM DESIGN CONSIDERATIONS

The purpose of this chapter is to demonstrate procedures that can be used to reduce the required capacity of system components for storage, conveyance, and treatment of water. It is proposed that treated-water intermediate storage be used to attenuate maximum day consumption rates. In addition, it is proposed that decision theory be used to determine the flexibility of a system to deviations from the predicted trends of water consumption caused by the uncertainties in population forecasts. It is shown that intermediate storage can be used profitably as a buffer against incorrect population forecasts. A procedure will be described for determining: 1) the effectiveness of intermediate storage in reducing required system capacities; 2) the economic feasibility of using intermediate storage; and 3) the time within the period of analysis during which this storage should be installed.

6.1 Intermediate Storage Provision

The minimum required design capacity of a municipal water supply system is usually equivalent to the maximum day consumption rate (Camp and Lawler, 1969). However, an inspection of a typical daily consumption rate-duration curve as shown in Figure 6.1 indicates that there are only a few days in a year that require such extremely high water deliveries in comparison to the average daily consumption rate. If the consumption for these few days could be reduced or satisfied in some other manner, the design capacity (that is, the peak rate of delivery) of the water supply system itself could be substantially reduced or conversely, the capability of an existing supply system could be increased significantly.

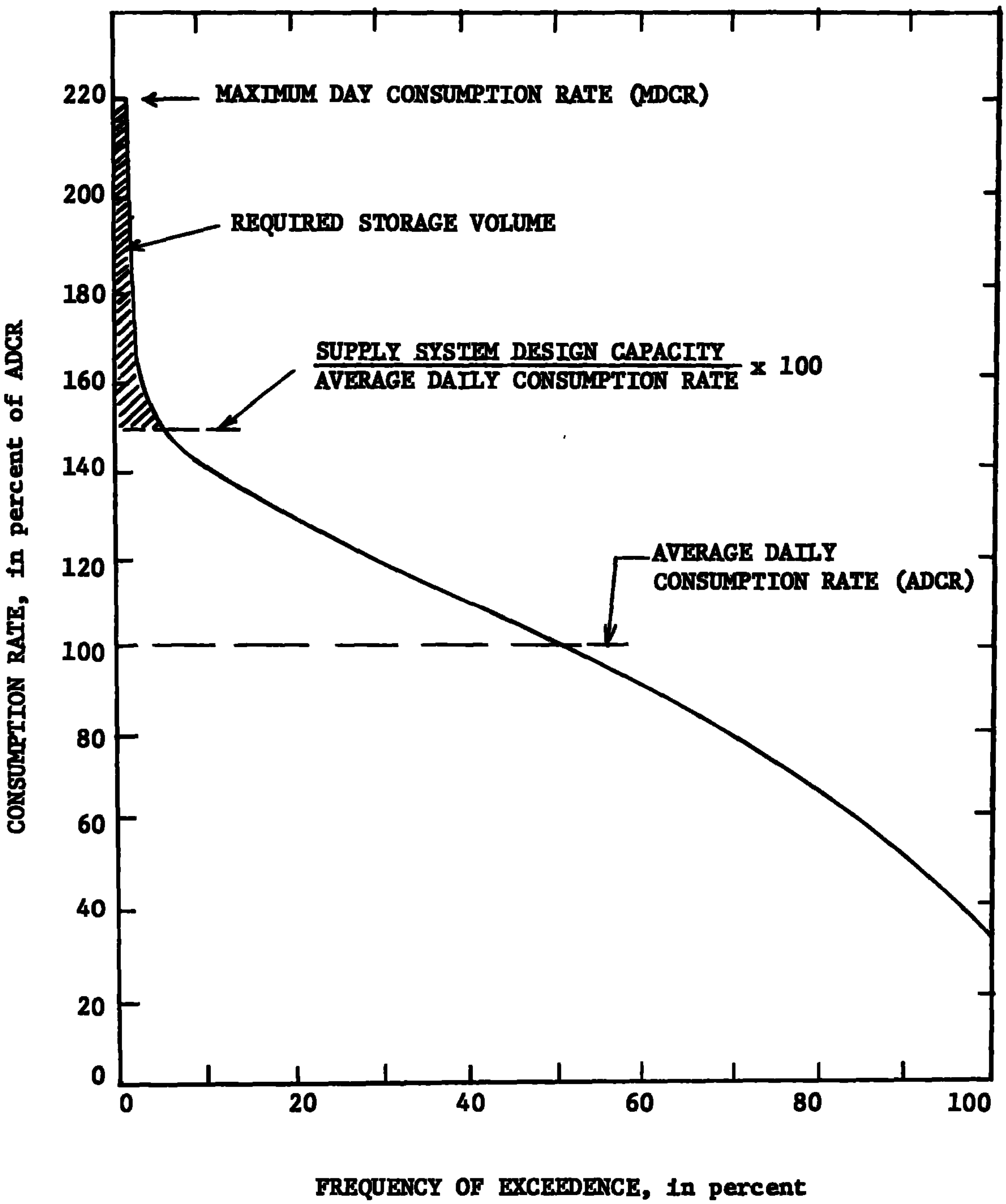

Fig. 6.1 Daily Consumption Rate-Duration Curve (after Babbitt, 1959)

The treated-water storage in a typical municipal system is designed to attenuate the hourly fluctuations in demand and to provide extra storage for fire fighting. The "intermediate storage" proposed in this chapter, most likely in the form of an underground tank, would be located just "downstream" from the water treatment plant and could carry the demand center over a few days of high demand.

To determine the intermediate storage volume needed to lower the required system capacity by a given amount, the conservative assumption is made that all days of high consumption rates occur in sequence. The required volume of storage is then calculated by measuring the cumulative area under the consumption rate-duration curve. for example, the shaded area under the curve in Figure 6.1 corresponds to the volume of intermediate storage that would permit a decrease in the system's design capacity from 220 percent to approximately 150 percent of the average day consumption rate. If high consumption days were assumed to occur nonsequentially, the volume of required intermediate storage would be reduced.

By calculating the storage requirement for several different ratios of design capacity to average daily consumption rate, a tradeoff function between intermediate storage and water supply system design capacity can be developed. A typical tradeoff function, presented in Figure 6.2 for the consumption rate-duration curve shown in Figure 6.1, has the distinguishing characteristic of an initially large decrease in design capacity requirement for a small increment of added intermediate storage.

All combinations of intermediate storage and design capacity shown in Figure 6.2 are based on a maximum to average day consumption ratio (RMA) of 2.20, and in this respect the systems are equivalent. However, since a

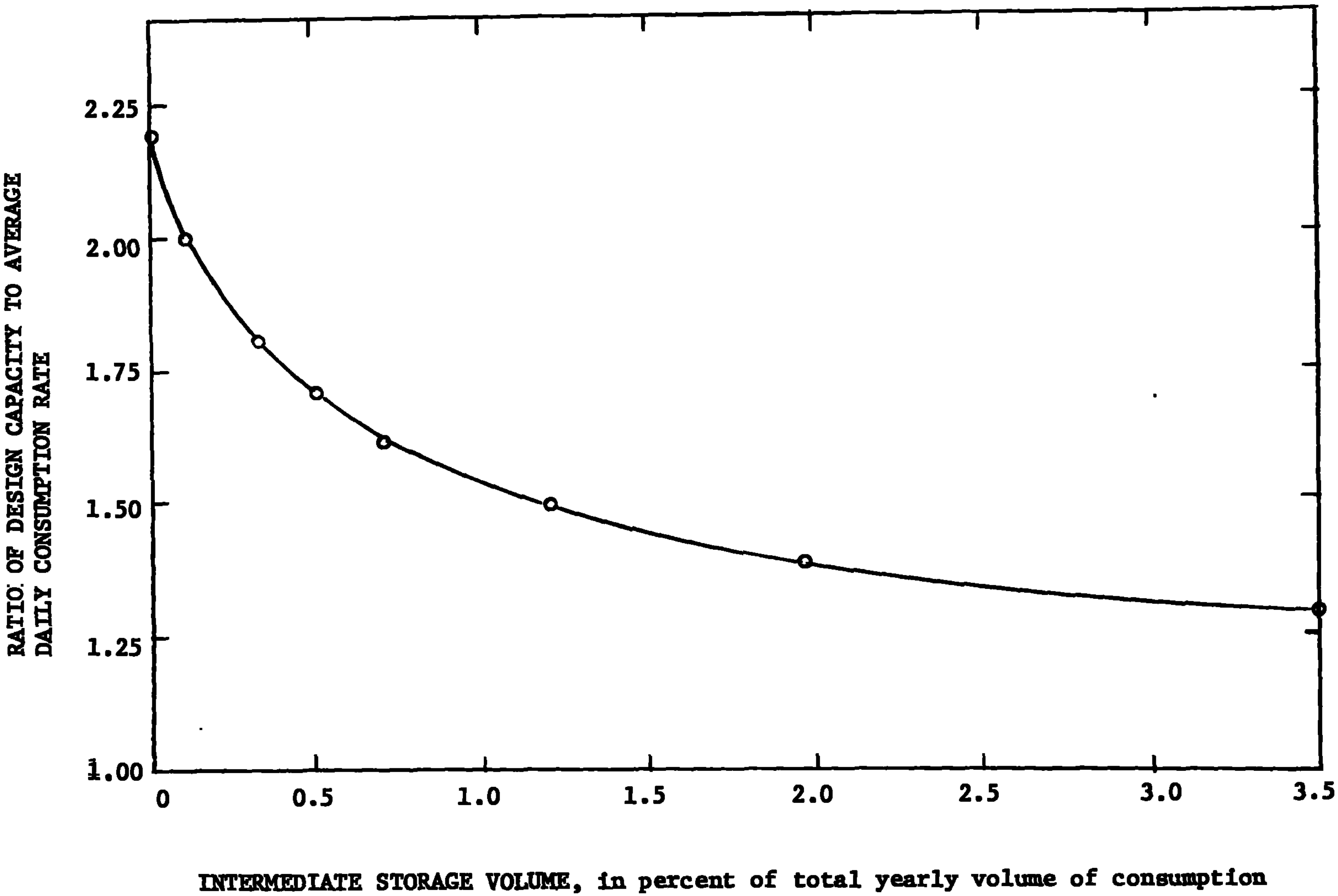

Fig. 6.2 Tradeoff Function Between Design Capacity of Supply System and Volume of Intermediate Storage

larger amount of intermediate storage permits greater flexibility in the operation of the system at operating levels below the mean daily consumption rate, the different systems are not necessarily equivalent in all respects.

The least-cost combination of intermediate storage and system design capacity cannot be determined from the physical tradeoff function alone, but must come from a detailed cost analysis. The cost of intermediate storage can be calculated with relative ease, as discussed in Appendix B, after the type of storage has been selected (i.e., surface storage, buried concrete reservoir, elevated steel tank, etc.). However, the counterpart reduction in cost of other system components is not as easily determined. To describe the procedure followed in this study, the hypothetical ground water supply system is analyzed using different volumes of intermediate storage. The design capacity for this system is determined as described in Chapter 4. The least-cost combination will be based on consideration of both investment and operation and maintenance (O&M) costs. Because water supply systems are normally planned for an increasing demand, the possibility of staging various system components is considered, including intermediate storage.

6.2 Economic Analysis of Intermediate Storage

The analysis for a time-invariant water demand system can easily be performed graphically. For a time-varying demand case, the numerous calculations are best performed by digital computer. Here, the computer program is used to develop all costs for both the constant demand and time-varying demand cases, although the procedure for determining the least-cost combination for the constant demand case will be demonstrated using the generalized cost curves.

6.2.1 Time-Invariant Demand Center

The procedure for finding the least-cost combination is as follows:

1. Determine the consumption rate-duration curve for the service area. This can be obtained from past daily consumption records for the service area or from a similar demand situation. Figure 6.1 is used in this example.

2. Calculate the physical tradeoff relationship between intermediate storage volume and system design capacity as illustrated in Figure 6.2. The curve can be drawn either in dimensionless form or in the units of the system analyzed.

3. Develop the capital investment as well as the operation and maintenance cost curves for the major system components, including the intermediate storage tank.

4. For a range of intermediate storages and associated design capacities add all costs and locate the least-cost combination.

Using the O&M cost equations presented in Appendix A, all costs for pumping power and energy, treatment, maintenance, etc. are added for a range of design capacities and load factors, or average consumption rate divided by design capacity. The result is a set of curves as shown in Figure 6.3. Across these curves an isoquant is drawn which corresponds to the system's average consumption rate, equal to 6 mgd for a hypothetical demand center of 40,000 population with 150 gpd use per capita.

Subsequently, system component costs curves as in Figure 6.4 can be drawn as a function of design capacity and corresponding intermediate

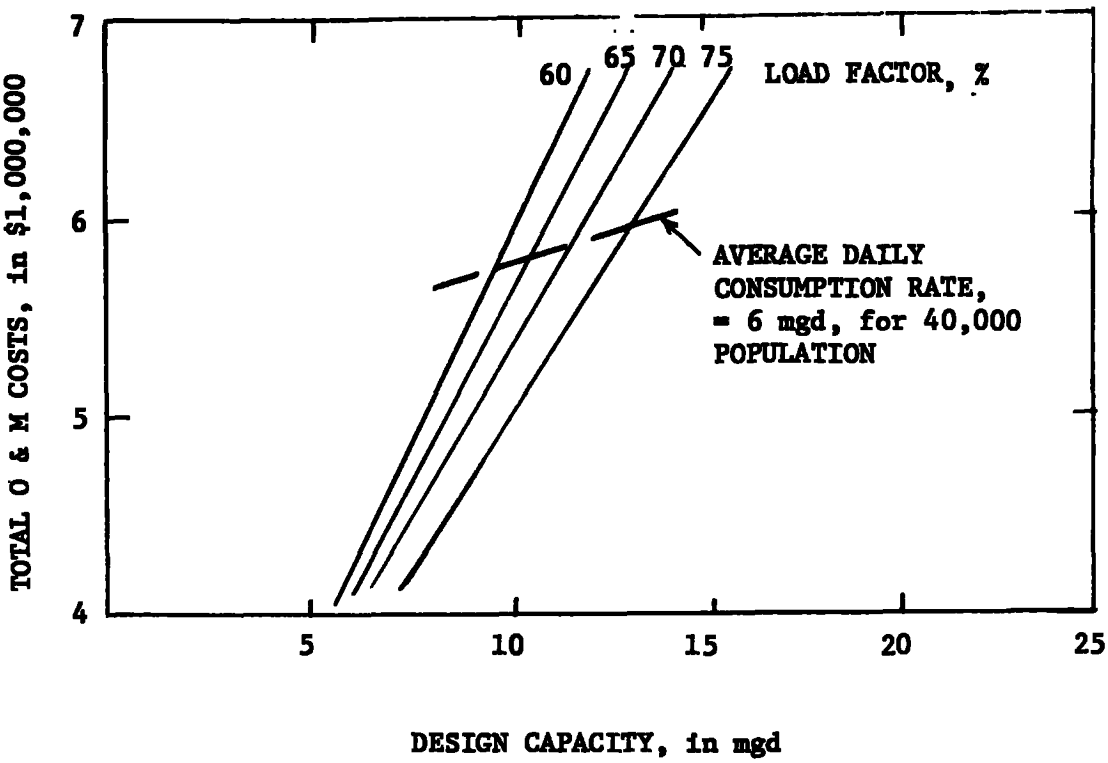

Fig. 6.3 Operation and Maintenance Costs for Water Supply System

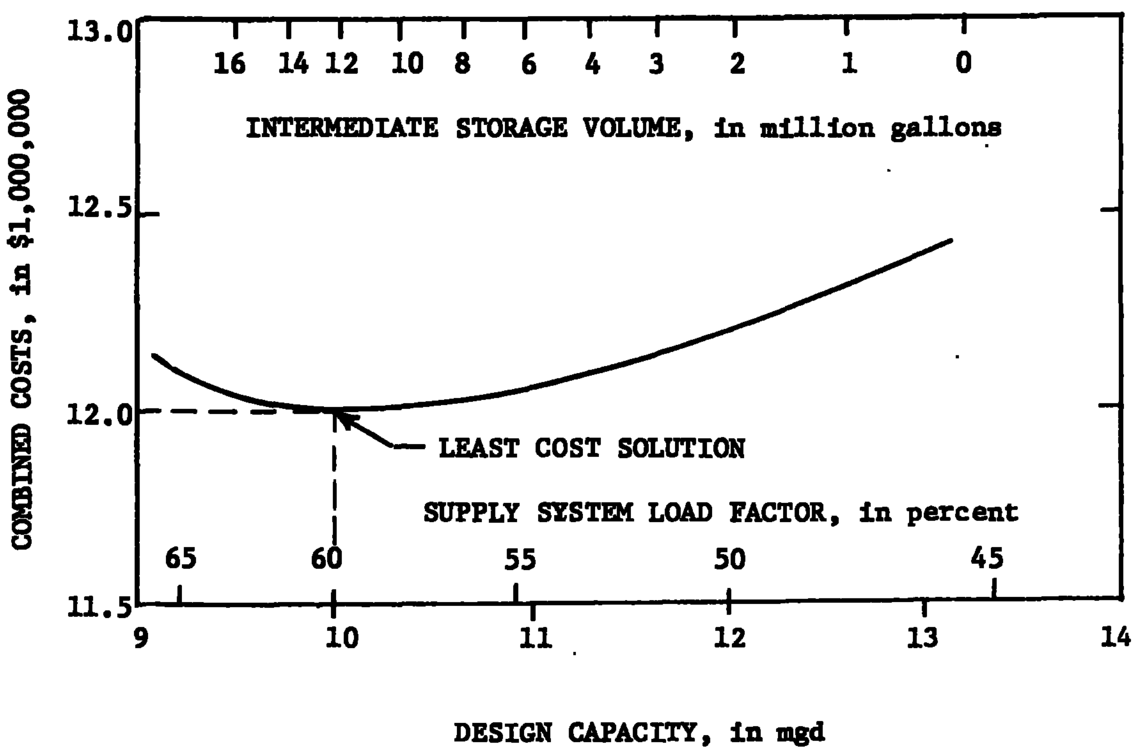

Fig. 6.4 Effect of Intermediate Storage Volume on Project Costs

storage volume, by computing the capital costs from the equations given in Appendix A and taking the O&M costs from the contour line in Figure 6.3. The low point of the total cost curve in Figure 6.4 denotes the "optimal" volume of intermediate storage. In this example, the combination corresponds to about 10 mgd capacity and 12 million gallons intermediate storage at a load factor of approximately 60 percent.

Total costs are not highly sensitive to storage volumes; however, it can be seen that the provision of intermediate storage lowered the total costs from almost $12.5 million to 12.0 million, or by about 4 percent. Further decreases of design capacities below 10 mgd would result in a rapid increase of the required storage volume, and thus in a steeply increasing cost curve. This example, it should be emphasized, is based on the counteractive assumptions of peak day demands occurring back-to-back; otherwise the savings would be greater.

6.2.2 Distribution of Costs

The distribution of component costs for a system with intermediate storage is best evaluated using a constant demand center because the present worth of the investment costs are not masked by stage construction or by discounting. A breakdown of the major cost items for the example system serving two different levels of constant demand is shown in Table 6.1. One fact is readily apparent -- that investment cost decreases with the addition of intermediate storage. The investment cost reductions for some components, such as pipelines, are limited in some cases by the discrete sizes that are commercially available. It is not realistic, for example, to design for a 38-inch pipeline.

Table 6.1. Distribution of System Costs for a Time-Invariant Demand Center, In $1,000's

Cost Items	Ratio of Design Capacity to ADCR			
	2.20	1.90	1.60	1.45
For Population = 40,000				
Investment Costs				
Transmission Line	2,174[a]	2,174	2,174	2,174
Wells	1,211	1,050	937	937
Pumps	395	324	290	290
Water Treatment Plant	2,719	2,406	2,109	1,992
Intermediate Storage	0	280	713	1,168
Subtotal	(6,499)	(6,234)	(6,223)	(6,561)
O&M Costs				
Transmission Line	96	96	96	96
Wells	536	531	527	527
Power	1,097	987	938	938
Water Treatment Plant	4,214	4,198	4,182	4,173
Intermediate Storage	0	40	55	65
Subtotal	(5,943)	(5,852)	(5,798)	(5,799)
Total Cost	12,442	12,086	12,021	12,360
For Population = 60,000				
Investment Costs				
Transmission Line	2,705	2,174	2,174	2,174
Wells	1,740	1,532	1,393	1,393
Pumps	571	563	496	496
Water Treatment Plant	3,816	3,376	2,924	2,694
Intermediate Storage	0	358	911	1,718
Subtotal	(8,832)	(8,003)	(7,898)	(8,475)
O&M Costs				
Transmission Line	120	96	96	96
Wells	800	794	789	789
Power	1,593	1,830	1,718	1,718
Water Treatment Plant	5,390	5,369	5,347	5,336
Intermediate Storage	0	43	60	71
Subtotal	(7,903)	(8,132)	(8,010)	(8,010)
Total Cost	16,735	16,135	15,908	16,485

[a]All costs are present worths and are for 50 years of operation.

In addition, there appears to be a substantial decrease in the power costs. This is due to the reduction in pump horsepower and to the larger ratio of power consumption to the installed horsepower which are factors in establishing the unit cost of electricity (West Penn Power Company, 1971). There is, however, little change in the total amount of electricity used.

6.2.3 Time-Varying Demand Center

Calculations for a system with a growing demand center are complicated by increasing water demands and changing consumption rates during the period of analysis. The installed volume of intermediate storage should be based on the year having the largest consumption, ususally the last year during the planning horizon. In addition to determining the system's design capacity and the ultimate volume of intermediate storage, it is necessary to determine when and if intermediate storage should be stage-constructed. The general procedure for these analyses is as follows:

1. Determine the consumption rate-duration curve for the demand center and the storage capacity tradeoff functions as for the time-invariant demand center. Unless the consumption rate-duration curve varies during the planning period, which is an unpleasant but realistic possibility, these two curves should be expressed in dimensionless form like Figures 6.1 and 6.2.

2. Assemble system costs and construct cost curves like those in Figures 6.3 and 6.4, producing a set of curves for various demands along the planning period. The counterpart to Figure 6.3, would have the same cost curves but would contain a series of isoquants.

3. Investigate various alternatives of time phasing and stage-incrementation of intermediate storage. Thus, a system of relatively

low design capacity could be operating adequately but at a low load factor during the early years of the planning period. As the water demand increases, only the intermediate storage facilities would be increased.

Instead of the multitude of cost curves that could have been produced for several stages during the planning period, a decision matrix for a demand center having a linear growth pattern with a 125 percent increase in population from 25,000 to 56,000 during a 50 year period of analysis is presented in Table 6.2. For this system the least-cost combination of intermediate storage volume and design capacity remains relatively constant for different periods of delay before installation of the storage. Furthermore, the total system costs for a particular combination of intermediate storage and design capacity do not vary greatly with different delays for the hypothetical ground water system analyzed. Once the ratio of design capacity to average daily consumption rate decreases below 1.60, however, the volume of intermediate storage required increases rapidly and the total project cost increases accordingly.

For the time-varying demand center used in this example, the least-cost time of installation of the storage was 10 years (see Table 6.2). This delay is unique for every system and must be individually determined. The system analyzed showed no significant advantage in staging the construction of the intermediate storage rather than building the total required volume of storage initially -- this being due to the economy of scale for the construction costs and the cost savings due to discounting from the original 10 year delay before initial construction.

Table 6.2 indicates that the least-cost solution calls for an ultimate ratio between design capacity and average daily consumption rate of 1.60 or

Table 6.2. Decision Matrix of Total Project Costs, In $1,000's

Ratio of Design Capacity to ADCR	Years of Delay Before Installation of Intermediate Storage				
	0	5	10	15	20
2.20	12,546[a]	12,546	12,546	12,546	12,546
2.10	12,495	12,459	12,423	12,444	12,448
2.00	12,373	12,263	12,205	12,306	12,320
1.90	12,181	12,126	12,055	12,184	12,189
1.80	12,180	12,056	11,946	12,151	12,085
1.70	12,153	12,001	11,860	12,082	12,169
1.60	12,222	12,018	11,847[b]	12,087	12,151
1.50	12,325	12,140	11,947	12,201	12,309

[a]All costs are present worths and are for 50 years of operation.

[b]The least cost alternative.

slightly higher, corresponding to the same optimal load factor of about 60 percent found in the invariant demand case. Since the average daily demand for a population of 56,000, at 150 gpd per capita, is 8.4 mgd, the optimal system design capacity should be 13.4 mgd and the volume of intermediate storage about 17 mg.

6.3 Aversion of Risks in Population Forecasts

Because the population at the end of the design period is uncertain, the sizing of a water supply system at the start of the period of analysis will always involve some risk. Previously, in Chapter 3, consumption rates were assumed to be proportional to the population. Because it is difficult to reduce the uncertainty of population forecasts, this section will illustrate how intermediate storage can be used to reduce the severity of the risk, given an undesirable outcome in future population projections.

6.3.1 Population Forecast Errors

Several analytical methods have been used for making population forecasts. All of these methods project into the future from recorded historical populations. Nevertheless, the forecasting of population is a risky undertaking no matter what method is employed. This premise is supported by the forecast errors shown in Table 6.3 which were adapted from McJunkin (1964). McJunkin gave no indication as to the sign of the error, that is, an underprediction (-) or an overprediction (+). The forecast error, according to McJunkin, is defined as

$$\text{Forecast Error (\%)} = \frac{\text{Pred. Pop.} - \text{Act. Pop.}}{\text{Pred. Pop.}} \times 100 \qquad (6.1)$$

Table 6.3. Population Forecast Errors[a]

Forecast Method	Average Error, in Percent	
	10-Year Forecast	20-Year Forecast
Graphic Comparison[b]	34.9	61.8
Geometric Projection[c]	33.0	61.0
Arithmetic Projection[d]	14.2	18.8
Ratio[e]	9.3	15.6
Logistic Curve[f]	8.8	10.6

[a]The table is adapted from McJunkin (1964).

[b]A graphical extrapolation of past population growth.

[c]Projections which show an exponential growth pattern.

[d]Projections which show a linear growth pattern.

[e]Here the projected population of a study area is compared by a simple ratio to the projected population of a region.

[f]Projections which show a S-shape growth pattern.

Berthouex and Polkowski (1970) concluded that no general statement could be made about the sign of the error. They did state, however, that optimistic and pessimistic population forecasts were related to the economic optimism prevailing at the time the forecasts were made. Furthermore, they stated that it is impossible to know whether a forecast made today will be too high or too low. Berthouex and Polkowski did find that there was approximately a one percent error in forecasting for each year of prediction; in effect, a 20 year forecast would likely have a 20 percent error.

6.3.2 Minimization of Losses Due to Forecast Errors

When population growth is incorrectly forecast, the normal pattern of costs for installing and operating a water supply system will be distorted. If the system is too small due to an underpredicted population, the O&M costs will generally be greater than for a system designed for the actual population. If the population is overpredicted, the initial investment will generally be greater than for a system designed for the actual population. A flexible water supply system would minimize added costs due to improper sizing of system components.

A cost matrix was used to determine if intermediate storage reduces the expected cost for a municipal water supply system, given the population forecast uncertainties. To develop the necessary payoff matrix, the following information is needed: 1) an assumed or calculated probability distribution function of population at the end of the period of analysis (O_j); 2) designs for the alternative systems to be evaluated (X_i); and 3) the payoffs or costs for each possible system outcome (C_{ij}).

6.3.2.1 Decision Function

The type of decision function that can be used best to select the "most flexible system" from a set of alternatives depends on the type and quality of information regarding the size of future populations (outcomes). The state of knowledge of future populations can be assumed to fall into one of three categories:

1) Certainty: the size of the population at the end of the period of analysis is known;

2) Risk: it is possible to estimate the probability of occurrence for each outcome in population at the end of the period of analysis;

3) Uncertainty: the probability of occurrence for any outcome of the population at the end of the period of analysis is unknown (Ackoff and Sasieni, 1968).

According to Berthouex and Polkowski (1970) the outcome of population forecasts is uncertain. This statement is not 100 percent correct -- there is generally some basis for population forecasts. Some information is usually available from which the probability distribution of population for any particular region can be estimated. For example, Young et al. (1972) developed probability distribution functions for 10 and 20 year forecasts using the historical census data for the states of Maryland and Virginia. Thus, estimates of future population for regions in these two states are no longer uncertain, but merely involve some specified risk of error. This assumes that forecast errors will continue to be distributed as they have in the past. In other regions where a study of the probability distribution function of forecast errors is not available, the problem is still one of uncertainty.

One decision function that can be used under uncertainty is the equal-likelihood criterion (Starr, 1963). According to this criterion each outcome is assumed to be equally likely to occur. This, in effect, reduces the decision problem from one of uncertainty to one of risk, where the decision rule most often used is that the alternative with the lowest expected cost (for the minimum case) is the best alternative.

There are other decision criteria that can be used for decision making under uncertainty. The decision criteria at the opposite ends of the spectrum are those of pessimism and of optimism. Using the criterion of pessimism for the case of minimizing the costs, the purpose of this study, we select the alternative having the minimum cost when only the largest cost for an alternative of any of its possible outcomes is assigned to that alternative. This is called the minimax solution. For this analysis the criterion of pessimism would always select the alternative that would best meet a population underprediction. Conversely, the criterion of optimism selects that alternative having the minimum cost when only the smallest cost for an alternative of any of its possible outcomes is assigned to that alternative or the minimum solution. The criterion of optimism would always select that alternative that would best meet a population overprediction. Since both of these decision criteria would always select one extreme or the other, they were rejected in favor of the equal-likelihood criterion. Furthermore, any procedure developed using the equal-likelihood criterion can be easily adapted to decision making under risk if a probability distribution function of forecast errors is developed.

When using the equal-likelihood criterion the decision rule is to select the alternative having the smallest expected cost. The expected cost for any alternative can be written as

$$EC_i = \sum_{j=1}^{n} d_j \, C_{ij} \tag{6.2}$$

where EC_i is the expected cost the <u>ith</u> alternative, d_j is the probability of the <u>jth</u> outcome, C_{ij} is the total cost of the <u>ith</u> alternative for the <u>jth</u> outcome, and n is the number of possible outcomes.

6.3.2.2 System Alternatives Compared

The supply alternatives compared under population forecast risk conditions were the following:

X_{1a} and X_{1b}: Exclusive ground water supply with and without intermediate storage, respectively.

X_{2a} and X_{2b}: Exclusive surface water supply from reservoir C, with and without intermediate storage, respectively.

Each alternative was analyzed for 50 years of operation. The physical characteristics of the two systems analyzed were described in Chapter 3.

6.3.2.3 Population Outcomes

The initial population assumed in this subsidiary study was 20,000, estimated to increase to 44,500 during a 50-year period of analysis. The reason for using such a low base population was to allow the two exclusive supply sources to provide the entire water supply even under the most severe underpredictions considered. With the larger population postulated in Chapters 4 and 5 for the hypothetical system, integrated use of surface and ground water would have been required and the analysis would have been unnecessarily complex. It is likely, however, that intermediate storage would show benefits also in integrated use systems if it does so for single source systems.

For the purposes of this analysis it was assumed that all the possible population outcomes could be represented by 5 outcomes. These are 22,250, 33,375, 44,500, 55,625, and 66,750, with each outcome being equi-probable. The populations are associated respectively with forecast errors of +50, +25, 0, -25, and -50 percent. This range of forecast errors was selected because Berthouex and Polkowski (1970) found that there was approximately a one percent error in forecasting for each year of prediction.

6.3.2.4 Development of Comparative Cost Matrix

Table 6.4 is a cost matrix for the outcomes and alternatives described in sections 6.3.2.2 and 6.3.2.3. The addition of intermediate storage to the alternatives using exclusive ground water X_{1b} and exclusive surface water X_{2b} reduced the expected costs by 4.2 percent and 2.5 percent respectively. Approximately 80 percent of the cost savings were in the investment costs for alternative X_{1b}, while only 65 percent of the cost savings were in the investment costs for alternative X_{2b}. This can be attributed to the fact that intermediate storage can reduce the number of wells required by the ground water system, but there can be no reduction in the reservoir capacity for a surface water system -- the reservoir is initially built to its full capacity. The greatest cost savings was for the outcome with the largest underprediction, 0_1. In this case there was a 7.6 percent cost savings for alternative X_{1b} and 3.9 percent for alternative X_{2b}.

Table 6.4. Cost Comparison Matrix of Water Supply Systems With and Without Intermediate Storage

	Information About Outcomes					
Outcome	0_1	0_2	0_3	0_4	0_5	
Initial Population	20,000	20,000	20,000	20,000	20,000	
Final Population	66,750	55,625	44,500	33,375	22,250	
Growth Rate	2.3852	1.8176	1.2500	0.6824	0.1148	
Forecast Error	-50	-25	0	25	50	
Probability of Outcome	0.20	0.20	0.20	0.20	0.20	
Alternative Systems	Total Costs for Outcomes, in $1,000,000					Expected Value
(1)	(2)	(3)	(4)	(5)	(6)	(7)
X_{1a} (Without Storage)	13.401	11.900	10.797	9.781	9.008	10.977
X_{1b} (With Storage)	12.376	11.221	10.414	9.523	9.008	10.508
X_{2a} (Without Storage)	12.227	11.015	9.928	9.026	8.513	10.142
X_{2b} (With Storage)	11.750	10.555	9.653	8.992	8.513	9.893

Among the alternatives compared, X_{2b} appears to result in least overall costs. The comparatively high costs of ground water supply are due to the assumed softening requirements and would change under slightly different assumptions. Table 6.4 should therefore be looked upon as a cost comparison between schemes with and without intermediate storage, rather than between surface and ground water use.

6.4 Conclusions Regarding the Use of Intermediate Storage

In physical terms, the use of intermediate storage provides water for days having high consumption while allowing a reduction in the design capacity of the system. The use of intermediate storage will neither change the volume of water extracted from the source nor substantially change the time pattern of water extraction. Essentially, increased use of intermediate storage will reduce the size of certain system components by adding storage for treated water -- the stored water being used to satisfy days with high consumption rates.

As demonstrated by the use of the hypothetical ground and surface water systems, it appears that cost savings can be realized through a reduction in the size and number of components such as pumps, wells, transmission lines, and units within the water treatment facility. It was found that a reduction in the transmission line diameter made possible by the added storage does not necessarily lower the cost of the system -- the savings in pipeline construction cost being mostly offset by the increase in power costs. It was also shown that increased use of intermediate storage can reduce the penalty of population forecast errors. This is especially important for regional systems involving longer conveyance distances.

CHAPTER 7

FEASIBILITY OF INTEGRATED USE - SENSITIVITY STUDY

The goal of the following analysis is to investigate the conditions under which integrated use schemes are more advantageous than single source developments. The general strategy of the analysis is to observe variations caused by perturbations in selected input parameters and then to deduce some general guidelines from the numerical results.

Young et al. (1972) found in their study of water supply systems that economic variability was the most important factor affecting planning uncertainty. Hydrologic factors even though subject to considerable uncertainty themselves, tended to balance out if the planning period was reasonably long.

For the purpose of the sensitivity study the hypothetical example system was reduced to a simple two-source system consisting of the aquifer and reservoir B. This measure makes calculations less voluminous without sacrificing essential information.

For a non-constraint situation, both aquifer and reservoir have the potential to satisfy the demand center's total water requirements; the feasibility of integrated use depends on the economic competitiveness between the ground water and the surface water source. The economic competitiveness of a water source is determined by a number of factors introduced previously as physical input parameters and external decision variables.

For a surface water source these factors include:

1. The geographical location of the surface water source with respect to the point of water use. These parameters include distances,

and differences in elevation and nature of terrain, which influence the competitive position of a surface source through water conveyance costs.

2. The total volume of surface water available as well as the stability of a baseflow determine the reservoir capacity necessary to produce a certain yield. A small stream with a substantial baseflow might be a more competitive water source than a larger stream with less favorable low flow characteristics.

3. The quality of the reservoir site, dependent on valley configuration, geological conditions, and relocation requirements largely determines the magnitude of reservoir construction costs.

4. Reservoir costs may be considerably lower if the required storage volume can be provided within the framework of a multi-purpose scheme.

5. The quality of surface water determines treatment costs and is a decisive factor in enhancing or retarding the competitiveness of surface water supply.

6. Existing facilities may be incorporated into a new design and thus reduce surface water development costs.

For a ground water source the important factors include:

1. The distance and difference in elevation and nature of the terrain between the aquifer and the point of use which influence the costs of a ground water supply in the same way as a surface water supply.

2. Physical aquifer characteristics such as permeability, specific yield, or initial water table depth influence drawdowns and, hence, pumping costs.

3. The ground water quality may or may not play a large role in determining the costs of ground water supply.

4. The existence of ground water supply facilities at the beginning of the planning horizon can be an advantage if these facilities are incorporated into the new scheme.

Other factors not directly related to the source of the supply are the following:

1. The discount rate used to express the time value of money has a significant impact on system expansion. Planning with a low discount rate favors alternatives with large initial expenditures, whereas high discount rates favor more flexible alternatives with stage construction.

2. The uncertainty involved in projecting future water usage calls for keeping expansion strategies as flexible as possible. For this reason schemes adaptable to change and stage construction may be preferable to alternatives which require large initial investments in spite of lower total costs.

3. Probable impacts of technological advances lead to the same considerations as the uncertainty in predicting water requirements.

4. Environmental considerations may make the construction of a reservoir less desirable.

The integrated effect of all these factors determines the competitive position of one source relative to the other. It is obviously impractical to test the impact of all of the above summarized parameters on the selection of least-cost alternatives, so only a limited sensitivity analysis was performed.

Water treatment is a single factor which influences the choice of a least-cost alternative considerably for the hypothetical system. Therefore, separate sensitivity studies were performed for four possible surface water-groundwater quality combinations. They are:

Quality combination 1: surface water needs full treatment (coagulation, flocculation, settling filtration, chlorination) and ground water requires chlorination only.

Quality combination 2: surface water is fully treated and ground water requires hardness removal plus chlorination.

Quality combination 3: surface water needs chlorination only and ground water requires hardness removal plus chlorination.

Quality combination 4: both surface and ground water require the same degree of treatment.

Within each of these quality combinations the change in optimal alternatives in response to perturbation in the source competitiveness was investigated. As representative parameters, the distances between reservoir and point of use and between aquifer and point of use were selected. Any other parameter or a combination of parameters could have been chosen. Numerical results would be different, but trends would have been the same.

7.1 Feasibility of Integrated Use under Quality Combination 1

In Chapter 5 alternative I AQ was found to be the most economical supply alternative for the hypothetical system with only chlorination required of the ground water. In the following analysis the competitive position of the aquifer is changed through variation of the distance between the aquifer and the treatment plant, the pumping lift, and the discount rate. All other parameters remain unchanged and as defined in Chapter 3.

7.1.1 Effects of Ground Water Pipeline Length on Total Costs

Figure 7.1 illustrates the sensitivity of least-cost solutions to LWEL, the distance from the wellfield to the treatment plant. As expected, costs for alternative I AQ rise as LWEL increases, and eventually approach the cost of alternative I B, the exclusive surface water supply, as LWEL approaches about 26 miles.

Cost of alternatives involving integrated use schemes also increase with distance to the aquifer, but at a smaller rate than the costs for alternative I AQ, because of the lower use of ground water under these schemes. As shown in Figures 7.2 and 7.3, a progressive shift to more surface water contributions takes place as ground water loses its competitive position. All supplementation by ground water in alternatives III B-AQ and III B-$\overline{\text{AQ}}$ is delayed, and conversely the timing of surface water supplementation is advanced. Eventually the integrated use alternatives will rely totally on surface water and merge with alternative I B.

7.1.2 Effects of Pumping Lift

It is of some interest to study how larger initial water table depth, DAQ, compares with LWEL as an indicator of economic ground water competitiveness. In Figure 7.4, ground water development costs which would occur if the demand center's total water supply requirements were to be met by the aquifer, were plotted for various initial depths to the water table and for a number of different values of LWEL.

According to Figure 7.4 every one-mile increase in LWEL results in an additional expenditure of $337,500. For a 100 ft increase in DAQ the costs rise by $375,000. Hence, a difference of 100 ft in initial water table

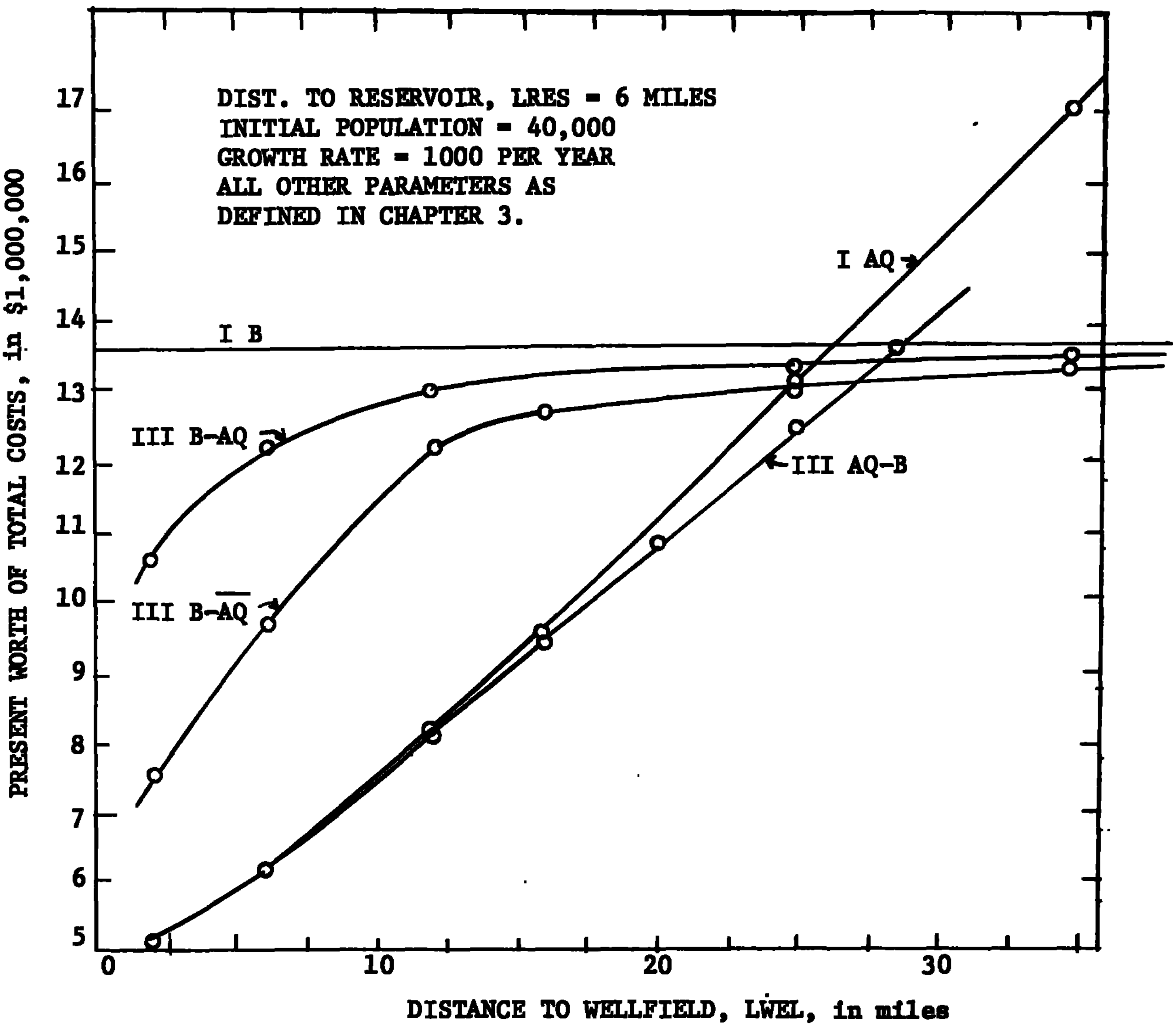

Fig. 7.1 Effect of Distance to Wellfield on Costs, for Water Quality Combination 1.

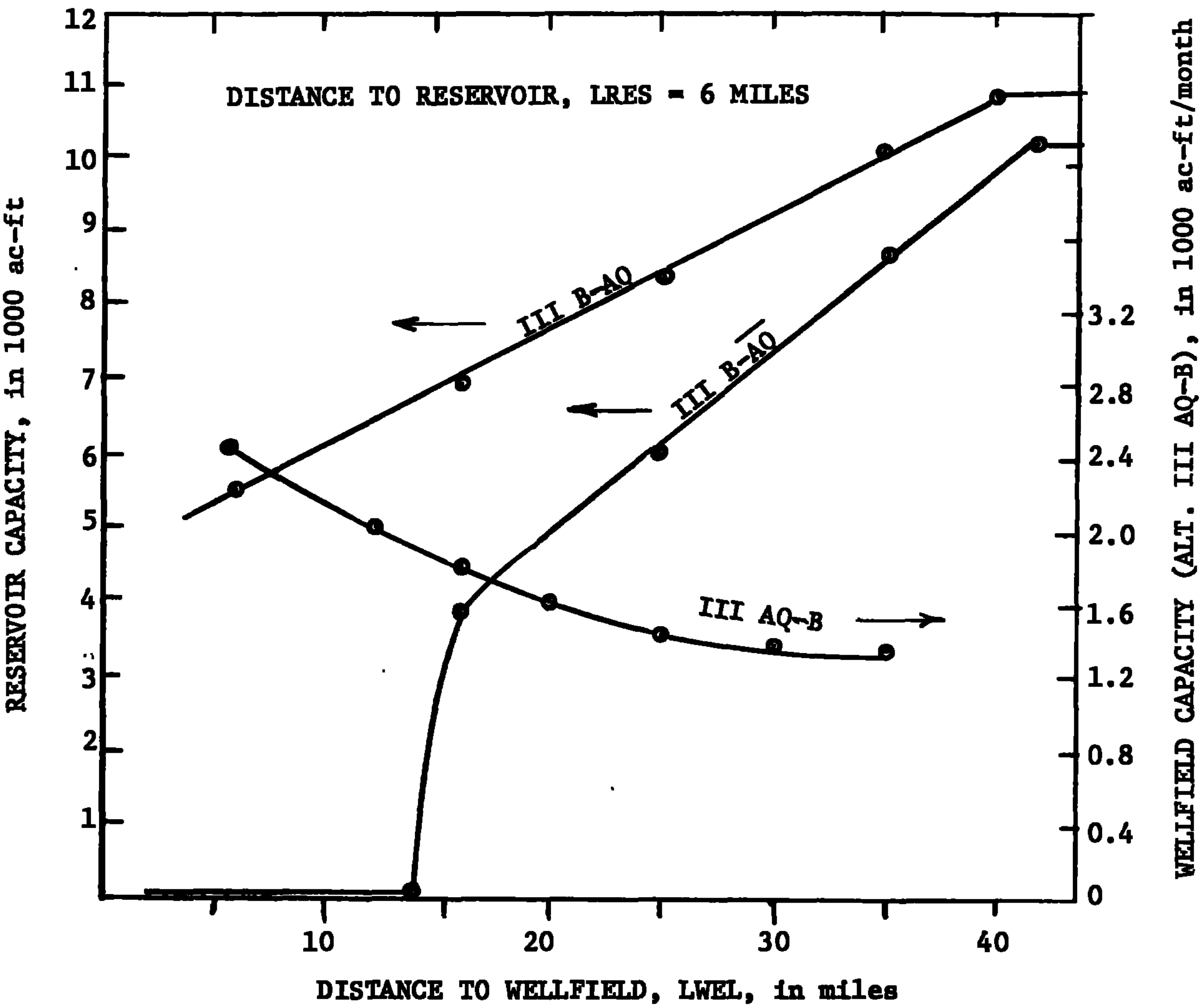

Fig. 7.2 Effects of Distance to Wellfield on Development Scale of Primary Source, for Water Quality Combination 1.

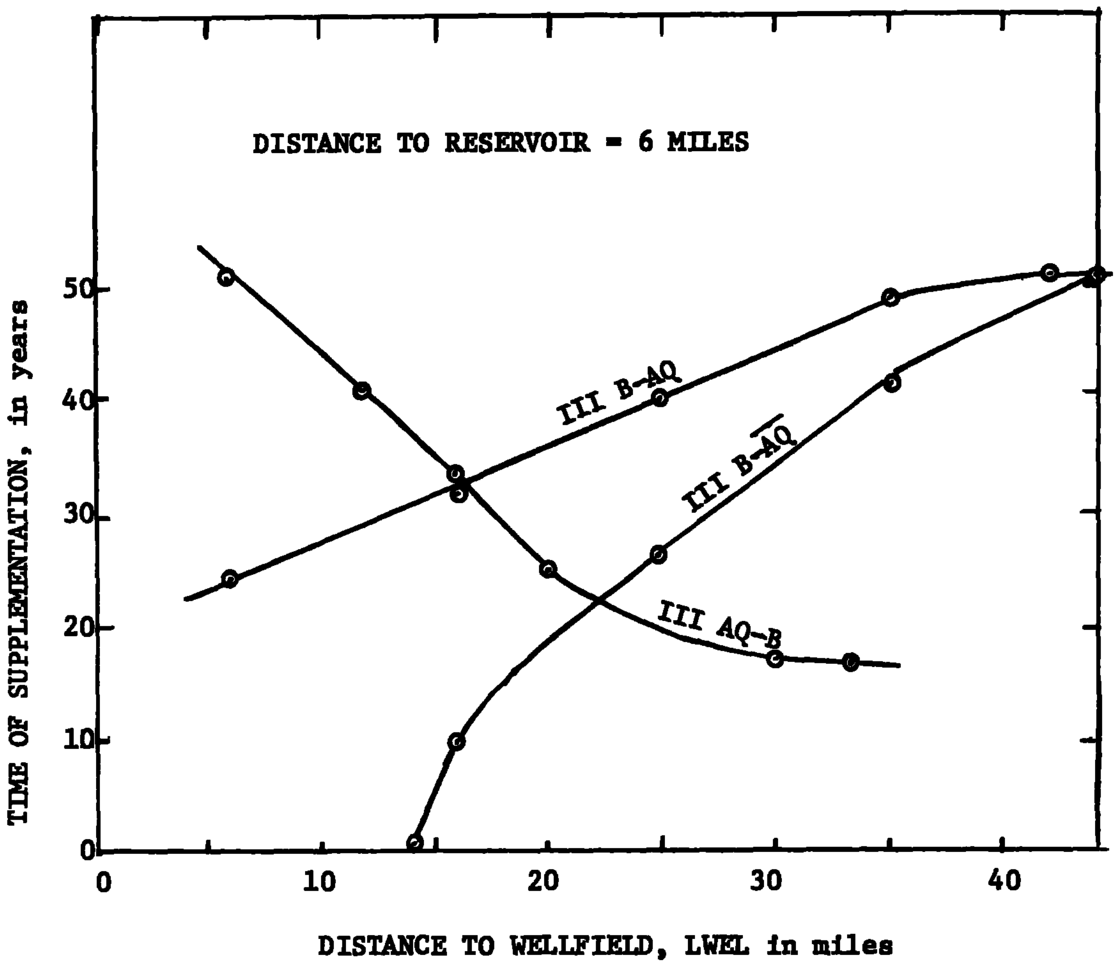

Fig. 7.3 Effect of Distance to Wellfield on Time of Supplementation of Primary Source, for Water Quality Combination 1.

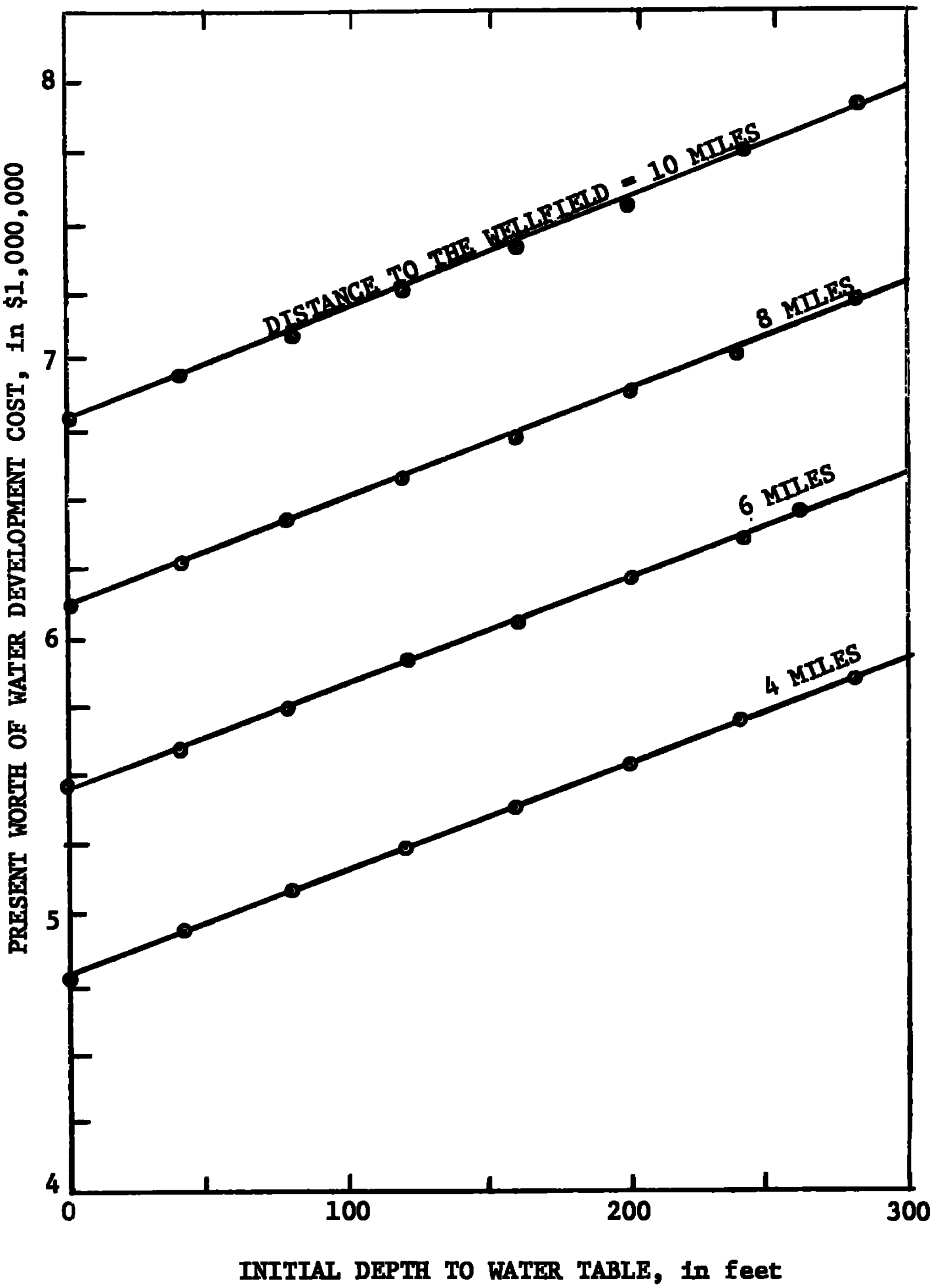

Fig. 7.4 Cost Tradeoff Between Distance to Wellfield and Static Water Table Depth

elevation has an effect on least-cost solutions equivalent to a 0.9 mile increase in wellfield distance.

7.1.3 Effects of Discount Rate

As a third step, the influence of discount rate on alternatives I AQ and III AQ-B is examined. Figure 7.5 illustrates the change in cost for those alternatives for various magnitudes of discount rates. A change of 1 percent in the discount rate from 4 percent to 5 percent for example, would decrease the present worth of costs for alternative I AQ by about 10 percent. The increasing cost difference between alternatives I AQ and III AQ-B evident from Figure 7.5 indicates that conditions for integrated use become more favorable with higher discount rates. Higher discount rates reduce the present worth of future investments, thus enabling surface water, in alternative III AQ-B, being the secondary source and to be installed later, to gain in economic competitiveness relative to the aquifer source. This increase in economic competitiveness is demonstrated by the change in least-cost source combinations as shown in Figure 7.6. With higher discount rates, smaller monthly wellfield capacities are called for, and the time of supplementation by the reservoir moves toward the beginning of the planning horizon.

The above analysis illustrates dramatically the impact of the discount rate on the planning of water resources allocation schemes.

7.2 Feasibility of Integrated Use Under Quality Combination No. 2

In this section the competitiveness of integrated use was investigated for the case in which the ground water hardness exceeds permissible standards. To make use of the characteristically low hardness of surface

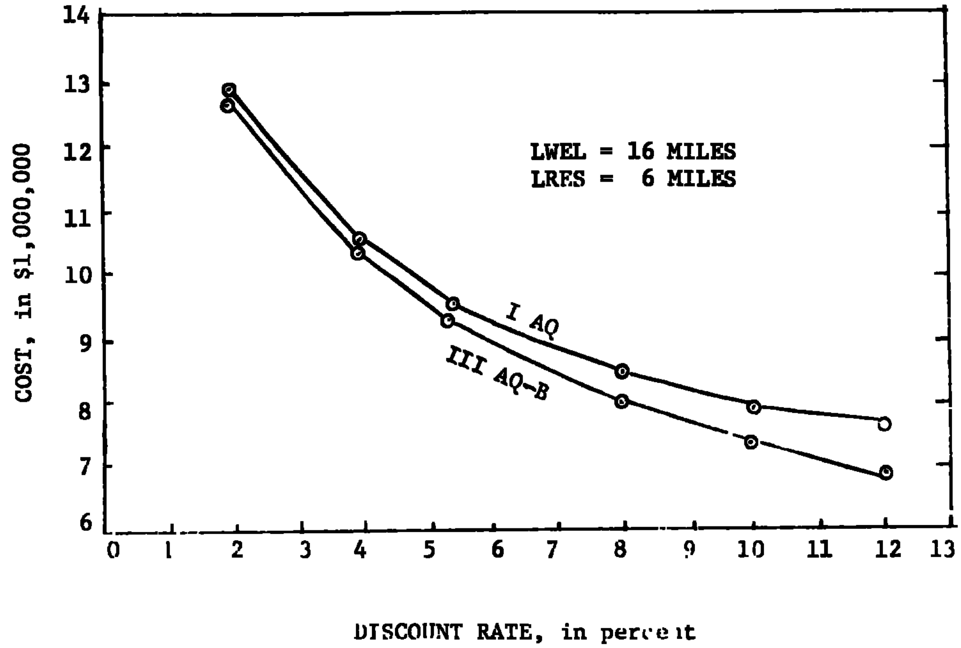

Fig. 7.5 Effect of Discount Rate on Costs, for Water Quality Combination 1.

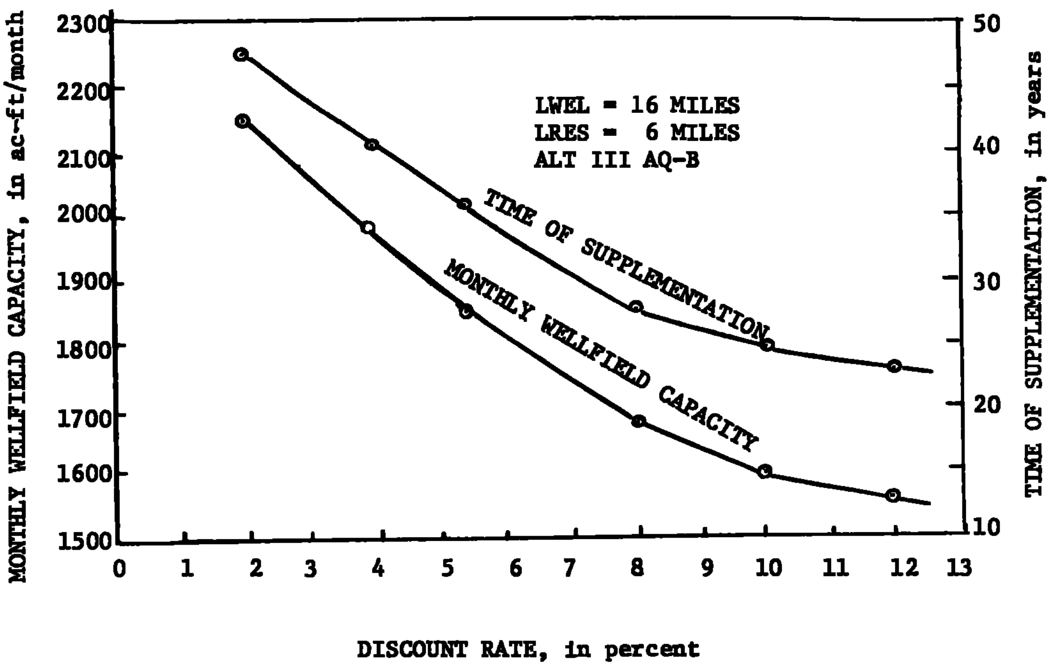

Fig. 7.6 Effect of Discount Rate on Least-Cost Source Scale and Timing for Water Quality Combination 1.

water and avoid high softening costs, the option of "mixed treatment" of surface and ground water after treatment was considered. Similar to the commonly used split treatment in which a given flow of water is divided, one portion treated, and both subsequently mixed again to obtain a blended product of desired quality, the ground water, which was assumed to be of acceptable quality except for its hardness, would simply bypass the treatment plant, mix with the fully treated surface water, and finally be subjected to chlorination. The mixing proportion would be adjusted in any given situation to yield a product of a hardness not exceeding the maximum permissible standard. This is discussed more fully in Appendix B.

With the hypothetical system of reservoir B and aquifer, both at 6 miles distance from the treatment plant, a surface water hardness of 40 ppm, a ground water hardness of 200 ppm, and a maximum permissible hardness of 120 ppm, the sensitivity of total costs to changes in distance LRES to the reservoir, distance LWEL to the aquifer, and ground water hardness was investigated.

7.2.1 Effects of Distance LRES to Reservoir B

The cost curves in Figure 7.7 illustrate the advantages of the "mixed treatment" option. Water treatment plays a large role in the total supply system costs. With the ground water bypassing all treatment except chlorination and requiring only moderate conveyance cost, it is certainly economical to use as much ground water as possible without exceeding the maximum hardness standard. For ground water hardness of 200 ppm and surface water hardness of 40 ppm, an optimal mixing ratio of 1 to 1 is determined, at least for the last years of the planning horizon, during which time both

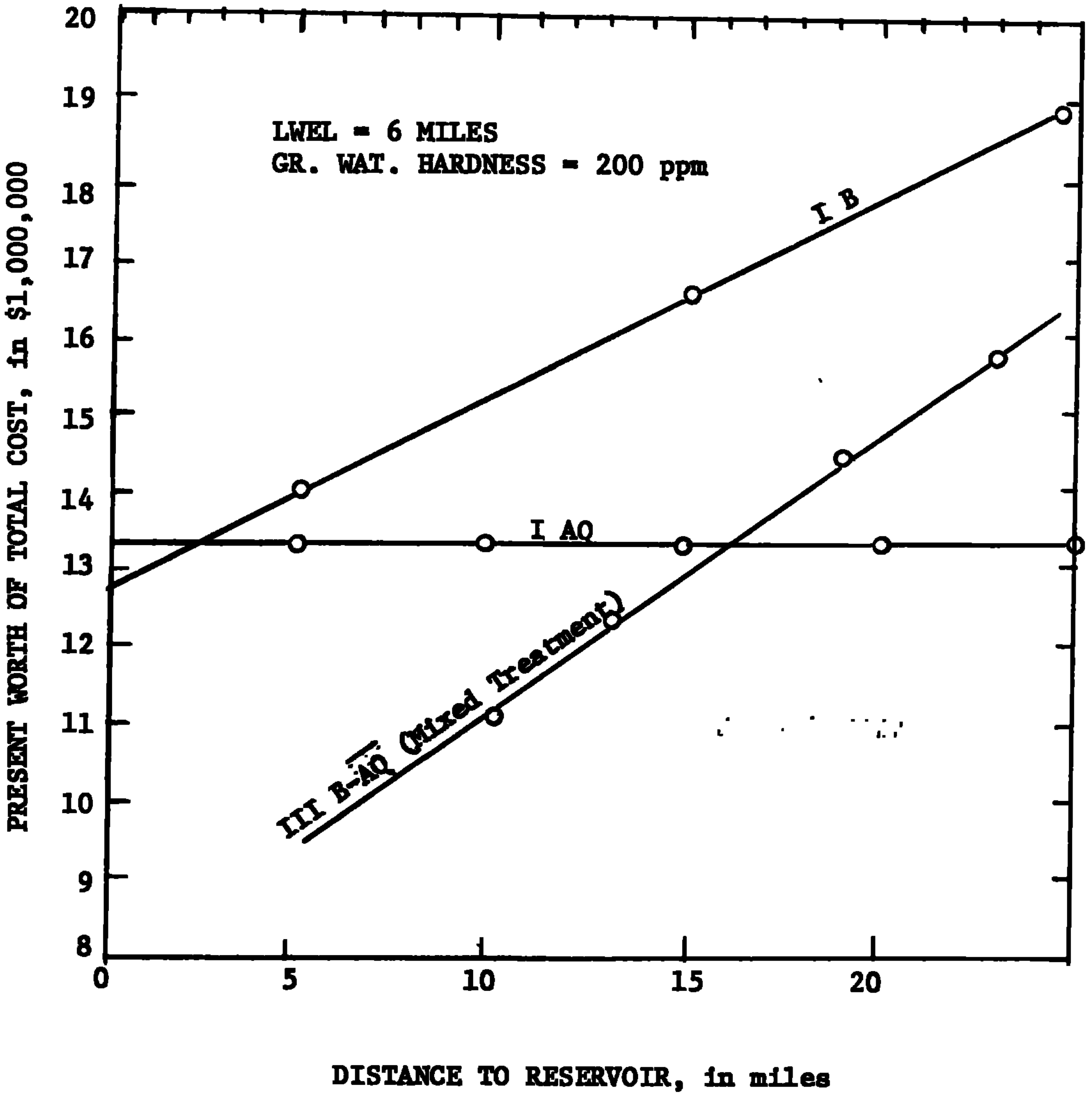

Fig. 7.7 Effect of Distance to Reservoir on Costs, for Water Quality Combination 2.

surface and ground water systems should have a capacity of 1,370 ac-ft/month. Due to the inflexibility of the surface water supply system, which under our assumptions is installed initially at full ultimate capacity, the surface-ground water mixing ratio during the early years of the planning horizon will be larger than 1 to 1. During the planning horizon, the mixing ratio decreases gradually as new wells are added.

Under the given conditions, the mixed treatment option is preferable to any other alternative until the distance from reservoir B to treatment plant exceeds about 23 miles, beyond which the exclusive ground water use (I AQ) becomes the least-cost alternative. For the mixed treatment option the least-cost combination involves building a small reservoir of 600 ac-ft of storage capacity on stream B and developing the well field from a monthly pump capacity of 500 ac-ft initially to 1,370 ac-ft per month in the final year of the planning horizon.

The above analysis demonstrates that integrated use is an efficient water supply scheme if expensive ground water treatment can be avoided through the mixed treatment process.

7.2.2 Effects of Distance LWEL to the Wellfield

When the distance to the aquifer is changed instead of the distance to the reservoir, a gradually increasing penalty for ground water use is to be expected, as is confirmed by Figure 7.8, in which the exclusive ground water use alternative I AQ is completely out of the competition, whereas the exclusive surface water use alternative IB begins to become attractive as the distance to the aquifer approaches 18 miles.

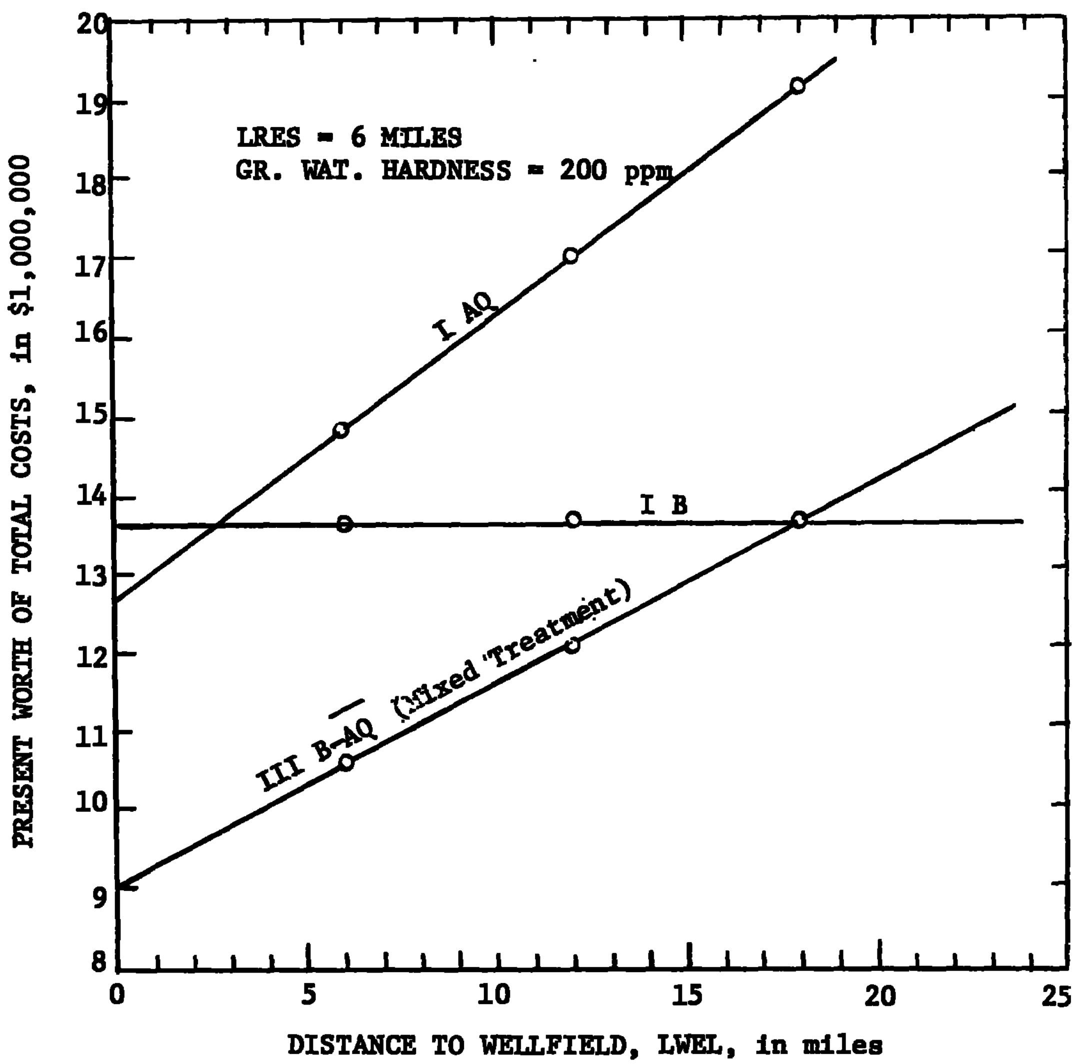

Fig. 7.8 Effect of Distance to Wellfield on Costs, for Water Quality Combination 2.

7.2.3 Effects of Ground Water Hardness

The third parameter to be varied under quality combination 2 was the ground water hardness. In Figures 7.7 to 7.10 the costs of alternatives I B, I AQ, and III B-AQ (mixed treatment) are rotating around the common pivot point for LRES = LWEL = 6 miles, ground water hardness = 200 ppm. As the hardness decreases, the mixing ratio of surface to ground water use will decrease until the mixed treatment option merges with alternative I AQ. As the ground water hardness increases beyond 200 ppm, the additional annual softening costs amount to about $75 per mgd of ground water treated for every ppm in excess of 200 ppm hardness. The result as shown in Figure 7.9, is a gradual shift toward larger mixing ratios and a sudden merger into the exclusive surface water use alternative I B at the point at which the benefits provided by the ground water supplementation do not offset the costs of wellfield and pipelines.

7.3 Feasibility of Integrated Use - Quality Combination 3

For quality combination 3 (surface water needs chlorination only and ground water requires hardness removal) the results are similar to those obtained for quality combination 2. Figure 7.10 shows that integrated use with mixed treatment is the least-cost expansion scheme, throughout the large range of distances LRES to the reservoir studied. For quality combination 3, the superiority of the integrated use scheme is not caused by savings in surface water treatment costs as in quality combination 2, but by savings in initial expenditures for reservoir construction if ground water can be used without incurring treatment expenses.

Least-cost source combinations in the III B-AQ integrated use scheme vary with the degree of competitiveness of the surface source expressed by

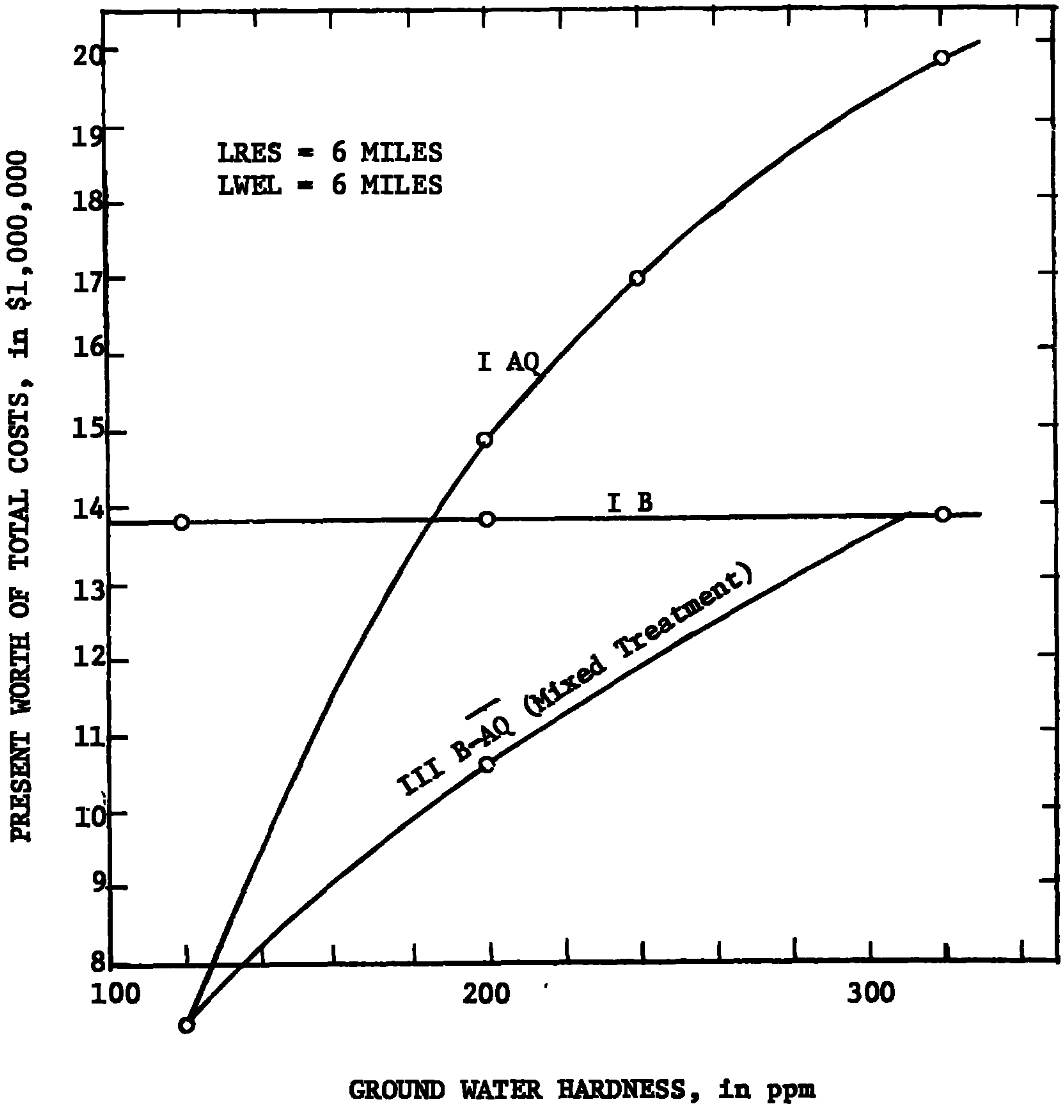

Fig. 7.9 Effect of Ground Water Hardness on Costs, for Water Quality Combination 2.

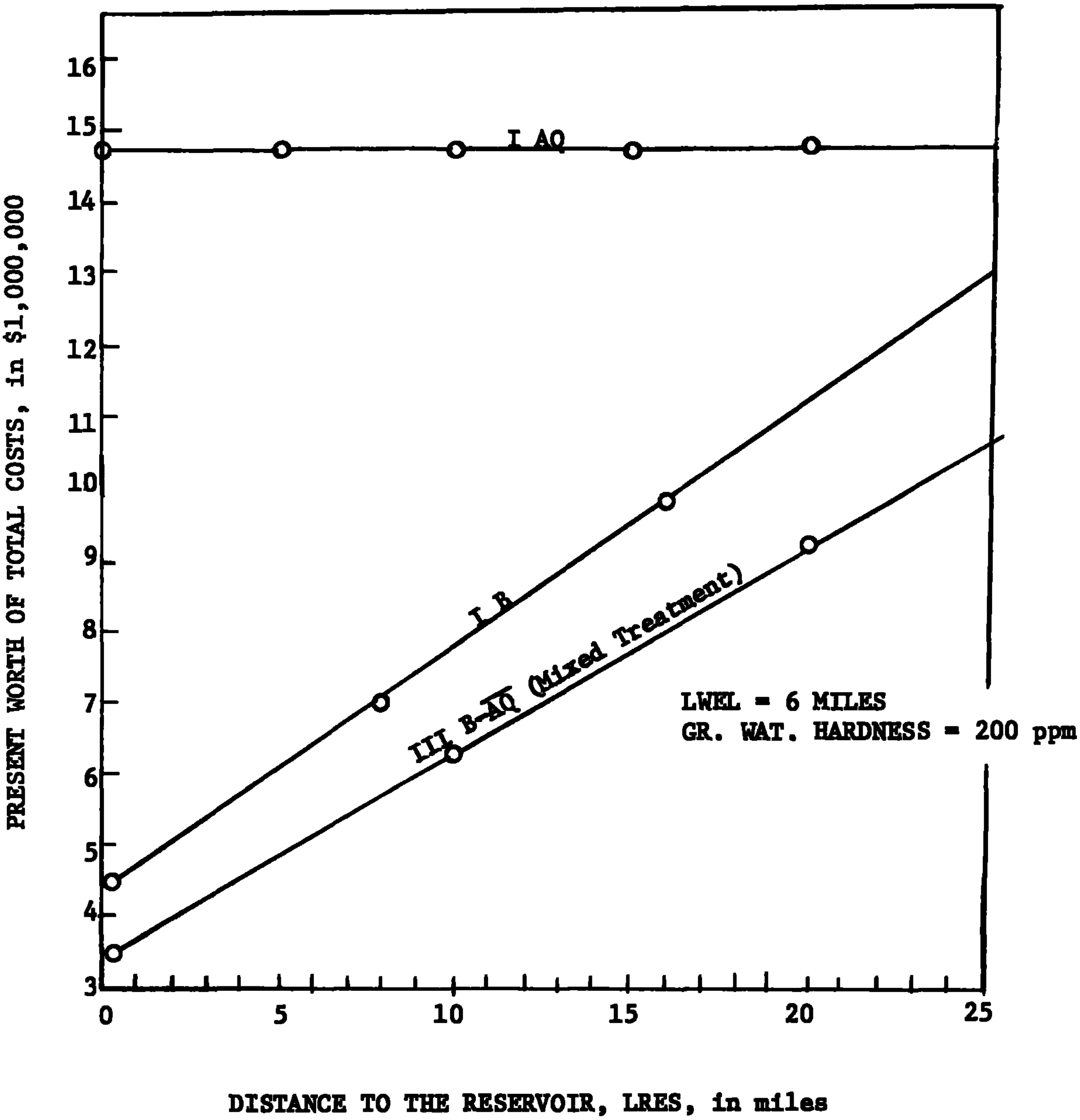

Fig. 7.10 Effect of Distance to Reservoir on Costs, for Water Quality Combination 3.

the distance to the reservoir. The relative contributions of surface and ground water under various degrees of surface water competitiveness are illustrated by the relationships between reservoir size, ultimate monthly pump capacity, time-of-supplementation, and distance to the reservoir as shown in Figure 7.11

The above analysis demonstrates that integrated use and mixed treatment is economically advantageous, even if the surface water requires chlorination only.

7.4 Feasibility of Integrated Use - Quality Combination 4

Finally, the feasibility of integrated use for a water source system with quality combination 4 (both waters require the same treatment) was investigated. Figure 7.12 shows the relationship between cost and distance to the reservoir, which is used again to describe the competitive position of the surface water source. Since both sources require the same treatment, treatment costs were deleted in this comparison and only the costs for source development and supply were considered.

The relationship presented in Figure 7.12 demonstrates that integrated use schemes with a high reliance on surface water tend to be more advantageous than single source development, as long as the distances to the two sources are not vastly different. The preference for integrated use schemes is caused by savings through avoidance of high initial reservoir expenditures.

7.5 Integrated Use Under Constraint Conditions

In situations in which neither the surface water nor the ground water source is able to guarantee the total water supply requirements during

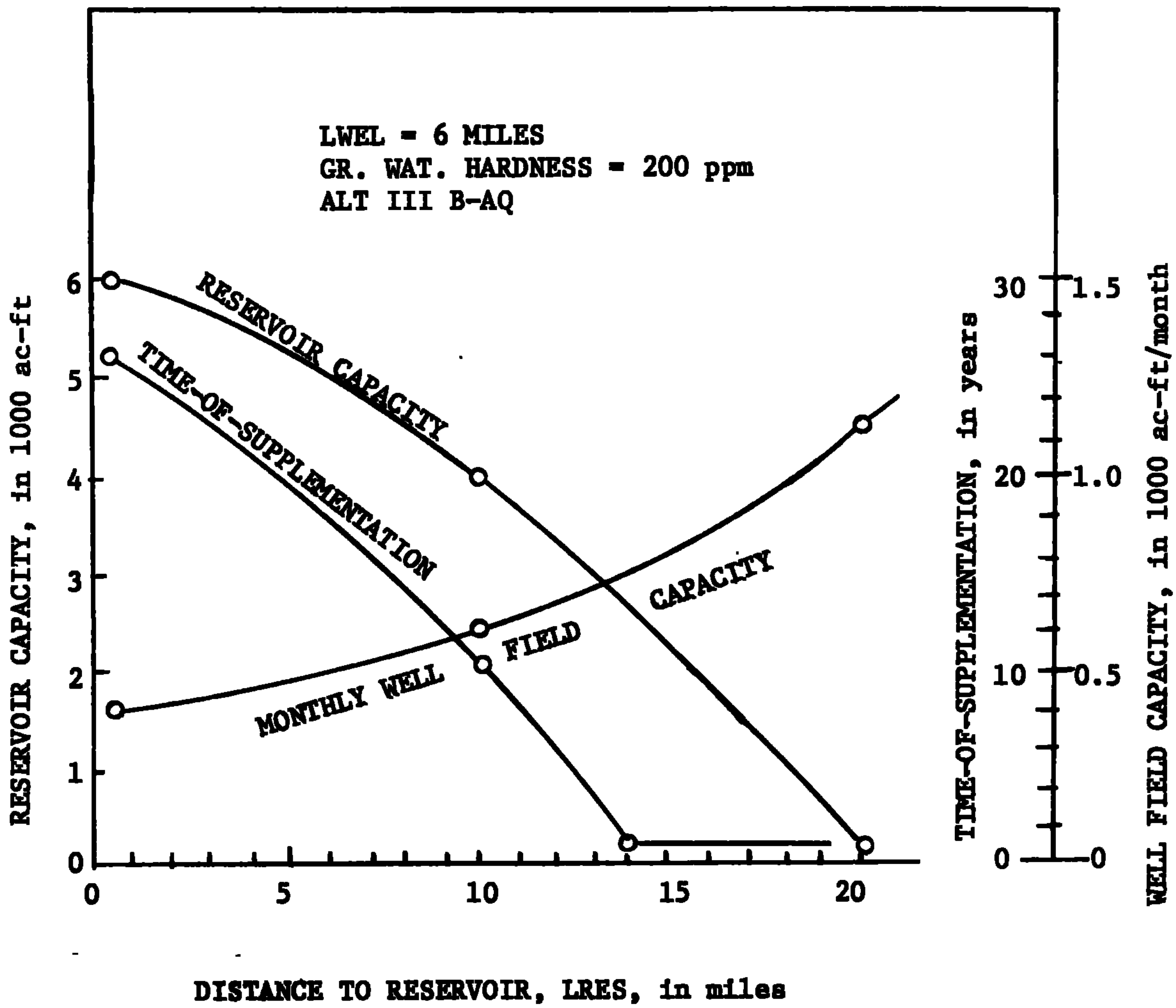

Fig. 7.11 Effect of Distance to Reservoir on Least-Cost Source Scale and Timing, for Water Quality Combination 3.

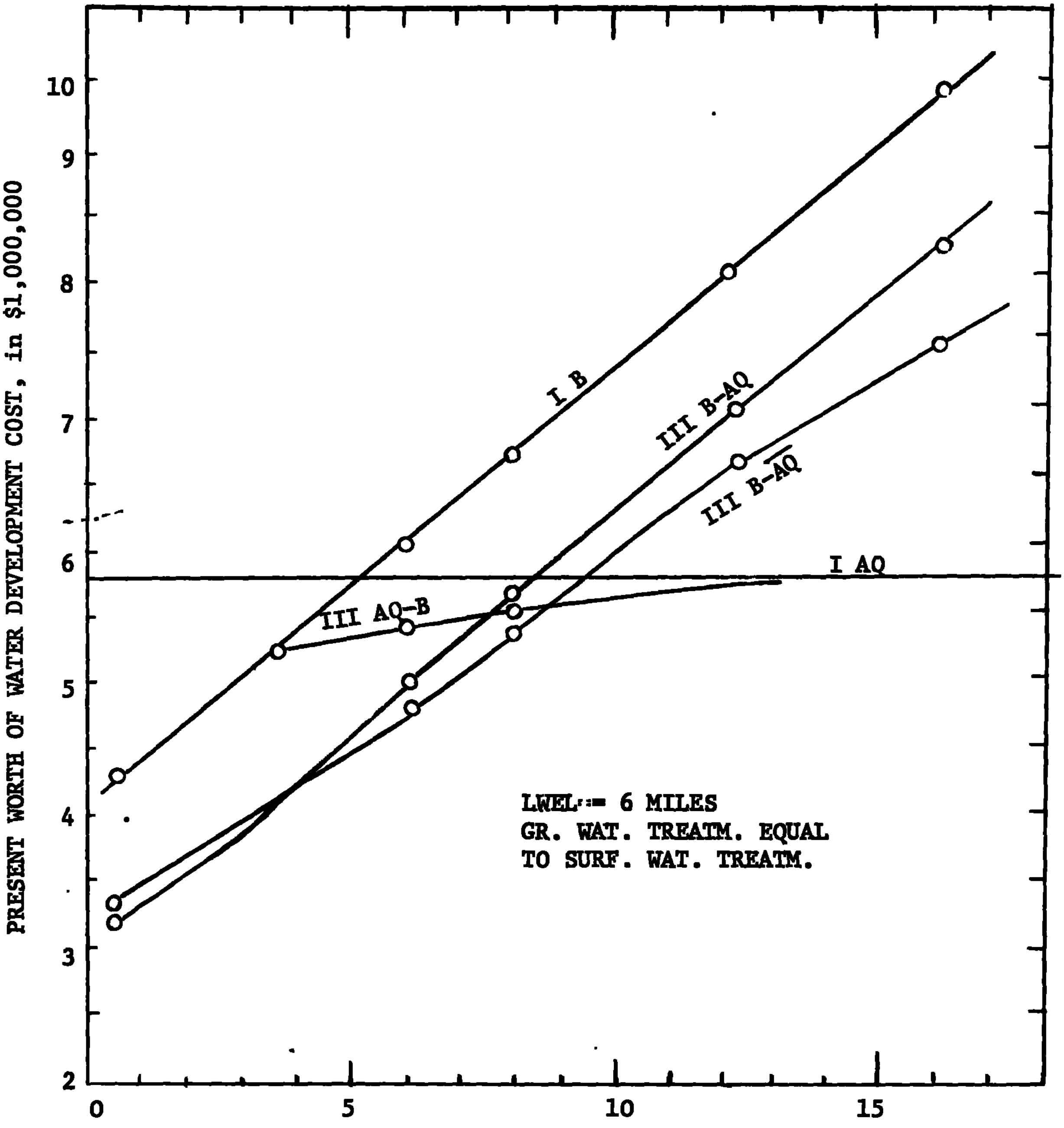

Fig. 7.12 Effect of Distance to Reservoir on Costs, for Water Quality Combination 4.

the entire time of the planning horizon, integrated use becomes a necessity. Water demand centers adopting an integrated use alternative for that reason may be less concerned with costs, but more with the operational alternative that results in most effective water conservation. In such a case one would be interested in initiating a scheme which utilizes the ground water resources most carefully. Of the three integrated use schemes dealt with in this study, alternative III AQ-B is the one requiring the largest volume of ground water. Alternative III B-AQ requires the least amount of ground water, because except for some preventive pumping, water from the aquifer is pumped only at times of actual surface water deficits.

Alterantive III B-AQ, though not necessarily economical under non-constraint conditions, might be the best operational scheme under conditions of limited water supply potential.

7.6 Summary of Results

The results of the sensitivity study demonstrate that integrated use schemes under certain circumstances are economically advantageous over single source development. Whether integrated use is feasible or not depends on the relative competitiveness between the surface and ground water sources. The single most important factor is the water quality of the source.

On the basis of the sensitivity study the following general planning guidelines may be formulated:

1. For a system with quality combination 1 (surface water needs turbidity removal and ground water requires only chlorination, initial aquifer development is the most economical alternative.

2. For a system with quality combination 2 (surface water needs turbidity removal and ground water requires softening), integrated use and mixed treatment is by far the most advantageous water supply scheme. Surface and ground water are developed simultaneously, keeping the surface-ground water mixing ratio at the lowest possible value.

3. For a system with quality combination 3 (surface water needs chlorination and ground water requires softening), integrated use and mixed treatment is recommended. The relative contribution of surface water varies with the competitive position of the ground and surface water source. Its lower limit is specified by the minimum surface-ground water mixing ratio.

4. For a system with quality combination 4 (both sources require the same treatment), integrated use is the most economical alternative, as long as the competitive position of the ground and surface water sources are about equal.

The findings in this chapter should encourage the practicing engineer to explore the possiblility of integrated ground and surface water use, whenever this is feasible.

CHAPTER 8

CASE STUDY - ELMIRA, NEW YORK

The water supply system of Elmira, New York, was chosen to demonstrate further the methodology developed in the hypothetical study. The time span between 1980 and 2030 was used as a planning period. The reasons for selecting this case study location include the following:

1. Elmira is a demand center which is expected to see fast growth in the future.

2. In the vicinity of Elmira good ground water as well as ample surface water sources are available.

3. Data required for this study were available and adequately documented.

Hazleton, Pennsylvania was considered for a second case study, but was dropped for the reasons cited in section 8.9.

8.1 General Description of Elmira

Elmira is located in Chemung County in South Central New York on the confluence between Newtown Creek and Chemung River, a major tributary of the Susquehanna River. The location of Elmira along with the stream network surrounding it is presented in the map of Figure 8.1. Water supplies in the Elmira area are provided by the Elmira Water Board. This utility serves the city of Elmira, the town of Elmira Heights, and the village of Horseheads. The total population served by the Water Board was estimated to be 73,000 in 1971 (Elmira Water Board, 1971).

The urban area of Elmira has experienced a steady growth in recent years, as expressed by a 50 percent increase in water use between 1960 and 1971.

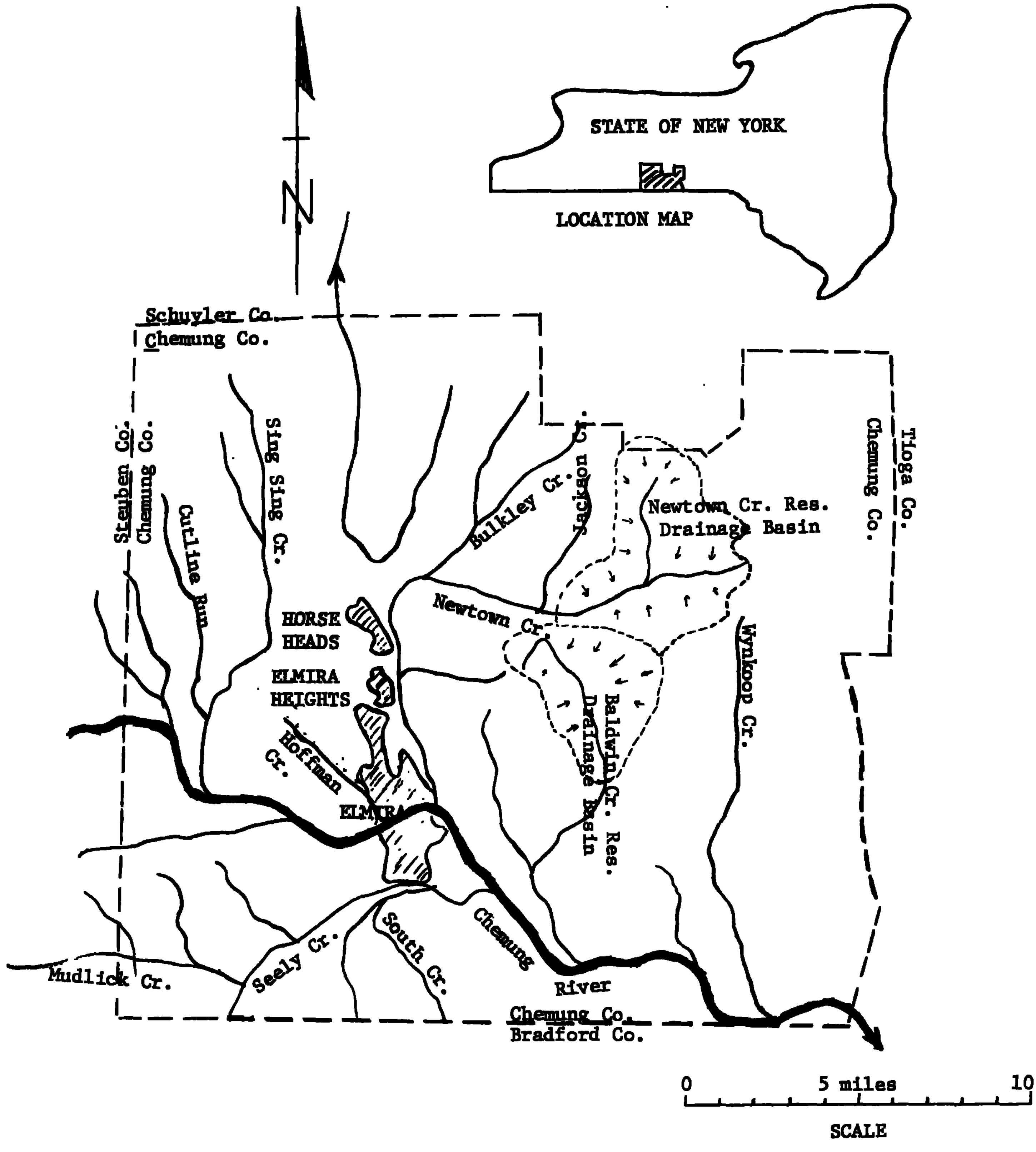

Fig. 8.1 Map of Chemung County and Elmira Area

Economic growth coupled with substantial increases in population and water requirements is expected to continue in the future.

8.2 Description of Water Resources in the Elmira Area

Both ground and surface water sources are available in Elmira. Surface water sources, as shown in Figure 8.1, are plentiful. Average annual yields in the neighborhood of 15-16 inches can be expected in the area. The major surface water source is the Chemung River. Its watershed has a size of 2,500 sq. miles and its average annual discharge is about 2,450 cfs according to records available since 1903. Discharges in the Chemung River are quite variable and the lowest discharge recorded was 49 cfs in August 1911. The second largest stream in the area is Newtown Creek. Its drainage area measures about 77.5 sq. miles, and its annual average flow recorded since 1938 is 85 cfs. The lowest flow in recorded history is 5 cfs and occurred in 1965. There are a number of smaller streams located in the vicinity of Elmira which may be considered as potential water sources. These include Hoffman Creek, Baldwin Creek, Jackson Creek, Bulkley Creek, South Creek, Seely Creek, Mudlick Creek, Cutline Run, and Sing Sing Creek. Unfortunately, none of these streams have gaging stations.

Even though the smaller creeks have no gages, it can be assumed with generality that their minimum sustained low flow may approach zero, and that they cannot be relied upon as water supply sources without holding reservoirs.

The only two seemingly suitable reservoir sites which could be found on maps of the vicinity of Elmira are located on Newtown Creek above Breesport and on Baldwin Creek above North Chemung. The location of these potential reservoir sites and their respective drainage basins is shown in Figures 8.1 and 8.2, and the statistics of the sites will be presented in Table 8.1, section 8.4.

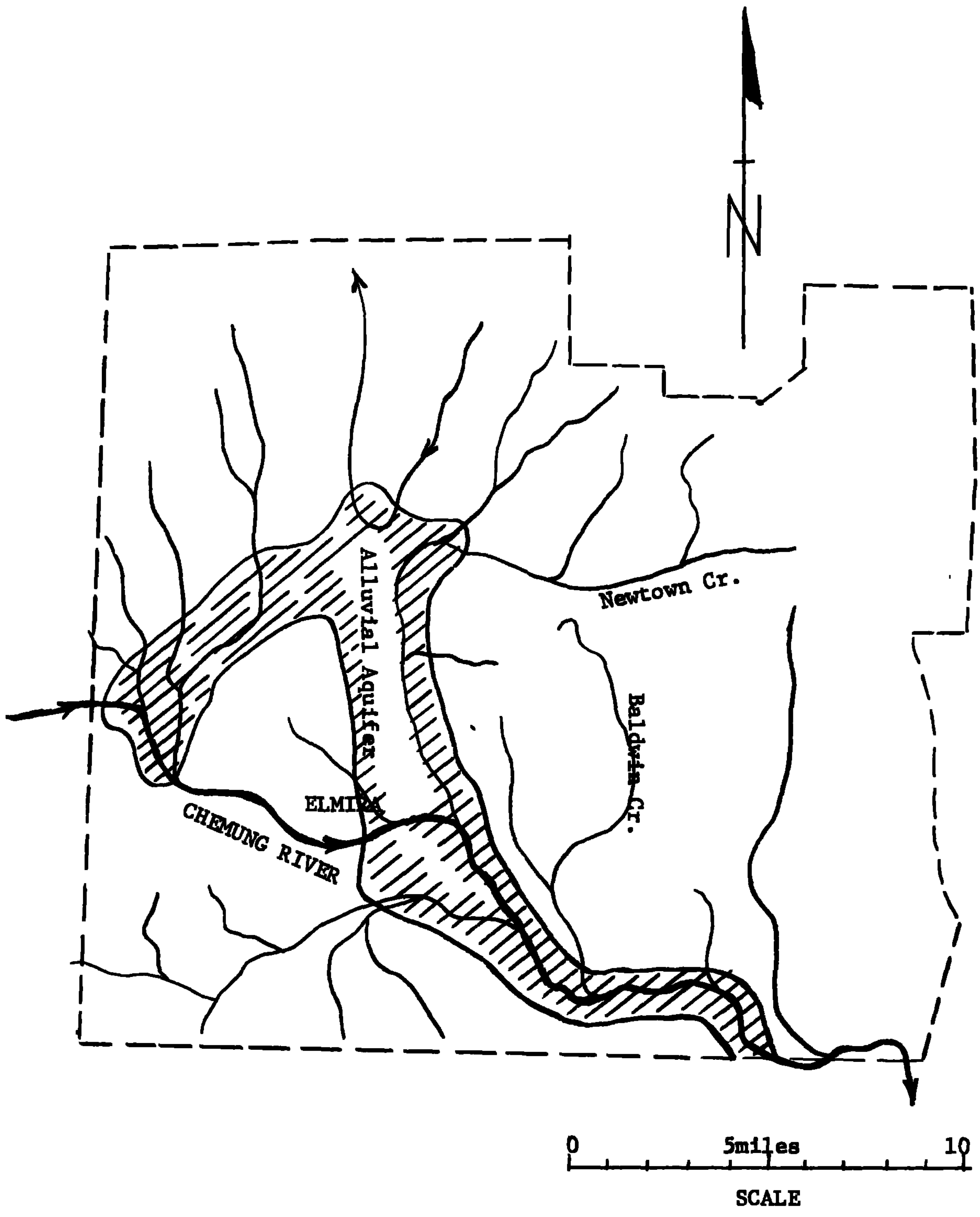

Fig. 8.2 - Map of Alluvial Aquifer in Elmira Area (Wetterhall, 1959)

Major ground water sources exist in the Elmira Region. Most of these aquifers are found in sand and gravel deposits along the river channels. A detailed study of the ground water sources in Chemung County was performed by Wetterhall (1959). The map in Figure 8.2 shows the major aquifers in the Elmira region as found by Wetterhall. The sand and gravel deposits below and in the vicinity of Elmira can sustain wells yielding more than 1,000 gpm. Aquifer thicknesses were found to vary between 30 and 400 ft. Permeability values ranging from 200 to 5,000 gpd/ft^2 were reported by the U.S. Army Corps of Engineers (1951). McNish et al. (1969) suggested that a realistic value of specific yield may be 0.2. McNish also found that aquifers and surface streams are connected hydraulically. Infiltration of surface water through the stream bed into the aquifers was estimated to be around 20 gpd/ft^2.

Wetterhall (1959) also studied the quality of ground water in the area. He reports mineral concentrations generally well below standards set for drinking water by the U.S. Public Health Service. Hardness, however, is rather high. The hardness of samples analyzed ranged from 32 to 480 ppm. More than 50 percent of the samples showed a hardness of more than 120 ppm.

8.3 Elmira's Water Supply System - Past, Present, and Future

Organized public water supply in the Elmira region was initiated in 1859 with the founding of the Elmira Water Company. The first source was Hoffmann Creek (see Figure 8.1). Supply was directly from the run of the river until a small reservoir was built in 1872. Soon Elmira's demand exhausted the potential of the small Hoffmann Creek and a pump station on the Chemung River was built. In 1897 a rapid sand filtration plant was constructed to treat water coming from both the Chemung River and Hoffmann

Creek. This plant, enlarged from time to time, is still in operation and is one of the oldest functional rapid sand filtration plants in the United States and the oldest in the State of New York. This system provided the total water supply of Elmira until 1960.

At that time all excess capacity of the treatment plant had been totally exhausted and the Elmira Water Board decided to tap the aquifer underlying the city. In 1961 a 3 mgd well was installed. Later more wells were drilled. In 1971 the city operated a total of 5 wells with a combined pump capacity of 11 mgd. Because the ground water is escessively hard, some hardness is removed by an ion exchange process before the water is pumped directly into the distribution system. In Elmira's present use scheme, ground water provides a certain baseflow and surface water furnishes the remainder of the water requirements. Figure 8.3 shows the water contributions of each source for Elmira's total annual volume of water consumption for the 1960-1971 period.

What does the future hold for Elmira? All predictions indicate that Elmira will experience a time of rapid growth in the next decades which will be accompanied by an equally rapid increase in water requirements. Two demographic forecasts for Chemung County are available from the literature. A survey performed by the State of New York (1968) predicts a 75 percent population increase for the 1970-2020 year period. Predictions made by the U.S. Army Corps of Engineers (1968) have indicated an even larger growth potential in the Elmira area. In that study the population is expected to grow by 140 percent in the next five decades. Both sources predict a linear growth pattern. Although the predictions are based on the total Chemung County population, the growth rates probably are also representative for the Elmira urban area, which contains the great majority of the County

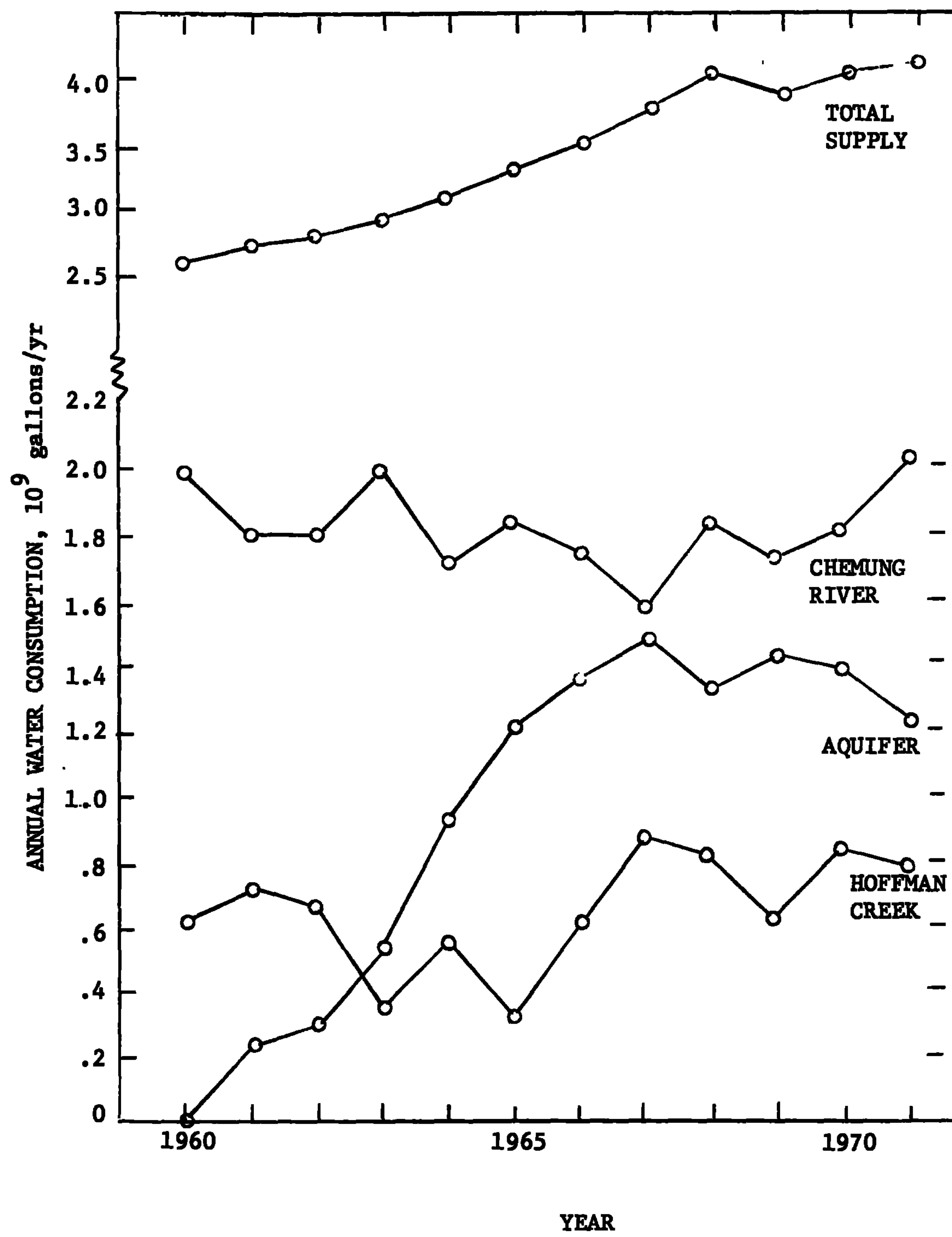

Fig. 8.3 Annual Water Consumption by Source for Elmira, New York, during 1960-1971 Period.

population. For this study a population increase of 100 percent for the 1980-2030 period is assumed.

This drastic increase in population and consequently in water requirements will necessitate a rapid expansion of the Elmira water supply system. Very soon Elmira will be forced to restructure its water supply system since most of the surface water supply installations, in particular the historic water treatment plant, will become obsolete in the near future. The need for system reorganization may come as early as 1980, according to information obtained from the current Technical Manager of the Elmira Water Board. Until 1980 Elmira will depend on the currently existing water supply scheme. In order to keep up with the expected growth until 1980 Elmira will add wells. Two or three more wells of 2 mgd capacity each will suffice to satisfy water requirements until 1980.

Because 1980 is the year in which the Elmira water system presumably needs total remodeling, the methodology developed here would be applicable for planning an essentially new water supply scheme for Elmira for the 1980-2030 period.

8.4 Input Data for Planning Procedure

The first step of the methodology consists of determining water requirements and of defining potential water sources.

8.4.1 Definition of the Demand Center

By 1980 the city of Elmira, plus Elmira Heights and Horseheads Village, will harbor about 90,000 inhabitants. It is assumed that the population will increase linearly to 180,000 by the year 2030. The following water use characteristics were estimated on the basis of the information provided by the Elmira Water Board:

Average daily per capita consumption = 160 gallons

Ratio of maximum daily consumption to average daily consumption in any one year = 1.8

A distribution of average monthly consumption as shown in Figure 8.4

Permissible hardness - 120 ppm.

Figure 8.5 describes the population, the total annual, and the maximum day water consumption projected for Elmira for the 1980-2030 period.

It is further assumed that by 1980 the only existing installations worthy of being incorporated into a new supply scheme are 8 wells and pumping equipment with a combined capacity of 15 mgd. If these wells are not used in a new design they are estimated to represent a sunk cost of $200,000. Another assumption is that a new treatment plant will be built in the same location as the present one. All water entering the distribution system is required to pass through the treatment plant. The plant's elevation is 1,000 feet above mean sea level.

8.4.2 Definition of the Source System

The water source system considered in this case study is presented in Figure 8.6. As surface water sources, the Chemung River is considered as a convenient and nearby source, without the need of a reservoir, whereas Newtown and Baldwin Creeks are considered to require a reservoir if any substantial firm water supply is to be drawn from these sources.

As a ground water source, an S-shaped portion of the alluvial aquifer along Newtown Creek and Chemung River was chosen, as outlined in Figure 8.6. To simplify the mathematical modeling task, this aquifer section was idealized as a rectangle, in which the well pattern would be arranged in a straight line, as shown in Figure 8.7, starting with the wells nearest

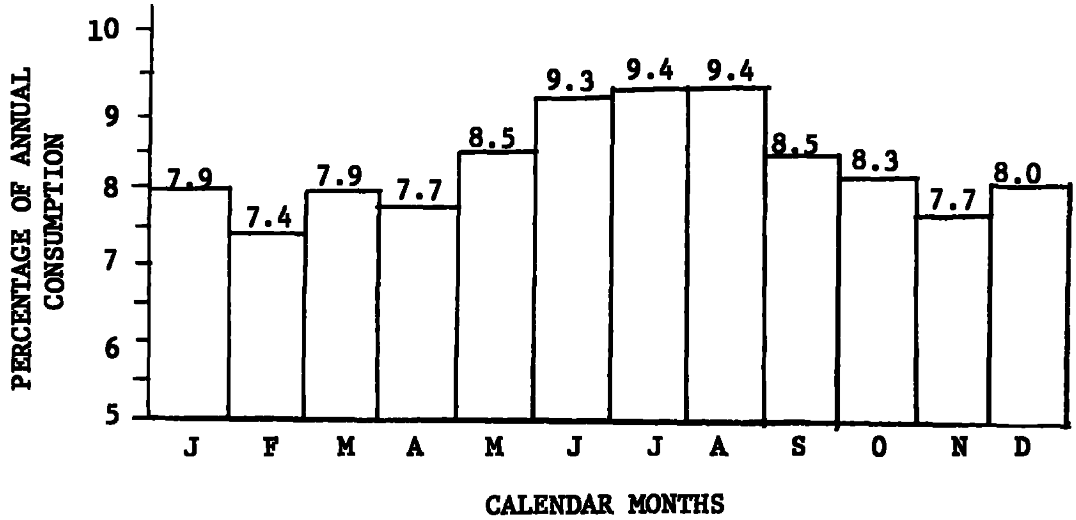

Fig. 8.4 - Monthly Consumption as Percentage of Annual Consumption

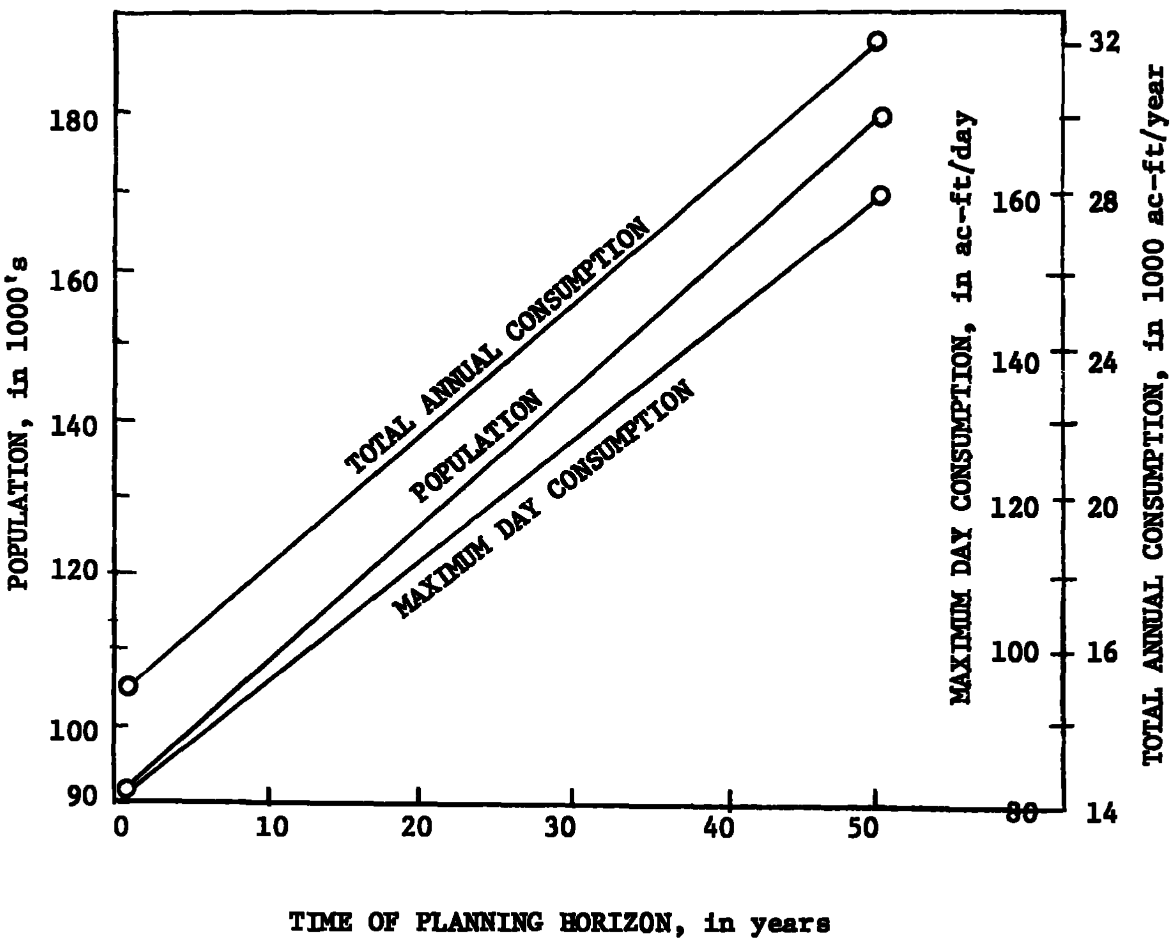

Fig. 8.5 Projected Population, Annual and Maximum Daily Consumption for Elmira, New York

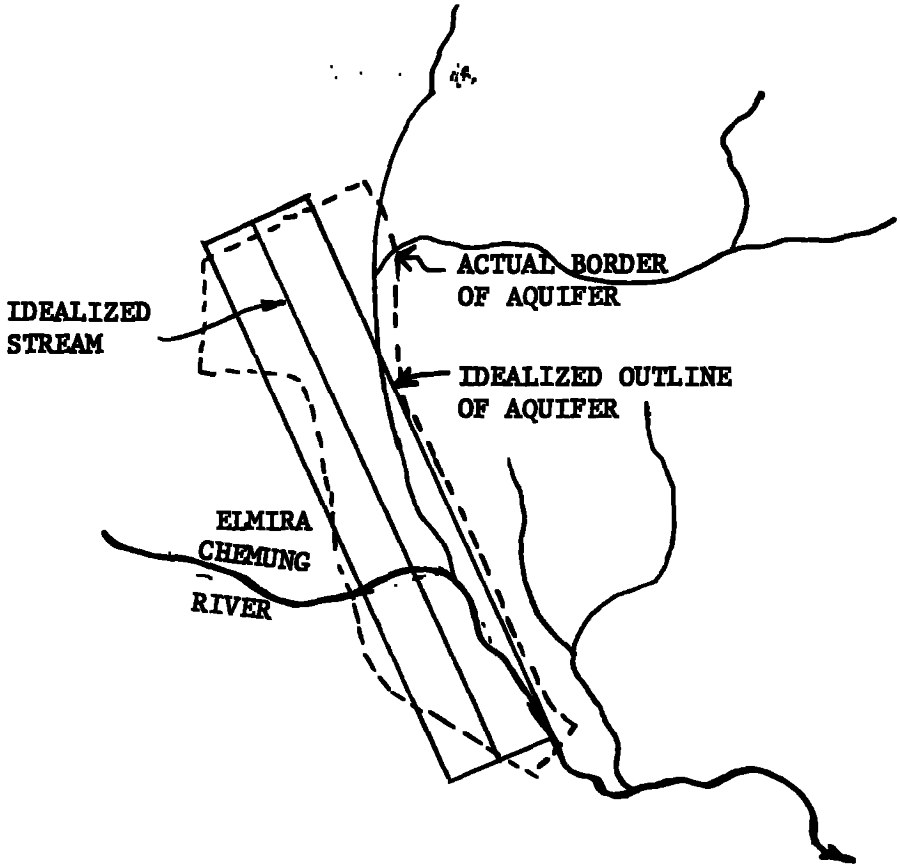

Fig. 8.6 Real and Idealized Outline of Aquifer Used in Elmira Case Study.

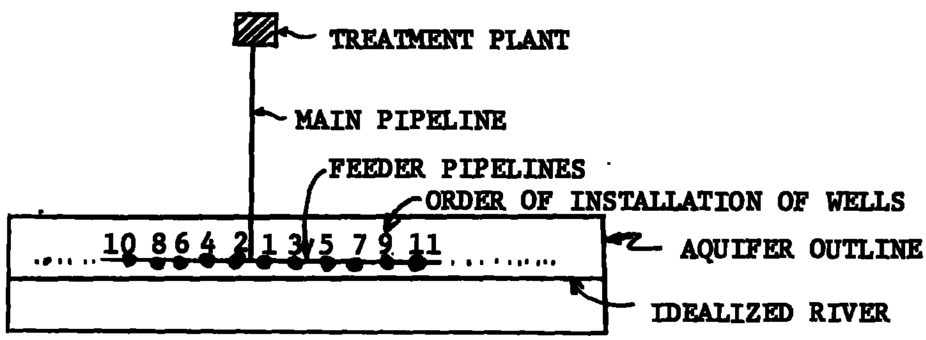

Fig. 8.7 Well Arrangement Used for Aquifer in Elmira Case Study.

the Elmira treatment plant and spreading out as the need for new wells arises. The assumed physical dimensions and properties are listed in Table 8.1.

The distance between wells was selected as 2,000 feet, which allows the final installation of 35 wells of a capacity of 1,200 gpm each. The wells are assumed to be gravel-packed with a borehole diameter of 20 inches. The length of the main pipeline is 2 miles and the static lift between treatment plant and average wellhead elevation is 150 feet. All costs are calculated on the basis of the cost equations summarized in Appendix A.

The dimensions and properties of the surface water sources selected for this study are also listed in Table 8.1.

From the data in Table 8.1 it is quite clear that the Chemung River holds economically a strong advantage over Newtown or Baldwin Creeks as a water supply source. Its proximity to the main demand center, as well as its large sustained flow which offers a run-of-the-river supply avoids the costs of long pipe lines and reservoirs. The small diversion dam presently in use at the Elmira water intake site can probably be retained even if the intake structure is rebuilt to increase its capacity. Thus, if the Chemung River water was of unquestionably high quality and there were no doubts regarding the legality of increased water diversions, there would indeed be little use for any economic comparison with the schemes involving the Newtown or Baldwin Creek reservoirs.

However, the Chemung River water could be expected to be of considerably poorer quality than the alternative creek water drawn from one of the reservoirs. According to Lohr and Love (1959) the Chemung River water carries an average turbidity of 116 ppm of SiO_2, and will thus require full

Table 8.1. Physical Data and Dimensions of Selected Water Supply Sources

Characteristics	Chemung River at Elmira	Newtown Creek abv. Breesport	Baldwin Creek abv. N. Chemung
Length of stream gage records, years	70	Flows interpolated from Newtown Cr. gage at Elmira, 35 years	
Drainage area, sq mi	2506	20	13
Average flow, cfs	2450	23	16
Ten year - one month low flow, cfs	50	1.0	0.8
Distance to treatment plant, mi	1	10	6
Water surface elevation at diversion structure	840	1200	1050
Pumping lift (+) or fall (-) to treatment plant	160	-200	-50
Ave. turbidity, ppm SiO_2	120	30	30
Ave. hardness, ppm $CaCO_3$	40	40	40

Alluvial Aquifer

Aquifer length, mi	13
Aquifer width, mi	1.5
Specific yield	0.02
Coeff. of conductivity, gpd/ft^2	750
Ave. aquifer thickness, ft	250
Ave. depth to static water table, ft	20
Ave. recharge rate through surface, mgd	10
Potential riverbed infilt. rate, gpd/ft^2	10
Potential riverbed infilt. rate, mgd	34
Ave. hardness, ppm $CaCO_3$	200
Turbidity	negligible

treatment including coagulation, flocculation, settling, filtration and chlorination. The water from either one of the two reservoirs should have a much lower turbidity and may require less treatment. In this analysis the surface water treatment costs were assumed to be equal for river and reservoir water, except for the operation and maintenance costs for coagulation, flocculation and filtration, because at present most Eastern States' public health regulations require full treatment of all surface water supplies, no matter how pure the source may be. The use of a purer supply source may, however, be a strong incentive to consider the Newtown or Baldwin Creek waters as an alternative. Furthermore, it is extremely difficult without a very detailed investigation to speculate whether or not the city of Elmira could acquire the necessary water rights to double and possibly triple the present diversions. Thus, the steps developed in the general methodology were applied to the Chemung River, Newtown and Baldwin Creeks and the alluvial aquifer water, with the full expectation of arriving at higher costs for the reservoir water.

8.5 Yield Determination

The next step in the methodology calls for determining the design yields that can be expected from the following source combinations:

Aquifer
Chemung River
Chemung River and Aquifer
Newtown Creek Reservoir and Aquifer
Baldwin Creek Reservoir and Aquifer

8.5.1 Chemung River and/or Ground Water Supply

From the yield point of view it was found that either the Chemung River or the aquifer outlined in Figure 8.2 can supply Elmira's projected water consumption throughout the planning period. In either of these sources, the yield will not be limited so much by quantitative constraints as possibly by legal constraints on water rights.

If both river and ground water are used, a joint rather than integrated water supply is provided, because neither source is under any shortage risk. The two sources would in that case provide water simply in some chosen proportion. The only type of river-ground water supply which could in any way be termed coordinated would be the "mixed treatment" supply, already recommended in section 7.2 as highly economical. Under this option untreated ground water would be mixed with treated surface water in a proportion to yield a product not to exceed the maximum desired hardness. Under the conditions and assumptions stated in section 8.4, a one-to-one mixture of the river and ground water, of 40 and 200 ppm hardness, respectively, would satisfy the specified limit of 120 ppm.

8.5.2 Newtown or Baldwin Creek Reservoir Supplies

Design yields for Baldwin and Newtown Creek Reservoirs are based on the historical "parent" streamflow record (lower Newtown Creek) used for Reservoir B in the hypothetical study. Therefore, the 50-year synthetic flow sequence number 20 for stream B, reduced in proportion to the respective drainage basins, was applied to the Baldwin and Newtown Creek Reservoirs. Furthermore, it was assumed that the City of Elmira is willing to accept an occasional water supply shortage expressed by an average annual shortage index of 0.05, and that the shortage-loss relationship shown in Figure 4.4 is valid also for Elmira.

In contrast to the joint river-aquifer supply which could be operated independently except in the case when mixed treatment is provided, it was found highly important to operate the reservoirs in close coordination with the aquifer to obtain as much yield as possible for this small surface water source.

The rule of preventive pumping was exercised in this case study application, but had to be modified slightly because the Newtown Creek water supply system was neither capable, nor designed to provide the full demand even during the wet months. The ground water system was instead assigned the dual role of providing a steady base flow plus the preventive pumping necessary to forestall surface water shortages. The equation for carry-over storage in the reservoir was thus written:

$$CS_i = \sum_{n=1}^{8} (D_n{*} - EXQ_n) - (8-i)\ PMPC* \tag{8.1}$$

practically identical to Equation 4.3, except for the reservoir demand D_n^*, which is the total water demand D_n in month n, minus the steady base supply SBS assigned to the ground water system, and for the preventive pump capacity PMPC*, equal to the total monthly pump capacity PMPC minus that same base supply.

Since the monthly pump and wellfield capacity PMPC is designed to provide the maximum day requirement, which equals the average daily requirement times the factor RMA, a logical value for the steady pump base supply is PMPC/RMA, leaving the quantity

$$PMPC* = (1 - \frac{1}{RMA}) \tag{8.2}$$

available for preventive pumping.

To determine the yield for a range of combinations of reservoir and wellfield capacities, the iterative computations proceeded along the following steps:

1. A wellfield and pump capacity was chosen. As a numerical example, let us take PMPC = 2400 ac-ft/month. The steady ground water base supply SBS was determined as

$$SBS = \frac{PMPC}{RMA} = \frac{2{,}400}{1.8} = 1{,}333 \text{ ac-ft/month} \tag{8.3}$$

and the remaining preventive pumping capacity was

$$PMPC^* = PMPC - SBS = 1067 \text{ ac-ft/month} \tag{8.4}$$

2. A sample yield was chosen, say 31,700 ac-ft/yr for the numerical example. From this annual yield and the monthly use percentages shown in Figure 8.4, the required monthly yields were computed. In the example, these monthly yields would range between a minimum in February of 0.074 x 31,700 = 2350 ac-ft and a maximum in July of 0.094 x 31,700 = 2980 ac-ft.

3. A trial reservoir size was chosen, and using the synthetic inflow series for the respective creek, a storage routing was performed. In this routing procedure, the steady ground water base supply SBS was subtracted for each month to give the surface water demand D_n^*. For the month of July, the third month of the drought-prone season,

$$D_3^* = 2980 - 1333 = 1647 \text{ ac-ft}$$

in the chosen example. Eq. 8.1 was applied each month between May and December to determine whether preventive pumping, up to a limit equal to PMPC* should be applied to guard against surface water shortages in the coming months.

This routing procedure was performed over 50 years and the shortages occurring during this period were recorded.

4. The shortages registered in the routing sequence under step 3 were converted to a shortage index. If the resulting shortage index differed from the chosen maximum value of 0.05 by more than 5 percent, the trial reservoir size was increased or decreased and steps 3 and 4 were repeated.

Steps 1 to 4 were then repeated for a large number of combinations of wellfield capacities and annual yields to determine relationships plotted in Figure 8.8

In Figure 8.9 the yield isoquants to satisfy the water requirements in years 0, 10, 20, 30, 40 and 50 from the reservoir-aquifer combinations were drawn as calculated according to the principles explained in Chapter 4. Some ground water supplementation will be needed from the first year on, because neither of the reservoirs can by itself produce a yield which could supply Elmira's first year water requirements. This early need for supplementation, however, will not result in any capital costs, because, as mentioned earlier, the existing wells with a combined capacity of 1350 ac-ft/month can be utilized.

Another set of curves produced in Figure 8.10 shows the wellfield capacity needed during any year of the planning period in order to supply the Elmira demand either exclusively or in integrated operation with Newtown or Baldwin Reservoir.

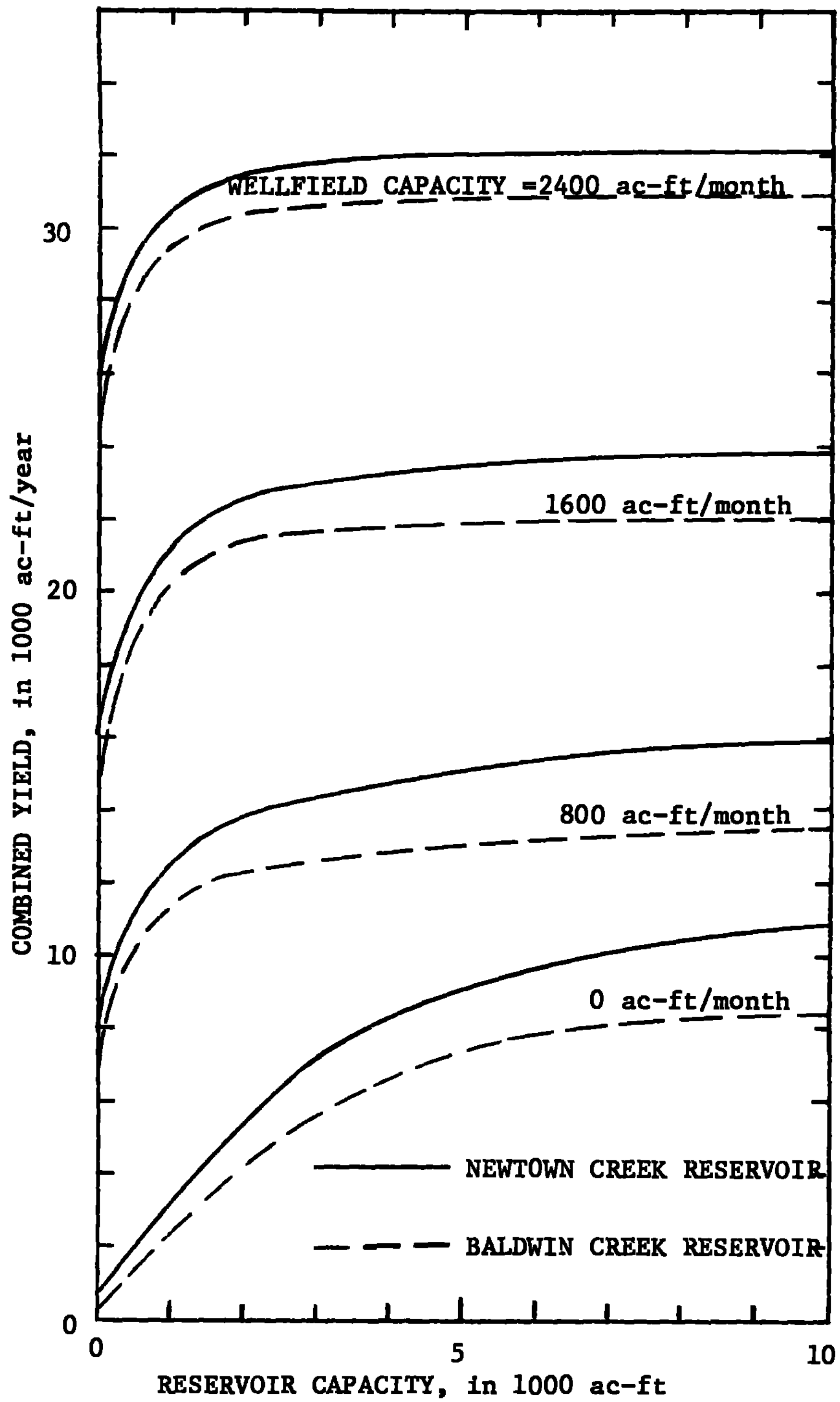

Fig. 8.8 Design Yield for Newtown and Baldwin Creek Reservoirs, Integrated with Aquifer

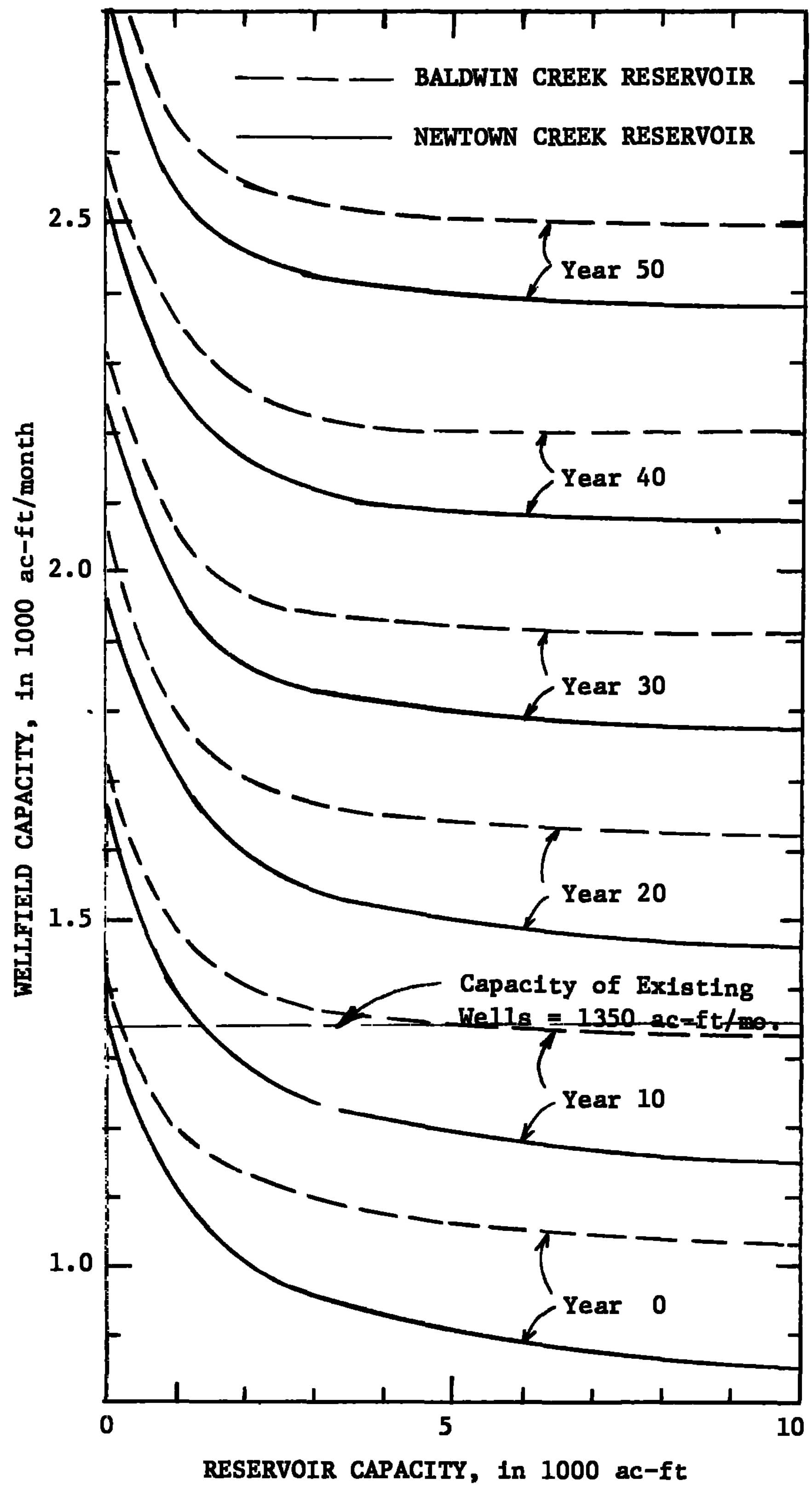

Fig. 8.9 Yield Isoquants Describing Possible Reservoir-Wellfield Combinations to Satisfy the Demand Throughout the Planning Horizon

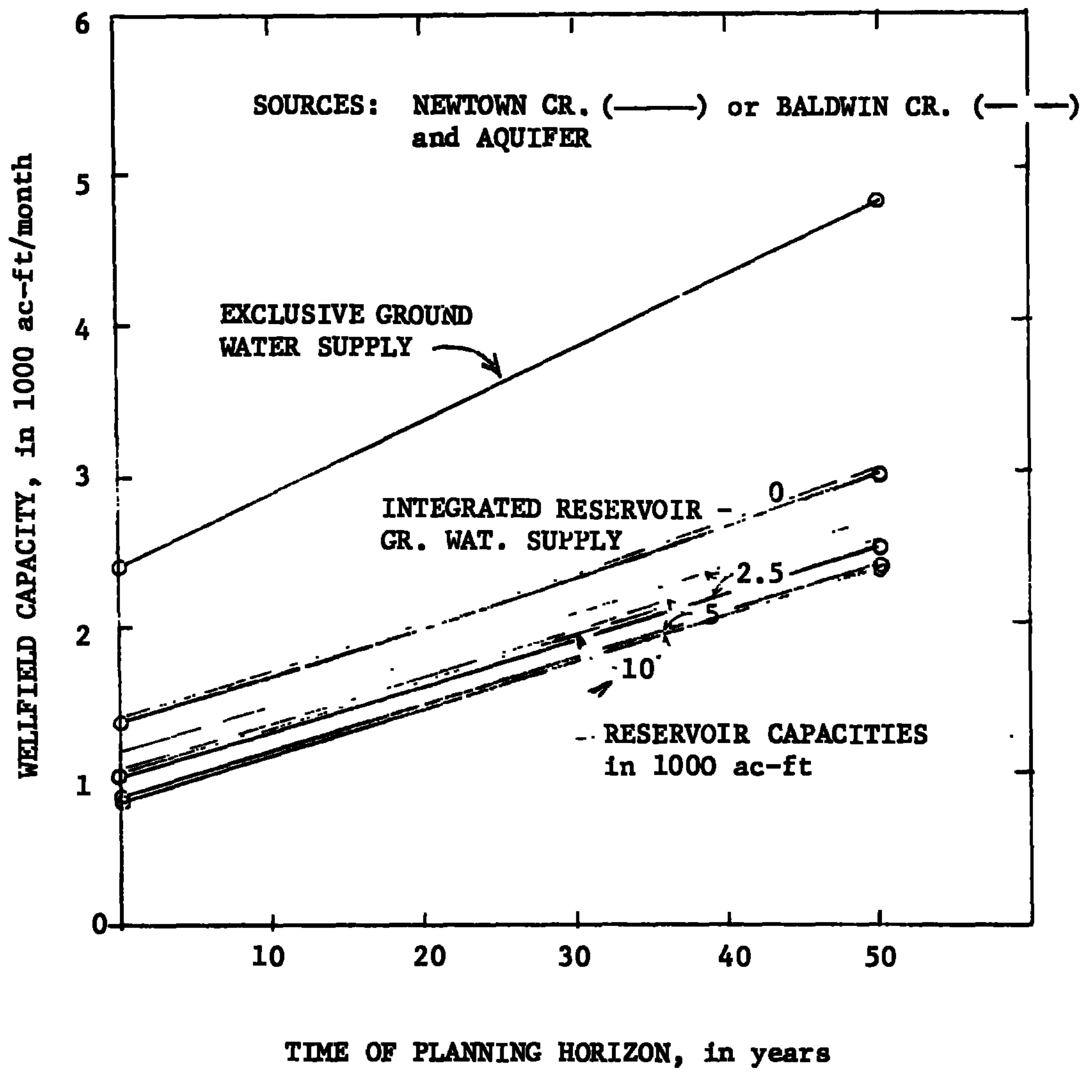

Fig. 8.10 Wellfield Capacity Required Throughout the Planning Horizon for Varying Reservoir Capacity.

Examination of Figures 8.8 to 8.10 shows that, due to the low flow of Newtown, little will be gained by designing the reservoirs for more than 5,000 ac-ft; beyond this capacity, the aquifer development needed to supplement the surface water is almost unaffected by an increase in reservoir size. In other words, any reservoir beyond 5,000 ac-ft on Newtown or Baldwin Creeks is unlikely to be filled very often.

8.6 Quantification of Decision Variables Under Various Alternatives

The quantification of flow volumes, required pipe capacities and number of wells, and degree of water treatment are the steps corresponding to stage 3 of the methodology which are needed for the cost determination and comparison.

The tasks in stage 3 include the quantification of the above mentioned variables not only for various alternatives but also as a function of time throughout the planning period. Computer programs can present these quantities in output stacks of reasonable volume, but in a report of this type only a small sample of the output can be produced in appropriate graphs. As in Chapter 5, the curves presented will describe the "ultimate" quantities or sizes, i.e. those corresponding to the last year of the planning period.

8.6.1 Chemung River and/or Ground Water Supply

As stated in section 8.5 the operation of a supply system consisting of Chemung River and/or ground water was essentially one of two separate systems, except when the mixed treatment option was considered, in which case the proportion of river and ground water would be specified by the hardness of the sources used.

Daily delivery and treatment capacities on which the capital costs of all pipelines, wells, pumps, and treatment plants will be based, were at all times scaled to be at least equal to the maximum day consumption rate, and were divided among river and ground water source according to the proportion selected for annual delivery volumes.

Wells, pumps and certain parts of treatment can be provided and added as the demand arises, but other components, like dams, intake towers pipelines and certain structural portions of the treatment plant, were assumed to be built originally to full ultimate size.

8.6.2 Newtown or Baldwin Creek Reservoir Supply

Under the integrated reservoir-ground water supply alternatives the capacities of diversion, conveyance and treatment facilities can be determined directly from the curves in Figure 8.9, but the expected supply volumes from any two sources depend on the synthetic stream inflow to the reservoirs and were obtained from the outputs of the routing programs.

Required wellfield capacities for various sizes of the two reservoirs were presented in Figures 8.9 and 8.10. Maximum day delivery capacities for the wellfield can be obtained simply by division of the monthly capacities by 30, since the maximum day use factor is incorporated into the wellfield sizing.

The maximum day capacities for the surface water supply elements, plotted in Figure 8.11, were computed as the difference between total maximum day consumption and wellfield capacities, under the assumption that the reservoir, integrated with the aquifer operation by the preventive pumping rule, can supply the occasional short-time high maximum day requirement even during drought periods.

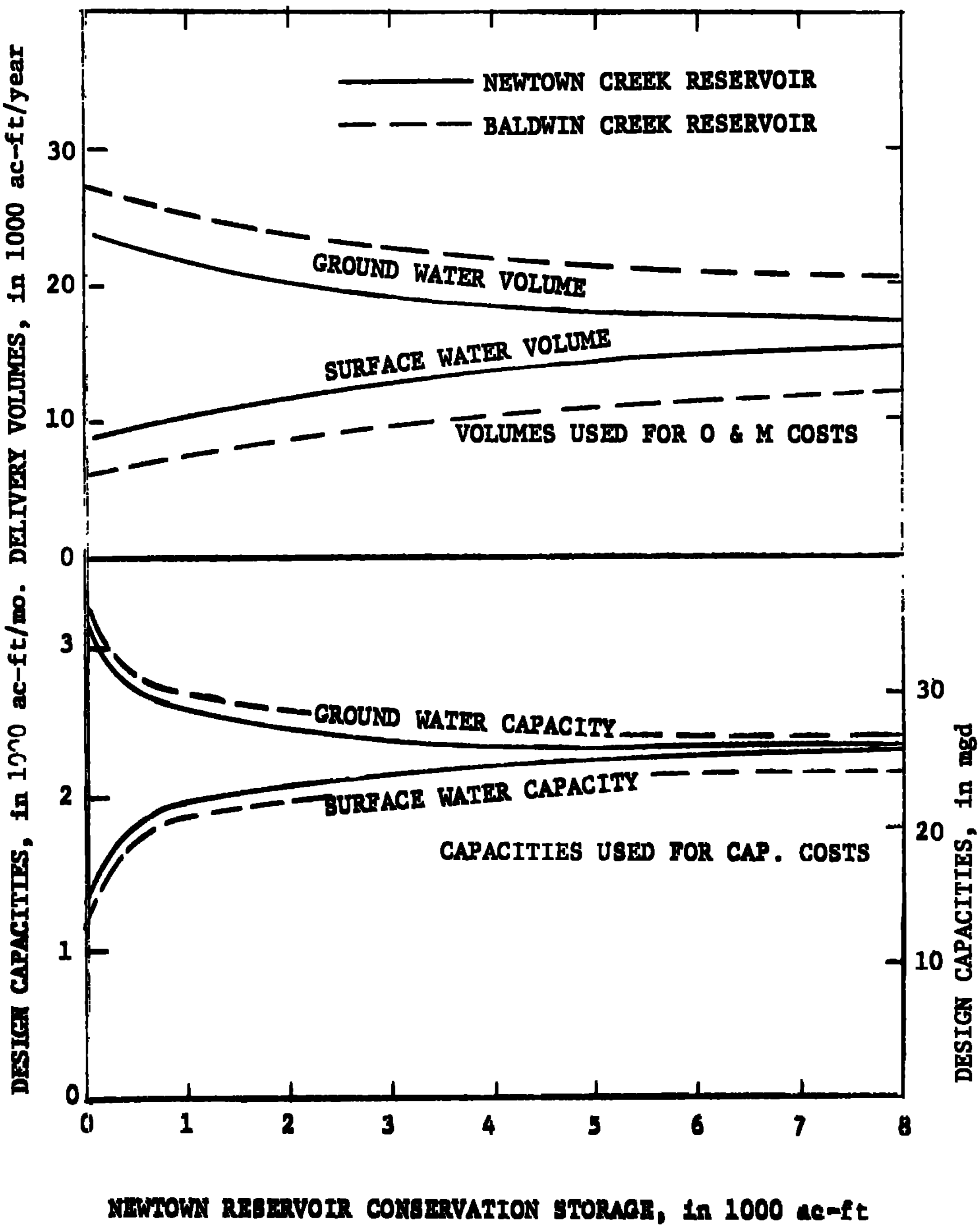

Fig. 8.11 Design Capacities and Delivery Volumes in Last Year of Planning Horizon.

The annual surface water delivery volumes, taken from the computerized routing output, are also shown in Figure 8.11.

8.7 Cost Computations and Comparisons

Once the decision variables and supply components were determined for a range of alternative supply combinations, the major investment and operational costs were computed, converted to present worth where necessary, and plotted for comparison.

8.7.1 River and Aquifer Supplies

Figure 8.12 shows the costs for exclusive surface supply from the Chemung River, dual supply from the river and the aquifer, and exclusive ground water supply. Surface water is assumed treated conventionally, ground water for 80 ppm hardness removal. No true integration of surface and ground water takes place, unless the phenomenon of induced aquifer recharge by river water through water table drawdown can be studied in detail. For this reason, dual water supply schemes present no economic advantages in this case and merely add to equipment and operation costs as evidenced by the upward curving shape of the total cost curves. It will be further noted that the bulk of the costs are due to treatment facilities and operation. Without more detailed knowledge of the treatment required, however, no truly reliable treatment costs can be estimated, particularly for the ground water treatment which is highly dependent on the degree of hardness encountered.

Under the assumptions made in this study, the total costs of exclusive Chemung River supply added up to $17.7 million as compared to $22.3 million for exclusive ground water supply. This cost difference could change drastically if the ground water pumped was found to be of lower or higher hardness.

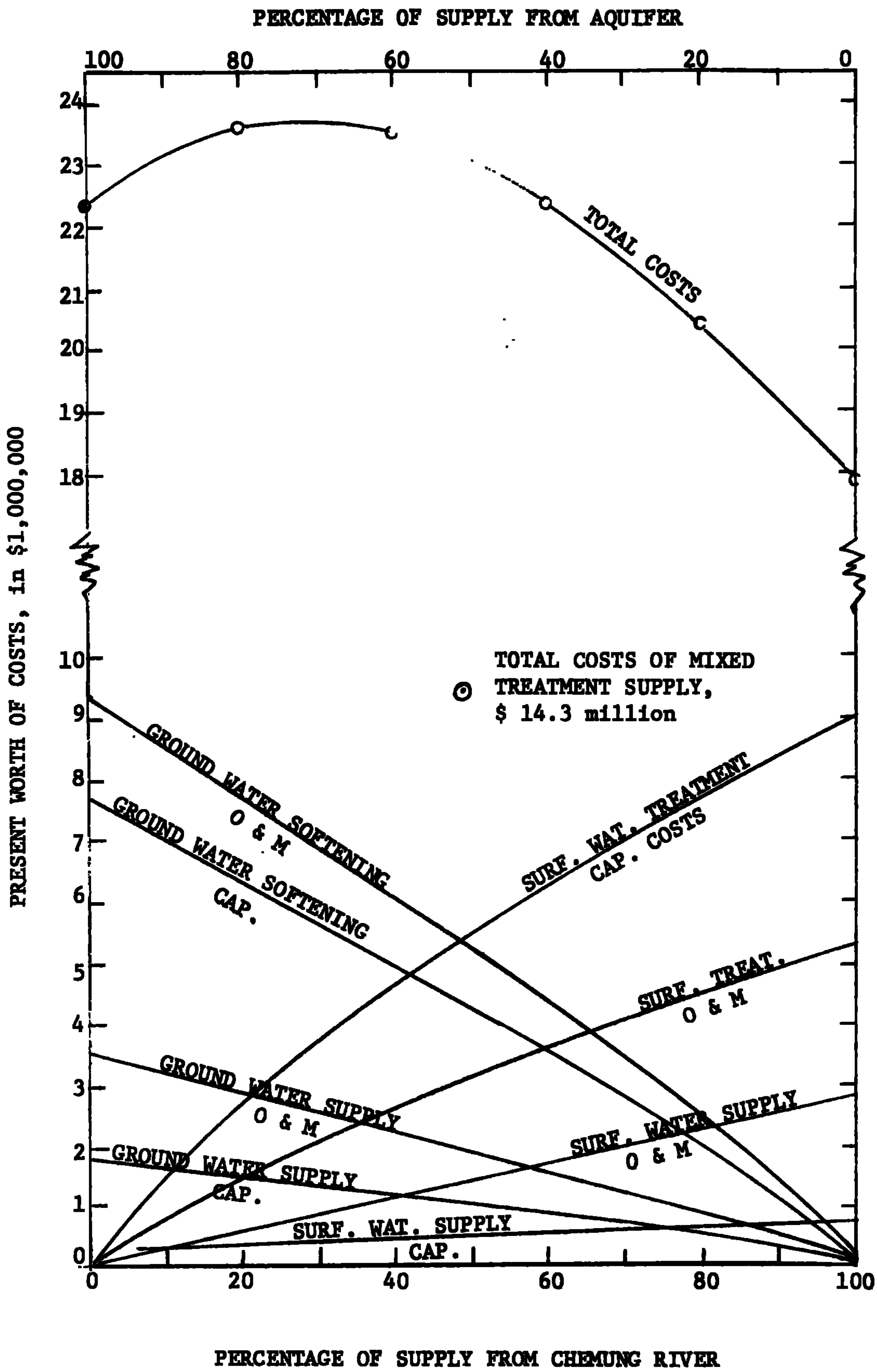

Fig. 8.12 Combined Costs for Water Supply from Chemung River and Aquifer.

Under the "mixed treatment option" mentioned in section 8.6.1, a one-to-one mixing ratio between river and ground water would satisfy the maximum hardness limitation, and eliminate $9.3 million of softening costs, resulting in a total cost of $14.3 million, considerably lower than any other alternative. ($0.5 million was allowed for any incremental costs needed for joining and mixing the two water supplies)

8.7.2 Reservoir-Aquifer Supplies

Costs for the major components of the integrated supply of ground water and Newtown Reservoir surface water were plotted on Figure 8.13. As could be observed from the earlier yield curves, the required wellfield capacities and ground water volumes for Baldwin Creek were not very much higher than those for Newtown Creek, and the additional ground water production and treatment costs were roughly offset by savings in surface water conveyance and treatment costs. The difference in costs between the Newtown and Baldwin Creek reservoir schemes was so small that it did not seem warranted to produce two sets of cost curves, particularly in view of the hydrologic uncertainty in the Baldwin Creek estimates in which the synthesized flows were generated from records of Newtown Creek.

As expected, the reservoir-aquifer combinations compared unfavorably in cost with the Chemung River water or river-aquifer mixed treatment supplies.

Among the reservoir-aquifer combinations, the lowest cost scheme corresponds to a Baldwin Creek reservoir of less than 1000 ac-ft capacity. The alternative with zero reservoir capacity, supplying surface water entirely from the natural Baldwin Creek flow, would have eliminated all reservoir costs but would provide such an unreliable supply that the required

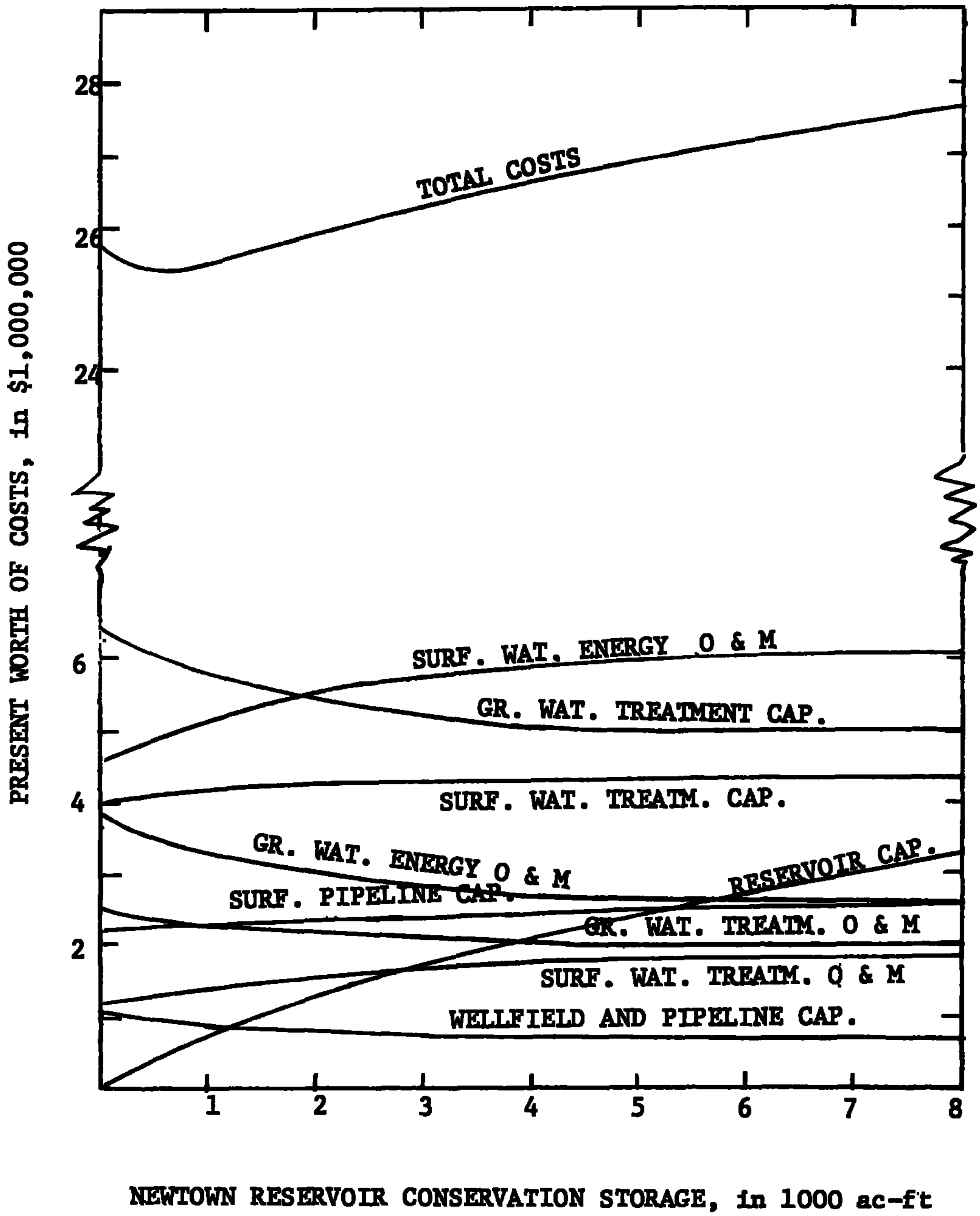

Fig. 8.13 Combined Costs for Water Supply from Newtown Reservoir and Aquifer

wellfield capacities would have been almost equal to those of the exclusive ground water supply. A small reservoir capacity, on the other hand, if operated under the preventive pumping rule, can assure a substantial firm supply on the few days of maximum water consumption, thus decreasing the required capacities of the wellfield and softening plant, and increasing the ground water systems load factor in much the same way as the provision of an intermediate storage tank, advocated in Chapter 6. The possibility of providing intermediate storage was not investigated in this case study, but should be given serious consideration in any major water supply system expansion plan.

The minimum-cost location in the region of very small reservoir capacities conflicts with the generally accepted concept of inefficiency of small reservoirs. The reason for this intuitively illogical low-cost point lies mainly in the use of coarse general equations and the high costs of surface water treatment. Particularly the equation used for reservoir costs (Appendix A, eq. A-22) seems highly simplified, ignoring the setup and other quasi-fixed costs which would tend to make reservoir costs rise abruptly at low volumes and flatten out as the volume increases. The high surface water treatment costs regardless of the degree of water purity, although consistent with the present regulations which require full treatment of all surface water, further contributed to discouraging the increase of the reservoir size.

8.8 Conclusions of the Elmira Case Study

On the basis of the data collected at Elmira and the assumptions stated in section 8.4, the alternative of mixing conventionally treated Chemung

River water with untreated ground water in a proportion to yield a product of acceptable hardness, and followed by chlorination of the mixed water, would result in the least-cost water supply.

The two suitable reservoir sites located in the vicinity of Elmira were not found economically competitive with the river-aquifer supply source because of the relatively small average yearly yields of the streams feeding the reservoir, the assumed requirement of costly surface water treatment even for high quality water, and the relatively long distances from the reservoirs to the demand center. However, the integrated reservoir-aquifer operation results in highly efficient water use from a resource conservation point of view, and should be seriously considered unless no problem whatsoever is anticipated in securing extensive Chemung River or ground water rights for the City of Elmira.

Due to the difficulty of making comparative economic analyses without the use of detailed design data, the cost curves should only be regarded as part of the application of the developed methodology to the chosen case study. The investigators believe that the choice of the case study site and the quality of the data collected and used were the best which could have been found among several alternative sites considered, and that the case study lent itself well to the demonstration of the proposed planning steps.

8.9 Hazleton Case Study Consideration

Initially it was intended to present another case study with the city of Hazleton, Luzerne County, Pennsylvania, as an example. Unfortunately, Hazleton turned out to be an unlucky case study site location for several reasons.

In the first place, sufficient aquifer data could not be gathered for the Hazleton area. Further, the little information available indicated that the nature of the aquifer is so complicated that a proper mathematical treatment would have exceeded the modeling capacity of the aquifer program developed for this study. To obtain a valid representation of the Hazleton area aquifers a custom-tailored aquifer model would be necessary. Such a venture was beyond the scope of this study.

The second reason for not performing a Hazleton case study is the complicated water supply system presently in existence. The current water supply scheme, operated by the Hazleton City Authority, consists of a multitude of smaller water sources, both ground water and surface water. It consists of ten different divisions, each of which supplies a small portion of the total Hazleton water consumption requirements. A brief description of the division is presented in Table 8.1. In 1970 all Hazleton water works installations were valued at about $7,000,000. Since this amount represents a considerable investment, a case study analysis would be useful and valid only if full consideration could be given to the incorporation of the existing facilities into a new design. The methodology developed in this study does not permit the inclusion of existing facilities as complex as in Hazleton's case. An application of the methodology to the Hazleton water supply situation would require unjustifiable system simplifications and distortions, rendering the results unrepresentative. A valid evaluation of expansion schemes in the Hazleton case would require detailed and careful modeling specific for the Hazleton situation. Such a detailed analysis would necessitate an investment of time and money which certainly exceeds the resources available for this study.

Table 8.2. Summary of Hazleton Water Supply Sources

Division	Name	Description of Supply	Population Served
1	Hudsondale	Quakake Creek, Alderson's Run Beisel's Run, total drainage area = 24 sq miles	7,100
2	Mt. Pleasant	Eleven artesian wells	6,100
3	Barnes Run	Wolff's Run, Barnes Run, Stoney Cabin Creek	14,400
4	Dreck Creek	Dreck Creek, drainage area = 2.5 sq miles	12,700
5	Harleigh	Two artesian wells	1,900
6	Buck Mountain	Shaffer's Run, and a number of springs flowing into Buck Mountain Reservoir	1,300
7	Derringer	Derringer and Tomhicken wells	400
8	Ebervale	Two artesian wells	1,490
9	Delano	Springs and artesian wells	200
10	Can-Do	Wells	Valmont Indus. Park

Hazleton's water requirements are expected to grow significantly in the near future. The Luzerne County Planning Commission contracted with an engineering consulting firm (Gilbert Associates) to study possible system expansions. The firm's proposal suggested that either the Lehigh River be tapped near White Haven or the Susquehanna be tapped near Nescopek. In contrast, the U. S. Army Corps of Engineers maintains that further wellfield developments in the immediate Hazleton area would be capable of satisfying Hazleton's estimated maximum daily water requirements through the year 2030 at much less cost. This controversy surrounding the expansion of Hazleton's water supply system is another reason why the writers think it would be imprudent to present a Hazleton case study, which would have to be based on poor data and unrealistic system modeling, and could possibly be misused to back up one or the other side of the controversy.

APPENDIX A

COST DATA FOR PLANNING EVALUATIONS

The costs of constructing and operating water resource systems is difficult to estimate, particularly so for planning which requires that estimates be projected into the future. Cost studies available in the literature and from governmental agencies vary widely in scope and completeness, according to the effort invested in the studies and the background and interests of the organizations conducting the studies. Assumptions and exclusions in the published literature are often not well-documented, thus limiting the usefulness of much of this available information.

A.1 Cost Indexes

Indexes applicable to construction costs have been used in the United States for several decades. They serve to correlate the cost of projects constructed at different times and different geographical locations. While not a substitute for actual cost estimates, such indexes do make possible the use of data from different times and locations. However, in a time of continuing inflation or deflation, it is impossible to fix costs at a given point in time and have them remain constant for any reasonable period. If longitudinalcost changes do not involve differential movements, analytical results remain valid in a relative sense and need only be updated by an appropriate index value. However, technological changes may cause one process to become more or less expensive relative to competing processes; operation and maintenance (O&M) costs may change more or less rapidly than capital costs; and so on. Analytical results sensitive to such changes will be somewhat in error.

Heiple (1967), who compared the different cost indexes applicable to municipal water systems, concluded that the Engineering News Record Building Cost Index (ENR-BCI) is the most representative for all cost centers except when applied to surface water reservoirs. This includes the O&M costs. For surface water reservoirs the Engineering News Record Construction Cost Index (ENR-CCI) was thought to be the most representative index. Thus, following Heiple's recommendations, all costs except for surface water reservoirs are adjusted to a common base of 1039 using the ENR-BCI. Moreover, for surface water reservoirs, both investment and O&M costs are adjusted to a common base of 1727 using the ENR-CCI. The common base time is mid-year, 1972. The ratio of the current index (ENR-BCI = 1039, ENR-CCI = 1727) to the index existing when the costs were developed is the factor used to raise the costs to the current cost levels. This index ratio (IR) is given for each cost equation presented here.

A.2 Discount Rate and Planning Horizon

Both discount rate and project planning horizon are treated as input parameters in the computerized analysis discussed in Appendix B and can be adjusted to conform to federal practice as needed. Treatment of salvage value is also explained in Appendix B.

The subject of appropriate discount rates has long been controversial, with recommended rates ranging from 3 percent or less to 10 percent for various theoretical and other reasons. The discount rate used is 5.375 percent as recommended by the Water Resource Council (1971) at the start of this study, although a long term rate of 7 percent is a future possibility.

The selected project life is also subject to debate. A planning horizon may be based on the physical service time of the structures, the financial repayment period, an economic life based on benefit accrual, or other considerations. Depending on the interpretation, a greater or lesser project life may be appropriate. In general, if capital costs are discounted over a longer planning horizon, project selection is prejudiced in the direction of capital-intensive alternatives. In this study, the alternatives have highly variable ratios of capital to annual expenses so the choice of project life could be critical in this regard. At relatively short discount periods, like 25 years, the capital recovery factor changes measurably with project life. For example, at a 5.375 percent discount rate, it becomes 0.08282 for 20 years and 0.06787 for 30 years, resulting in an error in excess of 15 percent for annual cost calculations. This sensitivity is much less pronounced as the length of the planning horizon increases.

A.3 Cost Equations

In general the construction cost equations do not directly include contractor's profit, land acquisition, engineering fees, or contingency allowances. Additional assumptions and limitations associated with individual cost items are documented throughout this appendix.

The equations presented here are for generating approximate costs for comparing the order of magnitude and are not intended for use in making detailed estimates for a specific site. A summary of the water supply system components and the applicable cost equations is given in Table A.1.

Table A.1. Summary of Water Supply Components and Cost Equations

Components	Applicable Cost Equations: Construction	Applicable Cost Equations: Operation and Maintenance
Ground Water System		
Wells	A.1	A.4
Well Pumps	A.2	A.10
Booster Pumps	A.3	A.10
Feeder Pipelines	A.8	A.9
Collector Pipelines	A.8	A.9
Transmission Line	A.8	A.9
Water Treatment Plant[1]	A.11,A.13,A.14,A.16,A.18	A.12,A.15,A.17,A.19
Treated Water Storage	A.20	A.21
Surface Water System		
Reservoir	A.22	A.23
Intake	A.8	A.9
Pump Station	A.5,A.6	A.7,A.10
Transmission Line	A.8	A.9,A.10
Water Treatment Plant[1]	A.11,A.13,A.14,A.16,A.18	A.12,A.15,A.17,A.19
Treated Water Storage	A.20	A.21

[1] A $15,000 charge for land purchase was added to the construction costs for water treatment plants.

Table A.2. Coefficients and Exponents for Well Cost Equation

Geological Formation	Range of Bore Hole Diameter in inches	COE	EXP
Tubular Wells Finished in Sand and Gravel	6 - 10	800.0	0.299
Tubular Wells Finished in Sand and Gravel	12 - 15	850.0	0.373
Gravel-Packed Wells Finished in Sand and Gravel	16 - 20	680.0	0.408
Gravel-Packed Wells Finished in Sand and Gravel	24 - 34	680.0	0.482
Gravel-Packed Wells Finished in Sand and Gravel	36 - 42	890.0	0.583
Shallow Sandstone, Limestone, or Dolomite Bedrock Wells	6	578.0	1.413
Shallow Sandstone, Limestone, or Dolomite Bedrock Wells	8 - 12	839.0	1.450
Shallow Sandstone, Limestone, or Dolomite Bedrock Wells	15 - 24	1781.0	1.471
Deep Sandstone Wells	8 - 12	29.0	1.870
Deep Sandstone Wells	15 - 19	1314.0	1.429

A.3.1 Construction Costs for Wells

The construction cost for a well is a function of the depth, the bore diameter, the geological formation, and the expected flow from the well. The basic well construction cost equation presented here was taken from Dawes (1970). In addition to the basic construction cost, a test hole costing $2,000 and a well house costing $2,500 were added. The construction cost is

$$C_w = (IR)(COE)(D_w)^{EXP} + 4500 \qquad (A.1)$$

where C_w is the construction cost of the well in dollars, IR is the index ratio equal to 1.60, and D_w is the depth of the well in feet. Both COE and EXP are functions of the well diameter and the geologic formation. Their values are presented in Table A.2. The life expectancy of a well was assumed to be 40 years.

The well diameter is a function of the pumping rate. For rates between 0 and 200 gpm a 6 inch diameter casing was assumed; between 201 and 450 gpm an 8 inch diameter; between 451 and 900 gpm a 10 inch diameter; and for any pumping rate greater than 900 gpm a 12 inch diameter was used in calculations.

A.3.2 Installation Costs of Well Pumps

Installation costs are a function of the maximum pumping head and the maximum flow rate. Dawes (1970) gives costs for the installation of vertical turbine pumps as

$$C_{pu} = (IR)(7.31)(Q)^{0.453}\ (H)^{0.642} \qquad (A.2)$$

where C_{pu} is the installed cost of the pump in dollars, IR has a value of 1.60, Q is the maximum discharge in gpm, and H is the maximum expected pumping heat in feet. The well pumps were assumed to have a life expectancy of 20 years.

A.3.3 Installation Costs of Booster Pumps

Close-couple booster pumps were assumed to be installed in the main pipeline. They are sized using the maximum possible pumping head in excess of 700 feet. The equation used is

$$C_{bp} = (IR)\ [(67.8)(Q_t) - (4.04)(Q_t)^2 + (0.123)\ (Q_t)^3]\ (H_e) \qquad (A.3)$$

where C_{bp} is the cost of the installed booster pumps in dollars, IR has a value of 1.70, Q_t is maximum discharge in gpm, and H_e is pumping head in excess of 700 ft. (Dolson, 1964). The booster pumps are for use in the ground water system. The life expectancy of the booster pumps was assumed to be 20 years.

A.3.4 Operation and Maintenance Costs of Wellfield

The O&M costs for a wellfield as given here include such items as labor, vehicles, and so on, but exclude power costs which are indicated as a separate item. The following equation does not come from a detailed study of wellfield operations but is an estimate by people in the field (U.S. Department of the Interior, 1966). The equation is

$$C_{wf} = (IR)(7.6)(VOL) \qquad (A.4)$$

where C_{wf} is the wellfield O&M costs in dollars per year, IR has a value of 1.70, and VOL is the volume pumped per year in million gallons. The O&M cost of both well pump and booster pump are assumed to be included in this estimate.

A.3.5 Construction Costs of Pumping Station

The costs of the pumps and the building are given by

$$C_{ps} = (IR)(HP)(0.290)(Q_t)^{-0.50} \quad 0.2 \text{ mgd} \leq Q_t \leq 2.0 \text{ mgd} \tag{A.5}$$

$$C_{ps} = (IR)(HP)(4.19)(Q_t)^{-0.12} \quad 2.0 \text{ mgd} \leq Q_t \leq 200 \text{ mgd} \tag{A.6}$$

where C_{ps} is the construction cost of the pumping station in dollars, IR has a value of 1.70, HP is the installed horsepower, and Q_t is in gpm (Koenig, 1966). The life expectancy of the pump station was assumed to be 25 years.

A.3.6 Operation and Maintenance Costs for Pumping Station

The operation and maintenance costs are for pumping stations between 150 and 15,000 horsepower. The original cost data were obtained from 18 U.S. Bureau of Reclamation pumping stations and 20 other similar stations. The cost is given by

$$C_{ops} = (IR)(0.311)(Q_t)^{0.54}(H)^{0.41}(T)^{0.43}(A)^{0.55} \tag{A.7}$$

where C_{ops} is the annual O&M cost for the pump station in dollars, IR has a value of 1.79, T is the equivalent annual hours of operation for the plant operating at design capacity, and A is the age of the plant in years (Eyer, 1965). The variables H and Q_t are as defined previously for equations A.2 and A.3.

A.3.7 Construction Cost of Pipeline

The construction costs of pipeline per mile is assumed to be primarily a function of pipe diameter. Although it is a simple relationship, the cost equation is highly reliable. It is

$$C_{pl} = (IR)(2{,}160)(D_p)^{1.29} \tag{A.8}$$

where C_{pl} is the construction cost of the pipeline in dollars per mile, IR has a value of 1.70, and D_p is the nominal diameter of the pipeline in inches (Dawes, 1970). Pipelines were assumed to have a life expectancy of 50 years. The right-of-way cost is minor and is taken as $569 per mile (Cederstrom et al., 1971). This quantity is added to the value computed with equation A.8.

A.3.8 Operation and Maintenance Costs of Pipeline

The pipeline O&M costs are a direct function of the total pipeline construction cost excluding right-of-way. The cost equation is

$$C_{opl} = (0.0025)(C_{pl}) \tag{A.9}$$

where C_{opl} is the O&M cost of the pipeline in dollars per year (U.S. Department of the Interior, 1966). The index ratio is not included since the cost level is adjusted in the construction cost equation.

A.3.9 Costs of Electrical Power

Electricity costs were obtained from West Penn Power Company (1971) according to "General Power Service-Schedule 31." Electricity costs are a function of both the installed horsepower and the amount of power used. Although the cost equation is a step function, all power costs fell into one step for this study. The modified cost equation used is

$$C_{pw} = (IR)(1.0525)\ [2.5 + (1.61)(KWD - 100) + 190 + (0.0070)(KWH - 20{,}000)] = (IR)(0.00023)(KWD) \qquad (A.10)$$

where C_{pw} is the cost of electricity per year, IR has a value of 1.00, KWD is the kilowatt demand, and KWH is the kilowatt-hours per month energy usage.

A.3.10 Construction and Equipment Costs for Chlorination System

This cost equation is for building and equipment used in plain or simple chlorination where the water receives no other treatment. Published costs for such a system were not found, and a hypothetical cost equation was developed. The cost for such a system would be a small percentage of the total cost and was assumed to be represented by

$$C_c = (IR)(8{,}700)(Q_{dc})^{0.6} \qquad (A.11)$$

where C_c is the construction cost in dollars, Q_{dc} is the design capacity in mgd and IR has a value of 1.00. The life expectancy of the system was assumed to be 50 years.

A.3.11 Operation and Maintenance Costs for Chlorination System

The O&M costs include chemicals, labor, and building and equipment. The chlorine dosage is assumed to be 28.8 lbs per million gallons (MG), the labor cost is $0.50 per MG, and the O&M costs for the building and equipment are 2 percent per year of the original construction cost. The cost equation is

$$C_{oc} = (IR)[(28.8)(0.13)(VOL) + (0.50)(VOL)] = (0.02)(C_c) \qquad (A.12)$$

where C_{oc} is the annual O&M cost in dollars for the chlorination system, IR has a value of 1.24, VOL is the volume of water treated in million gallons, and C_c is from equation A.11. The O&M costs were taken from Koenig (1967).

A.3.12 Construction Costs for Lime-Soda Softening

The construction costs for lime-soda softening although obtained from Metcalf & Eddy (1967), were originally developed by Howson (1962). These data are approximated here by two equations. These are

$$C_{sf} = (IR)(310,000)(Q_{dc})^{0.550} \qquad 1 \text{ mgd} \leq Q_{dc} \leq 10 \text{ mgd} \qquad (A.13)$$

$$C_{sf} = (IR)(151,000)(Q_{dc})^{0.862} \qquad Q_{dc} > 10 \text{ mgd} \qquad (A.14)$$

where C_{sf} is the construction cost of the equipment used for lime-soda softening in dollars, Q_{dc} is the design capacity in mgd, and IR has a value of 1.62. The construction cost does not include the operations building for the water treatment plant. The life expectancy of the equipment was assumed to be 50 years.

A.3.13 Operation and Maintenance Costs for Lime-Soda Softening

The O&M costs for lime-soda softening are a function of the hardness removed and average yearly flow rate. The original curves are nonlinear, and are approximated here as straight lines. The general form of the equation is

$$C_{osf} = (IR)(365)(COE)(Q_{ay})^{EXP} \tag{A.15}$$

where C_{osf} is the yearly O&M cost in dollars for lime-soda softening IR has a value of 1.62, and Q_{ay} is average yearly flow rate in mgd (Metcalf & Eddy, 1967). The values of COE and EXP are presented in Table A.3 for the various ranges of hardness removed and average yearly flow rates.

A.3.14 Construction Costs for Coagulation, Flocculation and Rapid Sand Filtration

Construction costs for conventional coagulation-filtration plants depend primarily on the design flow rate of the facility. Water quality may affect the sizing of some chemical storage, feed equipment and piping, and possibly the degree of monitoring instrumentation required. The cost of these items, however, would increase the overall cost of the treatment works by a relatively small amount (Metcalf & Eddy, 1967). The construction cost, which does not include the operation building for the water treatment plant is

Table A.3. Coefficients and Exponents for Lime-Soda Softening O&M Cost Equation

Hardness Reduction (mg/l)	COE	EXP	Range of Applicable Average Yearly Flow Rates (mgd)
300	174.0	0.632	$1 \leq Q_{ay} < 10$
300	87.0	0.919	$10 \leq Q_{ay} \leq 100$
200	162.0	0.585	$1 \leq Q_{ay} < 10$
200	71.4	0.919	$10 \leq Q_{ay} \leq 100$
100	147.0	0.530	$1 \leq Q_{ay} < 10$
100	53.0	0.919	$10 \leq Q_{ay} \leq 100$

$$C_{cf} = (IR)(330{,}000)(Q_{dc})^{0.678} \qquad (A.16)$$

where C_{cf} is the construction cost of the coagulation-filtration equipment in dollars, Q_{dc} is design capacity in mgd, and IR has a value of 1.62. The life expectancy of the equipment was assumed to be 50 years.

A.3.15 Operation and Maintenance Costs for Coagulation, Flocculation, and Rapid Sand Filtration

The annual O&M costs for coagulation-filtration depends on the average yearly turbidity and average yearly flow rate. The general form of the equation is

$$C_{ocf} = (IR)(365)(COE)(Q_{ay})^{0.620} \qquad (A.17)$$

where C_{cof} is the O&M cost in dollars per year for coagulation-filtration, Q_{ay} is average yearly flow rate in mgd, and IR has a value of 1.62 (Metcalf & Eddy, 1967). The values for COE are presented in Table A.4 as a function of the average annual turbidity.

A.3.16 Construction Costs for Operation Building

No published data could be found for costs of operation buildings for water treatment plants. However, operation buildings perform approximately the same function in wastewater treatment plants as they do in water treatment plants. Construction costs for operation buildings used in wastewater treatment plants are available from Smith (1968) who gives this cost as

Table A.4. Coefficients for Coagulation, Flocculation, and Rapid Sand Filtration O&M Cost Equation

Average Annual Turbidity "ppm of SiO_2"	COE
100	78.0
90	75.0
80	71.2
70	68.0
60	65.0
50	61.2
40	57.2
30	53.7
20	51.2
10	47.5
0	45.0

$$C_b = (IR)(40,000)(Q_{dc})^{0.70} \tag{A.18}$$

where C_b is the construction cost of the operation building in dollars, Q_{dc} is design capacity in mgd, and IR has a value of 1.88. The life expectancy of the building was assumed to be 50 years.

A.3.17 General Operation and Maintenance Costs for Water Treatment Plant

The general O&M costs for the water treatment plant includes O&M costs for the operation building and costs for chlorination. These costs do not apply to the case where chlorination is the only water treatment process. The chlorine dosage is assumed to be 28.8 lbs per MG. The O&M costs were obtained from Koenig (1967). The cost equation is

$$C_{og} = (0.02)(C_b) + (IR)(28.8)(0.13)(VOL) \tag{A.19}$$

where C_{og} is the annual general O&M costs for the water treatment plant in dollars, IR has a value of 1.24, VOL is the amount of water processed in million gallons, and C_b is found from equation A.18.

A.3.18 Construction Costs for Buried Concrete Reservoir

The conventional buried concrete reservoir is used for treated water storage. The equation is

$$C_{bc} = (IR)(67,000)(VOL)^{0.606} \tag{A.20}$$

where C_{bc} is the construction cost of the buried concrete reservoir in dollars, VOL is storage capacity in MG, and IR has a value of 2.03 (Heiple, 1967). The life expectancy of the reservoir was assumed to be 50 years.

A.3.19 <u>Operation and Maintenance Costs for Buried Concrete Reservoir</u>

The O&M costs for the buried concrete reservoir are a function of capacity. The cost equation is

$$C_{obc} = (IR)(860)(VOL)^{0.211} \tag{A.21}$$

where C_{obc} is the O&M cost for the buried concrete reservoir in dollars per year, VOL is storage capacity in MG, and IR has a value of 2.03 (Heiple, 1967).

A.3.20 <u>Construction Costs of Surface Reservior</u>

The construction costs used here are applicable to an earth fill reservoir. The term construction costs encompasses land clearing, spillway construction, and relocations. Included in this equation are engineering services, contingencies, and land. The equation is

$$C_r = (IR)(9{,}160)(V_r)^{0.54} + (0.49)(L)(V_r)^{0.87} \tag{A.22}$$

where C_r is the construction cost of the reservoir in dollars, IR has a value of 1.84, V_r is the reservoir volume in ac-ft, and L is the cost of land, assumed to be $500 per acre (Dawes and Wathne, 1968). The life expectancy of the reservoir was assumed to be 50 years.

A.3.21 Operation and Maintenance Costs for Surface Reservoir

The O&M cost for reservoirs is a function of the capacity given as

$$C_{or} = (IR)(3,420)(10)^{(0.000066)(V_r)} \tag{A.23}$$

where C_{or} is the O&M cost for the reservoir in dollars per year, V_r is reservoir volume in ac-ft, and IR has a value of 1.98 (Koenig, 1966).

A.3.22 Costs of Intake Structure

No specific cost equation was found for intake structures for a reservoir. For large lakes and rivers Richardson (1969) has stated that the cost of an intake is 2 to 5 times greater than for a similar pipeline of the same length. The factor of 5 is used here and the diameter of the intake is assumed to be one commercial pipe size larger than the transmission line. The pipeline cost equations presented in equations A.8 and A.9 are used to calculate intake structure costs. The life expectancy of the intake was assumed to be 50 years.

APPENDIX B

DESCRIPTION OF COST CALCULATIONS

All calculations in the water supply systems analysis were programmed in the Fortran IV language for use with the IBM 360/67. In general, the programs are too lengthy to be reproduced here, but assumptions and arbitrary decisions are documented in this appendix. Flow charts are also presented to aid in interpretation of details.

B.1 Description of Common Concepts

Certain computational schemes pertaining to discounting procedures, stage construction, and treatment of salvage value are common to all evaluations. These are described below.

B.1.1 Discounting Procedures

All costs analyses were made on a present worth basis. All single investment costs occurring at different points along the planning horizon were reduced to equivalent expenditures at the beginning of the time horizon by the present worth formula

$$P = \frac{F}{(1 + i)^n} \tag{B.1}$$

where P is the equivalent present worth of a future amount F, at n years in the future, discounted at i percent per year (Grant and Ireson, 1970).

Costs which normally occur uniformly throughout a year, such as operation and maintenance costs, including electric power, are assumed to occur as one single payment in mid-year. These mid-year payments are brought back to present worth by employing a semi-annual discount rate and a modified discounting equation. The semi-annual discount rate, j, is calculated from the annual discount rate, i, as follows

$$j = (1 + i)^{0.5} - 1 \tag{B.2}$$

The present worth, P_A, of a payment, A, occurring in the middle of the n^{th} year is

$$P_A = \frac{A}{(1 + j)^{2.n - 1}} \tag{B.3}$$

B.1.2 Salvage Value Calculations

Another requirement for a valid comparison between alternative schemes is that all costs must be evaluated for the same period of analysis. If the end of the period of analysis and the end of the physical life of a water supply element do not coincide, an adjustment must be made through a negative cash flow or salvage value equal to the remaining value of the element at the end of the planning horizon. Throughout this study a straight line depreciation (James and Lee, 1971) was assumed and the salvage value is calculated by the expression

$$S = (1 - \frac{X}{L}) \cdot K \tag{B.4}$$

where S is the salvage value of the element, L is the length of its physical life in years, X is the length of its unused life in years, and K is its initial value.

There are several other procedures for computing salvage value, including the sinking fund method, the sum-of-the-years digits, and the declining balance methods. These various procedures lead to differing computed salvage values at various times along the planning horizon. Preferences for a particular method are based on tax considerations to a large extent.

The considerations are not relevant here, but it is still worth noting the effect of one method versus another. For example, the sinking fund method uses the expression

$$S = \frac{(1 + i)^L - (1 + i)^X}{(1 + i)^L - 1} K \tag{B.5}$$

where all terms have been defined previously. Figure B.1 shows that the sinking fund method gives salvage values higher than the straight line depreciation method at all points along the time horizon. More importantly, the sinking fund method causes the computed annual cost of a component to be the same in each year regardless of the period of the analysis, whereas the straight line depreciation method causes the computed annual cost to increase with decreasing period of analysis. This is shown in Figure B.2.

If the cost of a component whose initial value is $1,000,000 with a life expectancy of 40 years is analyzed for 20 years of service by the straight line depreciation method, the present worth cost of service would be $824,520; by the sinking fund method it would be computed as $740,211. Thus the sinking fund method is relatively more favorable to alternatives whose life expectancy exceeds the period of analysis.

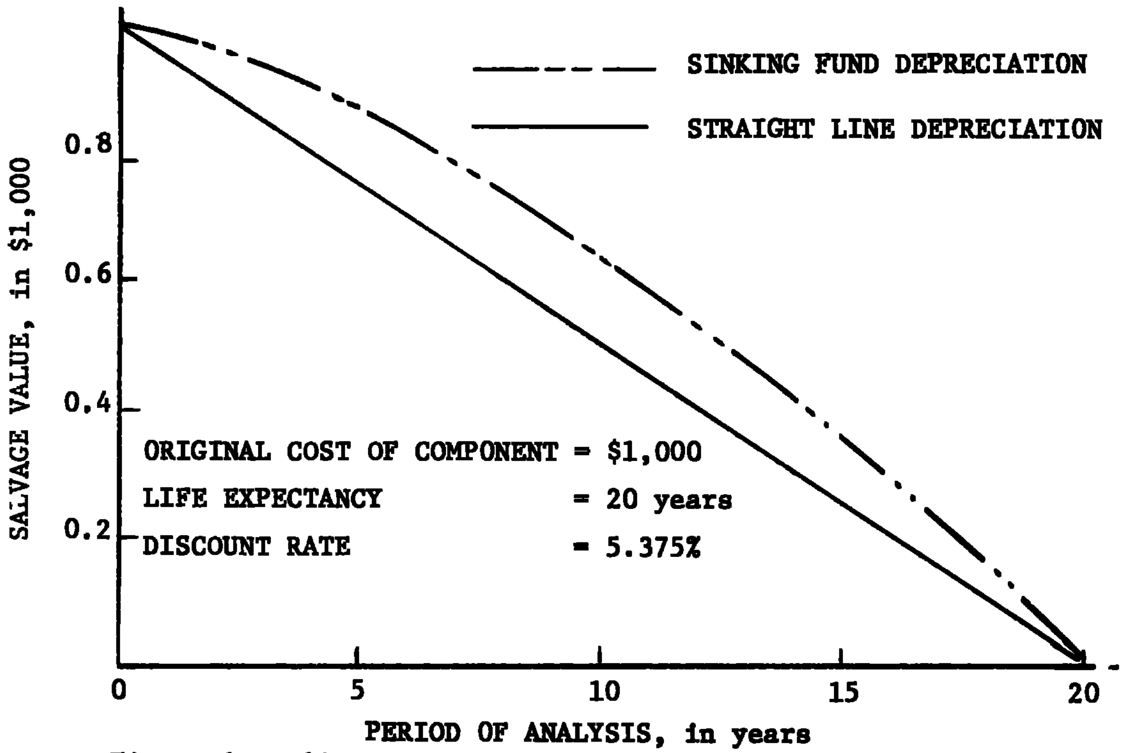

Fig. B.1 - Illustration of Salvage Value by Sinking Fund and Straight Line Depreciation Methods

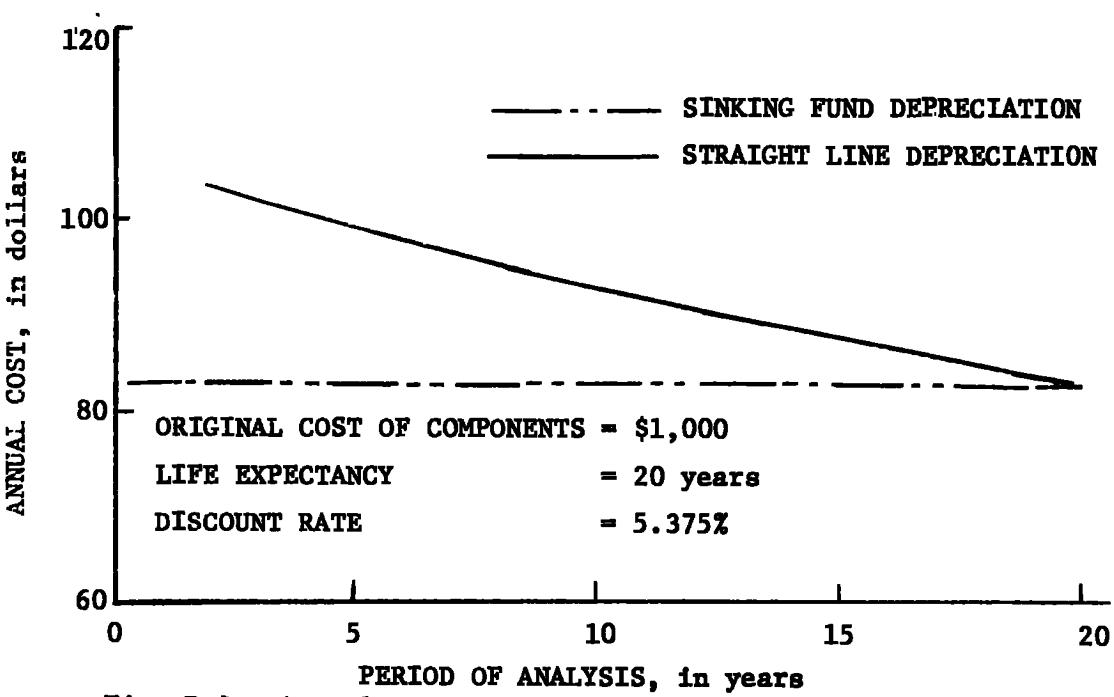

Fig. B.2 - Annual Cost Comparison of Salvage Value by Sinking Fund and Straight Line Depreciation Methods

There are ways of avoiding salvage value computations altogether. A period of analysis equal to the least common multiple of the life expectancies of the considered alternatives may be used, or an infinite project life can be used (Grant and Ireson, 1970). However, objections to these procedures can also be raised. In particular, long project lives may be unrealistic, and they do tend to bias calculations toward capital intensive alternatives.

B.1.3 Stage Construction

Proper stage construction can reduce costs significantly because of the combined effects of economies of scale and the time value of money. The concept of stage construction and a direct analytical solution for optimal staging in a simple case are described by Rachford et al. (1969). Most staging problems encountered in this study are too complicated to be analyzed by a direct analytical solution. Therefore, a staging policy based on judgment was developed. The adopted policy will not yield truly optimal solutions, but does introduce some of the advantages of stage construction into the cost evaluation.

The procedure is best illustrated by an example, using a time-capacity expansion schedule as shown in Figure B.3. The capacity rises from an initial value, A (5 units) to a maximum value, B (25 units), N years (40 years) later. The increase in capacity between years 1 and N expressed as a percentage increase is:

$$PI = \frac{100.\ (B - A)}{A} \qquad (B.6)$$

The number of stages, M, is determined by the magnitude of PI as follows

$$M = 1 \quad \text{if} \quad PI \leq 15 \tag{B.7a}$$

$$M = 2 \quad \text{if} \quad 15 < PI \leq 50 \tag{B.7b}$$

$$M = 3 \quad \text{if} \quad 50 < PI \leq 125 \tag{B.7c}$$

$$M = 4 \quad \text{if} \quad 125 < PI \tag{B.7d}$$

For the example, PI is 100 (25 - 5)/5 = 400. Hence, capacity installation will be performed in 4 stages.

Knowing the number of stages, the incremental capacity, S, to be installed in each stage is defined as follows

$$S = \frac{A - B}{M} \tag{B.8}$$

For the first stage the initial capacity A must be added to S.

For the example, S is (25 - 5)/4 = 5. Hence, capacities of 10, 5, 5, and 5 units are installed in years 1, 10, 20, and 30, respectively. The resulting relationship assumes the form of a step function as illustrated in Figure B.3.

B.1.4 Calculating Construction Costs

Construction costs are calculated repeatedly in the evaluation process and the basic scheme is identical for all components of the water supply system. Construction costs are always based on a design value associated with the maximum daily service requirement. For stage constructed components, this is the highest daily flow rate or storage volume requirement occurring during the last year of the staging increment. For

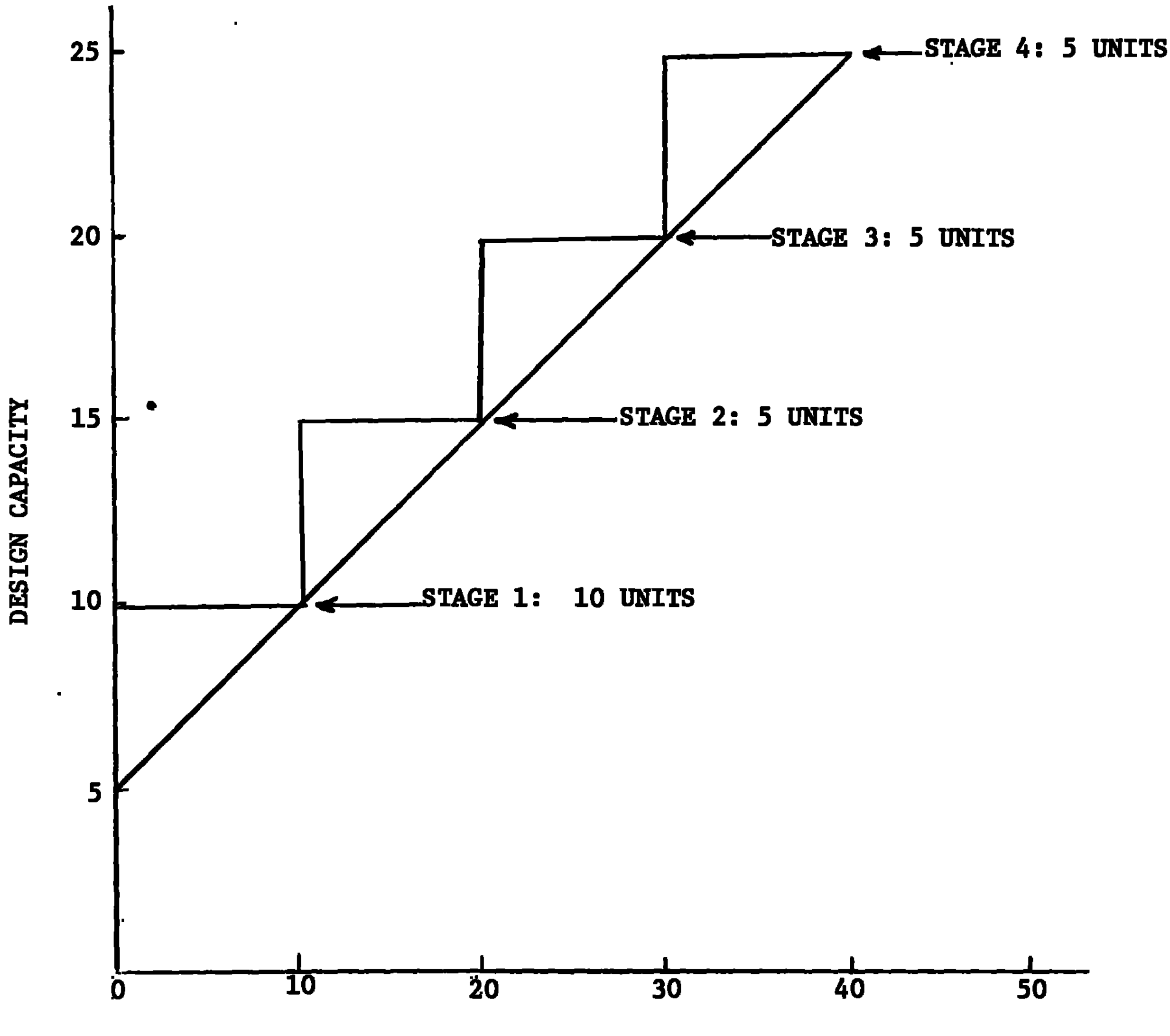

Fig. B.3 - Illustration of Stage Construction Concept components that are not stage constructed, this value is assumed to occur during the last year of the planning horizon.

The flow chart in Figure B.4 depicts the basic computational steps used in determining the present value of construction costs for each system component. Components without staging are treated as a trivial case of stage construction with only one stage..

B.1.5 Calculating Operation and Maintenance Costs

Operation and maintenance costs are also calculated repeatedly in the analyses performed. In this case, the operational design parameter is normally either a monthly or annual schedule of flow volume, energy utilization, or chemical dosages. In some instances O&M costs are computed directly as a fraction of initial construction costs. The flow chart in Figure B.5 demonstrates the general procedure for computing O&M costs. These calculations are tedious because O&M costs are not uniform but increase each year during the planning horizon in this computational scheme.

B.2 Calculations for Specified Cost Centers

The computer programs used are built around the identification of five cost centers:

1. Reservoir
2. Water Treatment
3. Surface Water Conveyance
4. Ground Water Pumping and Conveyance
5. Treated Water Storage

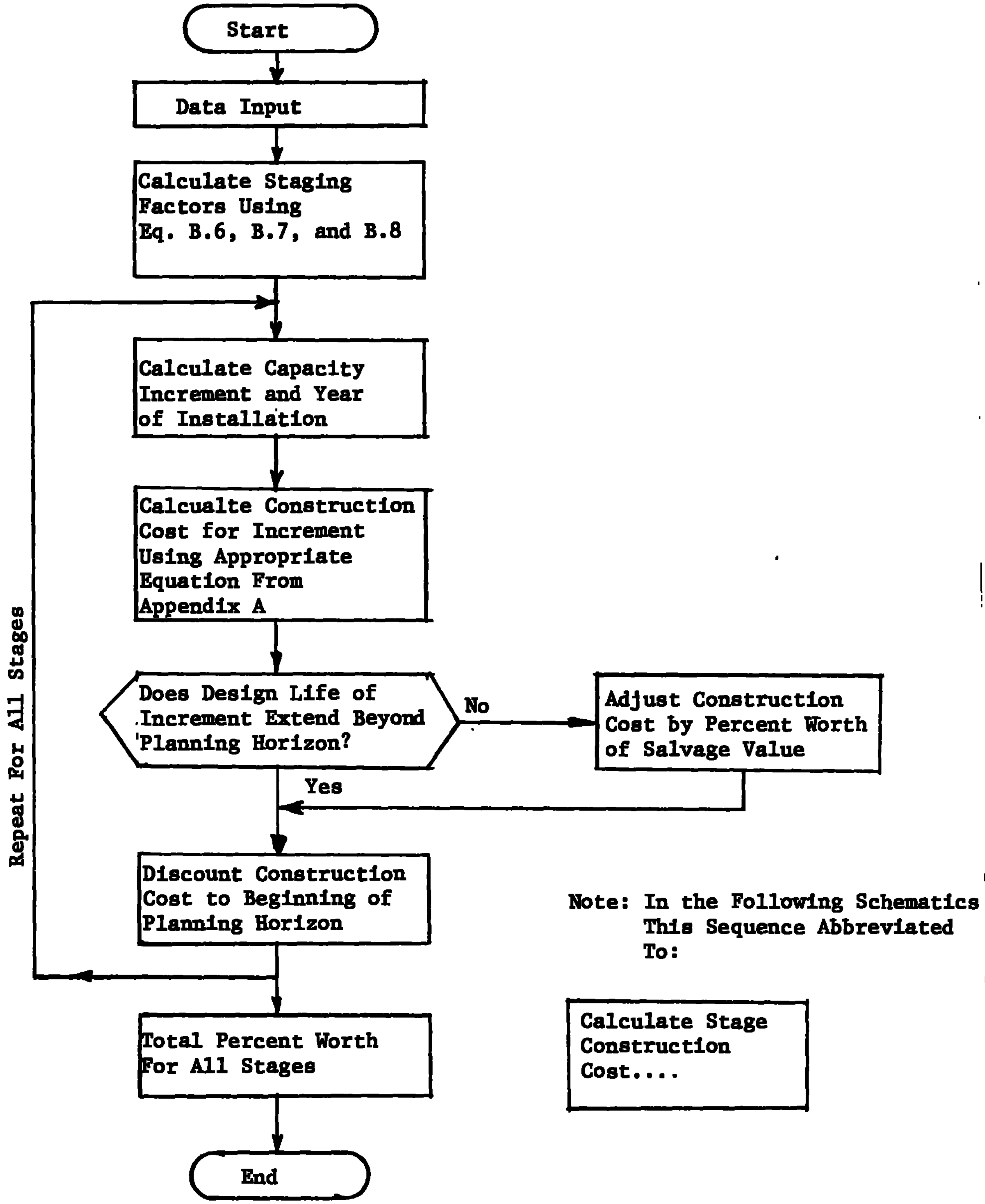

Fig. B.4 - Schematic of Stage Construction Capacity and Cost Calculation Procedure

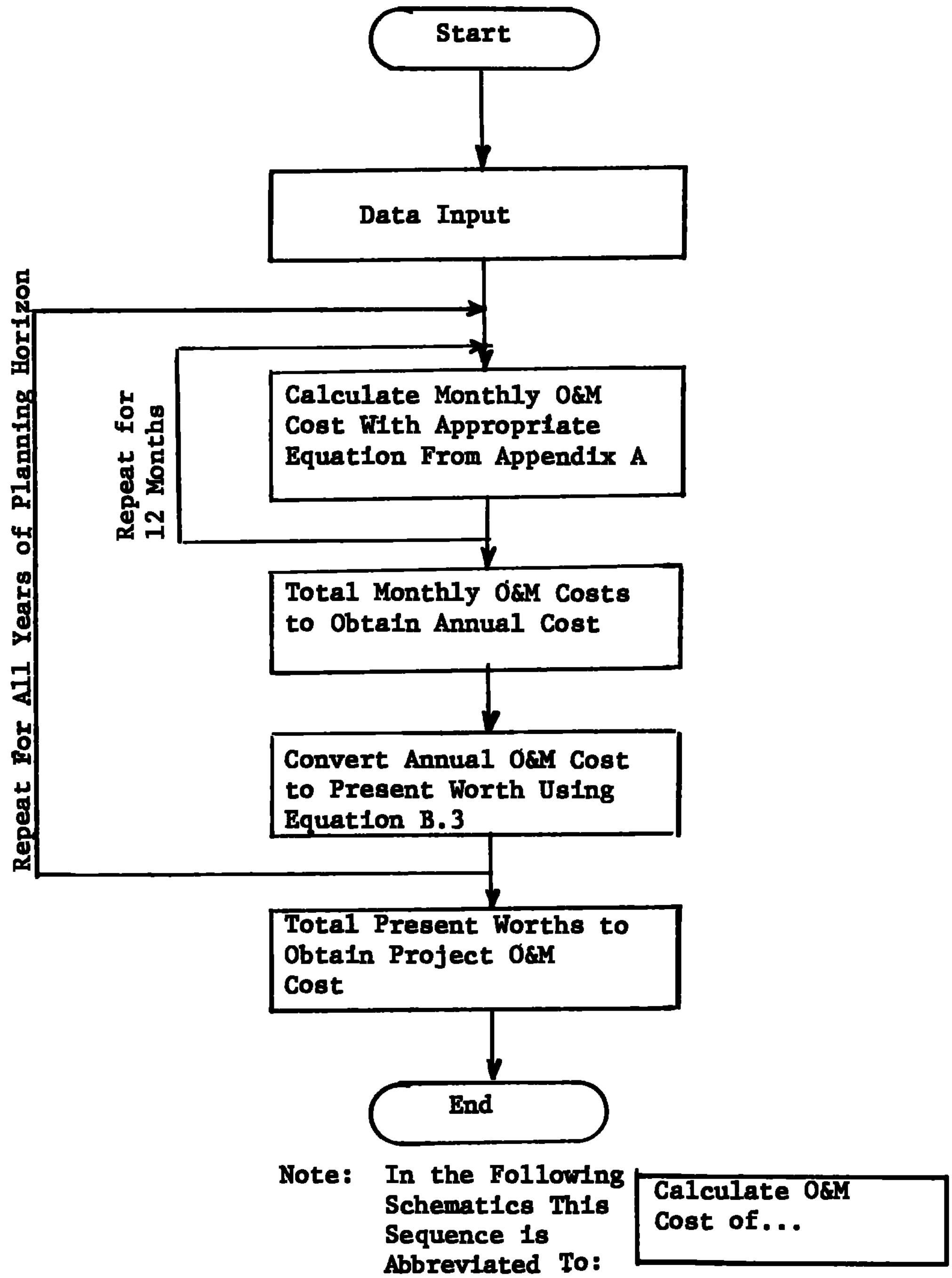

Fig. B.5 - Schematic of Operation and Maintenance Cost Calculating Procedure

An exclusive surface water source is evaluated by using programs associated with cost centers 1, 2, and 3. An exclusive ground water source would require use of cost centers 2 and 4. Cost center 5 is used with both ground and surface water if intermediate storage, as discussed in Chapter 6, is assumed to be used.

The boundaries of each cost center and the components within the cost center are shown in Figure B.6 for both ground and surface water systems. The cost equations associated with each cost center were shown previously in Table A.1.

Certain components, such as pipelines and pumps, are combined into a single program. This is done partly for convenience and partly because the optimal design often involves tradeoffs between different components. In an obvious case, for example, the increased cost of a larger pipeline leads to less friction losses and hence lower energy requirements for pumping.

B.2.1 Reservoir Sub-program

The reservoir cost center includes only the reservoir. The program requires specification of the reservoir capacity and the time at which the reservoir is assumed to be constructed. Capacity is based on the ultimate yield of the reservoir as determined in Chapter 4. Staging is not considered for reservoir construction, but calculations generally follow the procedure outlined by Figures B.4 and B.5.

B.2.2 Water Treatment Conveyance Sub-Program

This program was developed to handle any combination of the following treatment alternatives:

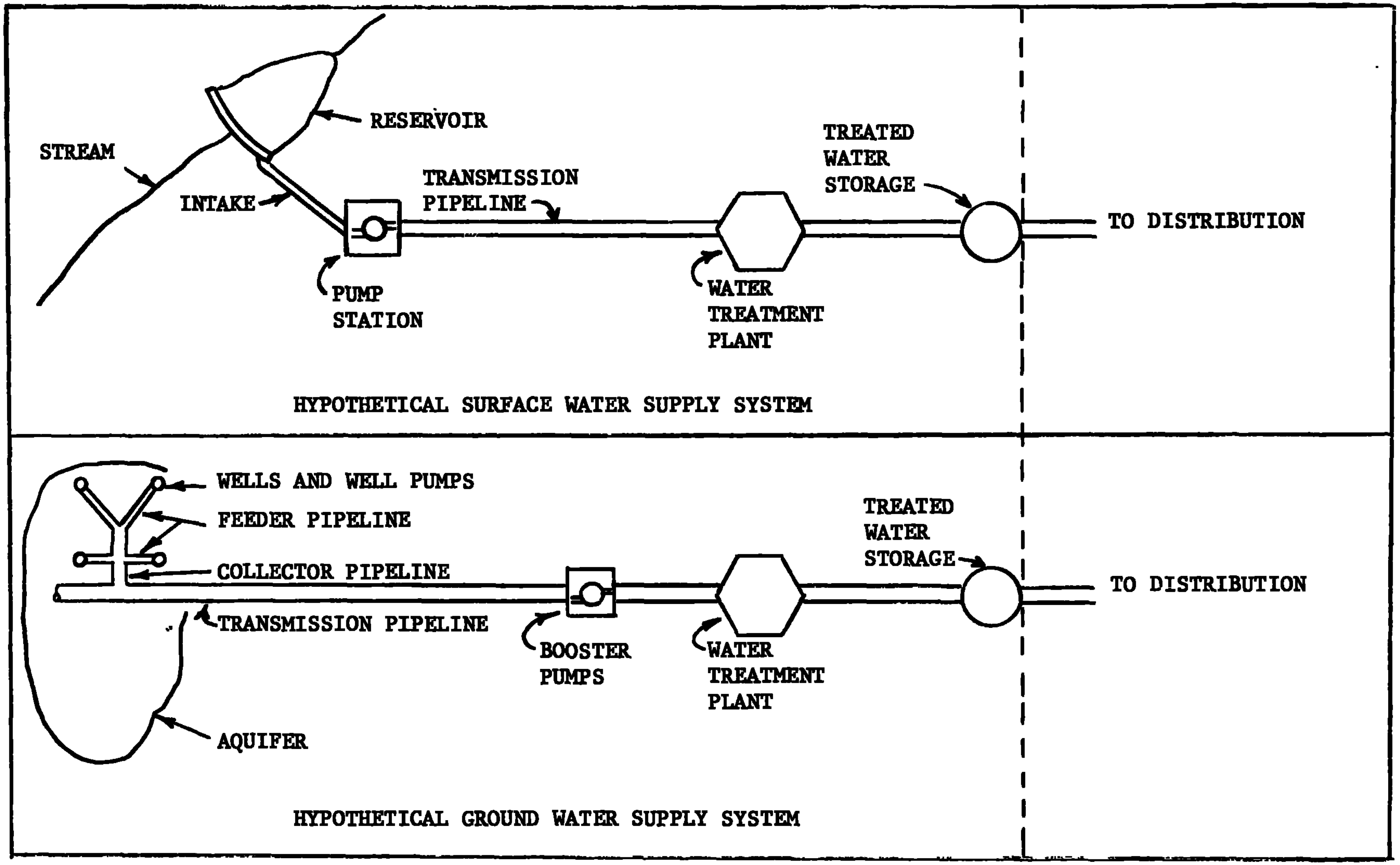

Fig. B.6 - Cost Centers and Components for Ground and Surface Water Supply Systems

1. Disinfection by chlorination

2. Turbidity removal by coagulation, flocculation and rapid sand filtration

3. Softening by lime-soda process.

Two levels of treatment of ground water were considered at various times in the study; in one case, only chlorination; in the other, both chlorination and softening. Surface water was assumed to require only chlorination plus turbidity removal. Treatment was assumed to meet U.S.P.H.S. standards (1962).

When ground water and surface water were used simultaneously, a "mixed" treatment process was also considered. Treated surface water and "hard" ground water were assumed to be mixed so that the resulting hardness of the mixture lies between the initial hardness for each source. The mixing ratio of the two sources can be calculated by the expression

$$\upsilon s/g = \frac{Cg - Cp}{Cp - Cs} \tag{B.9}$$

where $\upsilon s/g$ is the ratio of surface to ground water volume required to convert an initial ground water hardness concentration, Cg, and initial surface water hardness concentration, Cs, to a final permissible concentration, Cp. For example, if Cg = 200 mg/1, Cs = 50 mg/1, and Cp = 100 mg/1, surface and ground water would need to be mixed in a 2:1 ratio.

In the water treatment program, costs for ground water treatment and for surface water treatment are calculated separately. The input to the program requires specification of the following:

1. Yearly schedule of maximum daily flow rate

2. Schedule of average monthly flow volumes

3. Type of treatment desired

4. Cost coefficients from Table A.3.

All facilities except the operation building are constructed in stages. A flow chart of the water treatment cost program is presented in Figure B.7.

B.2.3 Surface Water Conveyance Sub-program

This cost center includes the intake, a pumping station, pumps and transmission line. In view of the hilly terrain and climate throughout the Eastern United States, conveyance exclusively by pipelines rather than canals was assumed.

The basis for all headloss calculations is the Hazen-Williams formula

$$S = \frac{K \cdot v^{1.852}}{D^{1.167}} \tag{B.10}$$

where S is the friction loss in a pipeline in ft/1,000 ft, v is the velocity of flow in the pipeline in ft/sec, D is the inside diameter of the pipeline in ft, and K is the friction factor.

Assuming a value of 0.501 for K (C = 100) and expressing v as Q/A where Q is the rate of flow through the pipeline in cfs, and A is equal to the area of the pipecross-section, the friction loss, SF, for a pipeline of a specific length, L, in miles becomes

$$SF = \frac{(4.96)(L)(Q)^{1.852}}{(D)^{4.871}} \tag{B.11}$$

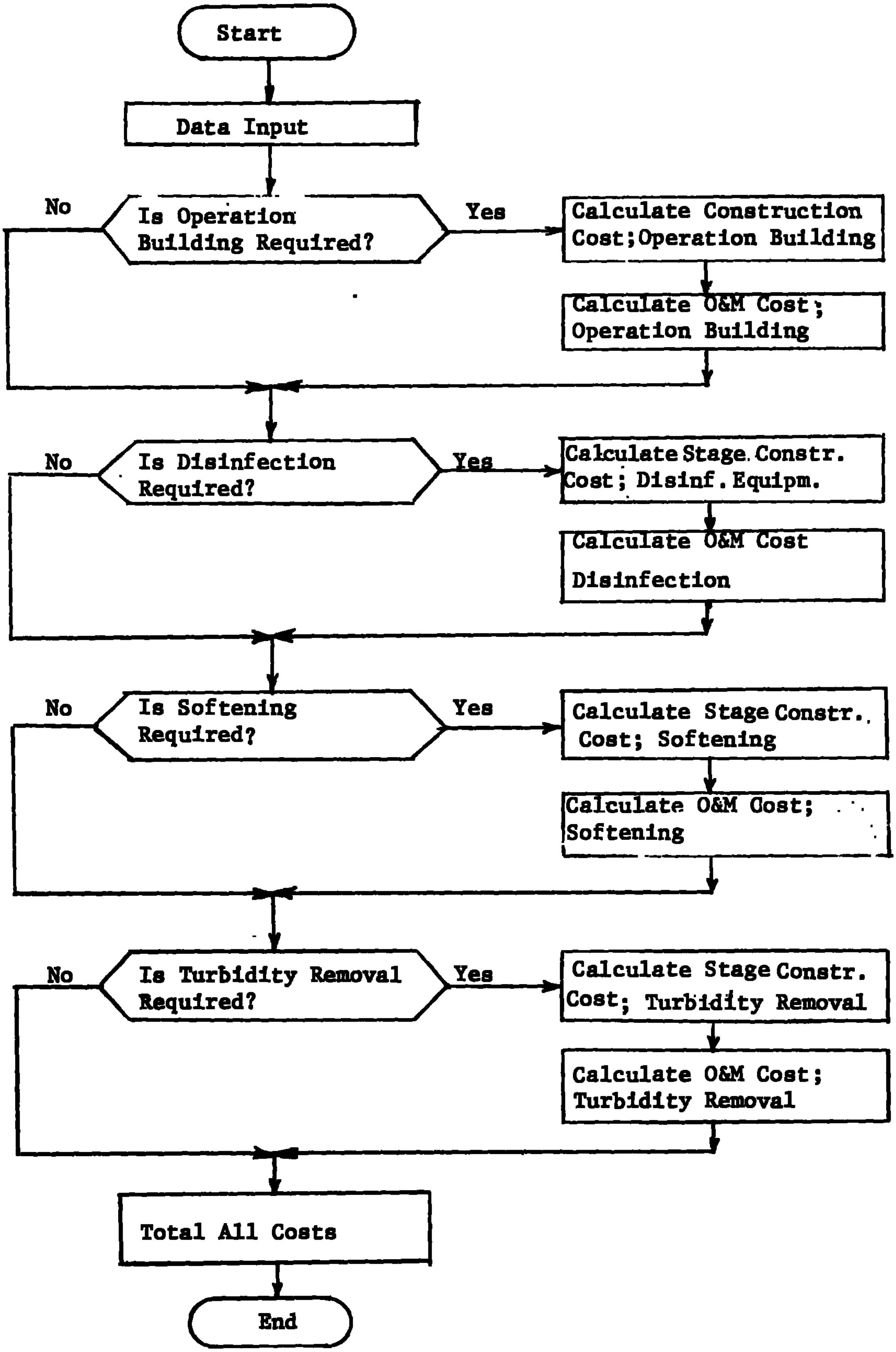

Fig. B.7 - Schematic of Water Treatment Capacity and Cost Calculation Procedure

The total headloss, ST, is the sum of friction loss, SF, and the hydrostatic head difference, H, between the pipeline endpoints

$$ST = SF + H \tag{B.12}$$

where H is assumed to be positive if the water is to be pumped against gravity.

The power required to pump a flow of magnitude Q against a head of ST is

$$HP = \frac{(Q)(W)(ST)}{(500)(\eta)} \tag{B.13}$$

where HP is the power required in horsepower, W is the weight of water in lb/ft^3 and η is the wire to water efficiency of the pump system.

Assuming for η a value of 0.7, the amount of energy required to pump a flow rate Q for t hours against a head of ST is

$$E = (0.75)(HP)(t) \tag{B.14}$$

where E is the energy requirement in Kilowatt-hours.

Water conveyance can be accomplished either by gravity flow, if the negative hydraulic head between the pipe endpoints outweighs the friction losses in the pipeline, or by pumping, if the hydrostatic head is not sufficient. The pipeline diameter, D*, necessary to facilitate gravity flow can be computed from Equation B.11 and B.12 by setting ST equal to zero and solving for D* by the expression

$$D^* = \left(\frac{(4.96)(L)(Q)^{1.852}}{|H|}\right)^{1/4.871} \tag{B.15}$$

Figure B.8 presents a schematic flowchart of the surface water transportation cost program. The input to this program requires the specification of the following items:

1. A yearly schedule of maximum daily flow rate
2. A schedule of average monthly volumes of flow for each year
3. Distance and hydrostatic head between pipeline endpoints.

Construction costs for all components are based on the yearly schedule of maximum daily flow rate. Operation and maintenance costs are calculated from the schedule of average monthly flow volumes, except for the pumping station. Pumping station O&M costs are based on equation A.7, which includes a term for maximum daily flow rate.

The pipeline and a 0.2 mile length intake are not considered to be stage constructed. The pumping station is treated according to the stage construction procedure.

For water conveyance by pumping, the tradeoffs between pipeline construction costs and pumping energy requirements are considered. The program determines by an iterative procedure the commercially available pipe diameter that minimizes these costs.

Gravity flow is also a possibility, and the program considers this option. First, the diameter D* is computed. Then the cost of gravity flow system is computed with the pumping station deleted. Finally, the gravity flow system is compared to a pumped conveyance system and the least cost alternative selected.

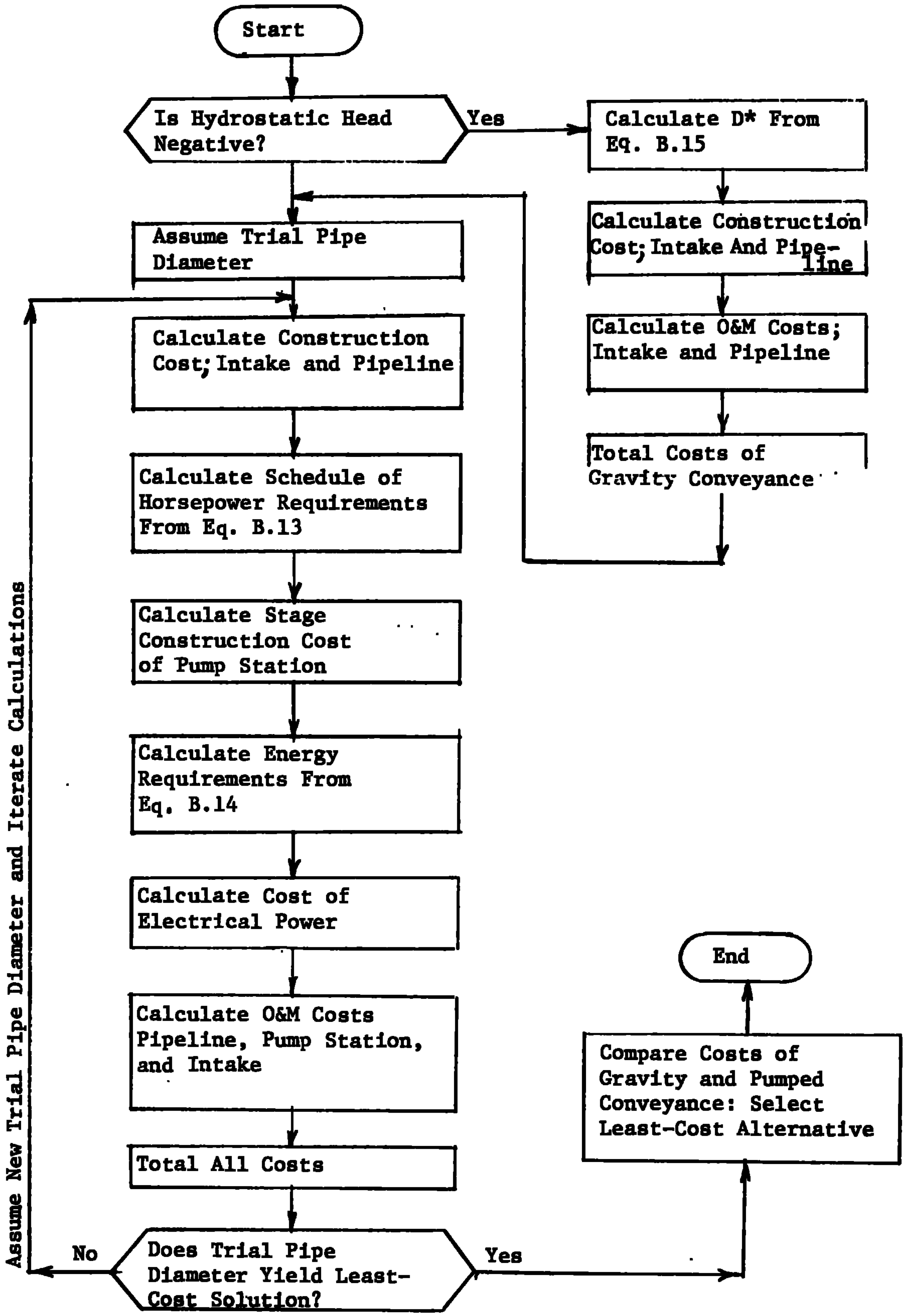

Fig. B.8 - Schematic of Surface Water Conveyance Capacity and Cost Calculation Procedures

B.2.4 Ground Water Pumping and Conveyance Sub-program

This cost center contains several components, including wells and well pumps, feeder pipelines, a collection pipeline, and a transmission line. The program itself is written in two parts. The first simulates the response of the aquifer to a schedule of monthly water usage, the second uses the resulting computed monthly average aquifer drawdowns to calculate ground water pumping costs.

A flow chart of the aquifer simulation procedure is shown in Figure B.9. This program requires specification of:

1. Schedule of average monthly pumping volumes
2. Aquifer characteristics
3. Wellfield configuration

The aquifer is described by the following parameters:

area, length, width, specific yield, permeability, depth to top, average thickness, maximum permissible drawdown, initial depth to water table, surface area recharge rate, river bed infiltration rate, length of river, and width of river.

The last three parameters appear only if the aquifer is recharged from a surface stream.

The drawdown simulation begins with a specified initial depth to the water table. The number of wells necessary to satisfy a specified monthly water withdrawal schedule is calculated, assuming a well load factor of 0.85 and a uniform discharge rate for all wells. The number of wells required is determined by taking the largest of either the peak flow rate to be met by the system, assuming a 100 percent pumping load factor, or the maximum monthly pumping rate using a 75 percent pumping

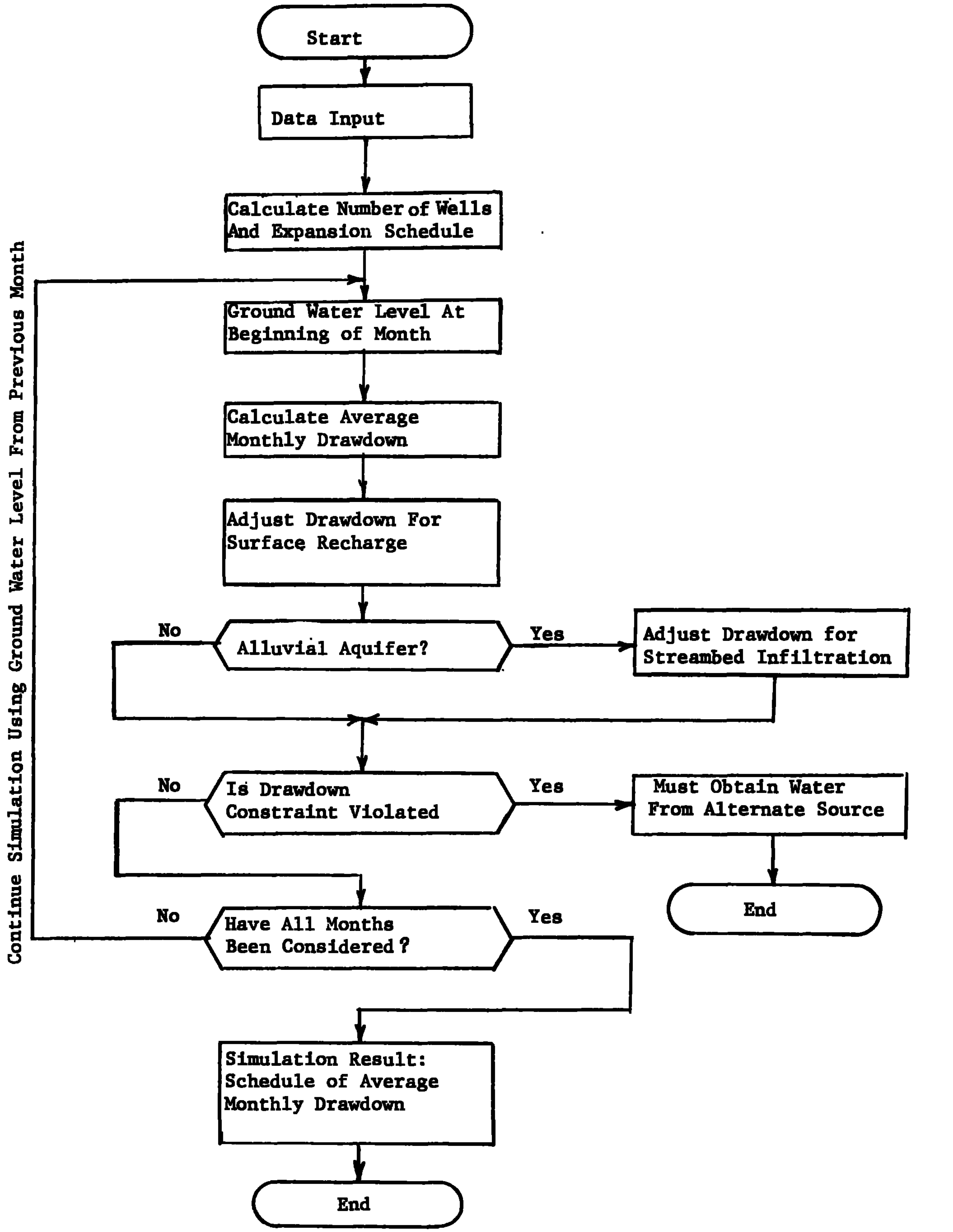

Fig. B.9 - Schematic of Monthly Aquifer Simulation Procedure load factor. One emergency well is always included. Wells were assumed to be arranged in a square grid with 1/2 mile spacing between wells and a maximum of 64 wells over the 20 mi^2 aquifer.

For every month during the planning horizon the drawdowns for each well are computed by means of Theis's non-equilibrium drawdown equation (Todd, 1969). Next, individual well drawdowns are superimposed, and monthly average drawdowns are determined. For every month adjustments are made for aquifer recharge from the ground surface and, if desired, from streambed infiltration. If drawdowns exceed a predetermined value of permissible drawdown, the simulation terminates, indicating that the aquifer cannot fulfill the input requirements.

The second part of the program calculates the cost of the ground water pumping and conveyance system as shown in Figure B.10. This program requires the following input specifications:

1. Annual schedule of maximum daily flow rates
2. Schedule of monthly flow volumes
3. Lengths of feeder, collector, and main pipeline
4. Hydrostatic head difference between average well elevation and treatment plant
5. Cost coefficients according to Table A.2
6. Number of wells and schedule of average monthly aquifer drawdowns as determined in the aquifer simulation program.

Construction costs are based on the annual schedule of maximum daily flow rates, and O&M costs are computed from the schedule of monthly flow rates. Construction of wells, well pumps, and feeder and collector pipelines proceeds in stages. Since all these facilities are dependent

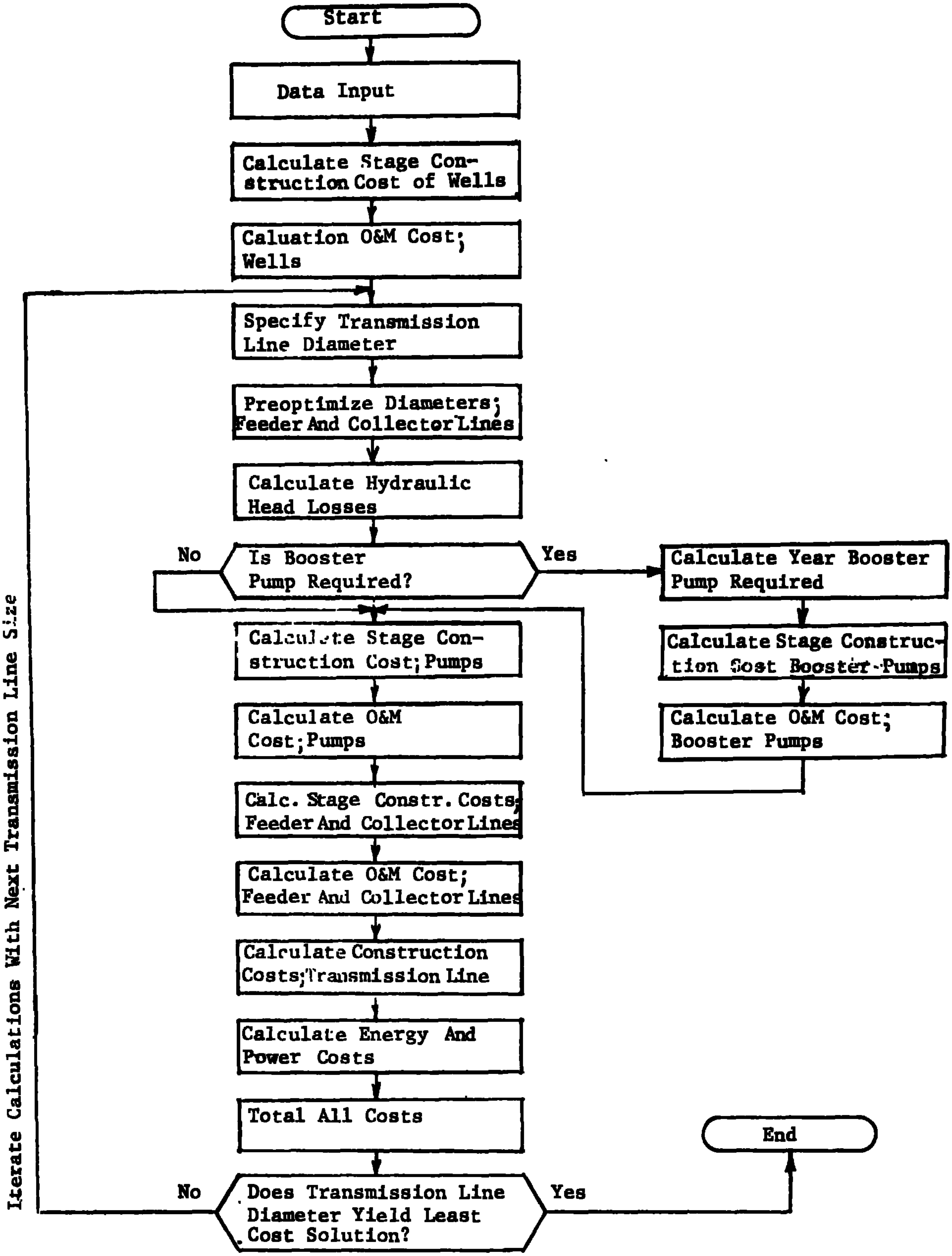

Fig. B.10 - Schematic of Ground Water Pumping and Conveyance Capacity and Cost Calculations on each other, they are installed in an identical staging pattern.

The wellfield is arranged according to a recurring pattern of wells as shown in Figure B.11. The length of the feeder pipeline from the wellhead to the collector pipeline is assumed to be 0.75 miles, and connects 4 wells to the transmission line. The diameters of each feeder and collector pipeline were preoptimized using a steady state formula presented in Linaweaver (1964).

The diameter of the transmission line is optimized by means of an iterative solution identical to the one used in the surface water program. For low or average total hydraulic head conditions, the well pumps provide all the energy to lift the water from the ground water table to the treatment plant. If the total head of the system exceeds a certain magnitude (assumed 700 ft in this study) booster pumps are installed.

B.2.5 Treated Water Storage Sub-program

This cost center includes only the buried concrete reservoir. This program requires specification of:

1. Yearly schedule of maximum daily flow rate
2. Schedule of average monthly flow volumes.

A general flow chart of the procedure is shown in Figure B.12. This procedure must be combined with the procedures described in section B.2.2 and B.2.3 or B.2.2 and B.2.4 because the purpose of the intermediate storage is to reduce the required design capacity and risk of shortage for the entire supply system by attenuating the effect of peak day consumption rates.

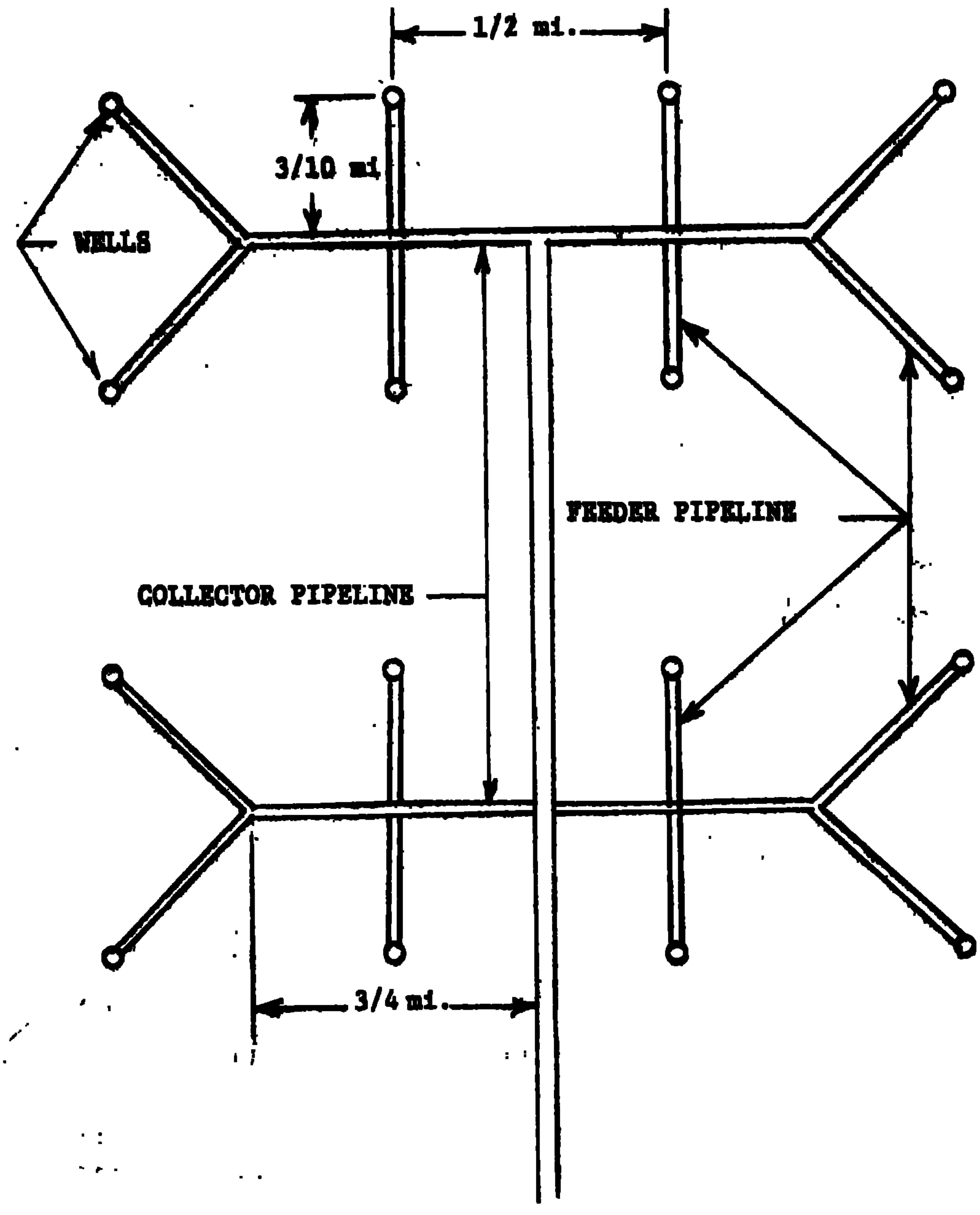

Fig. B.11 - Typical Wellfield Patterns

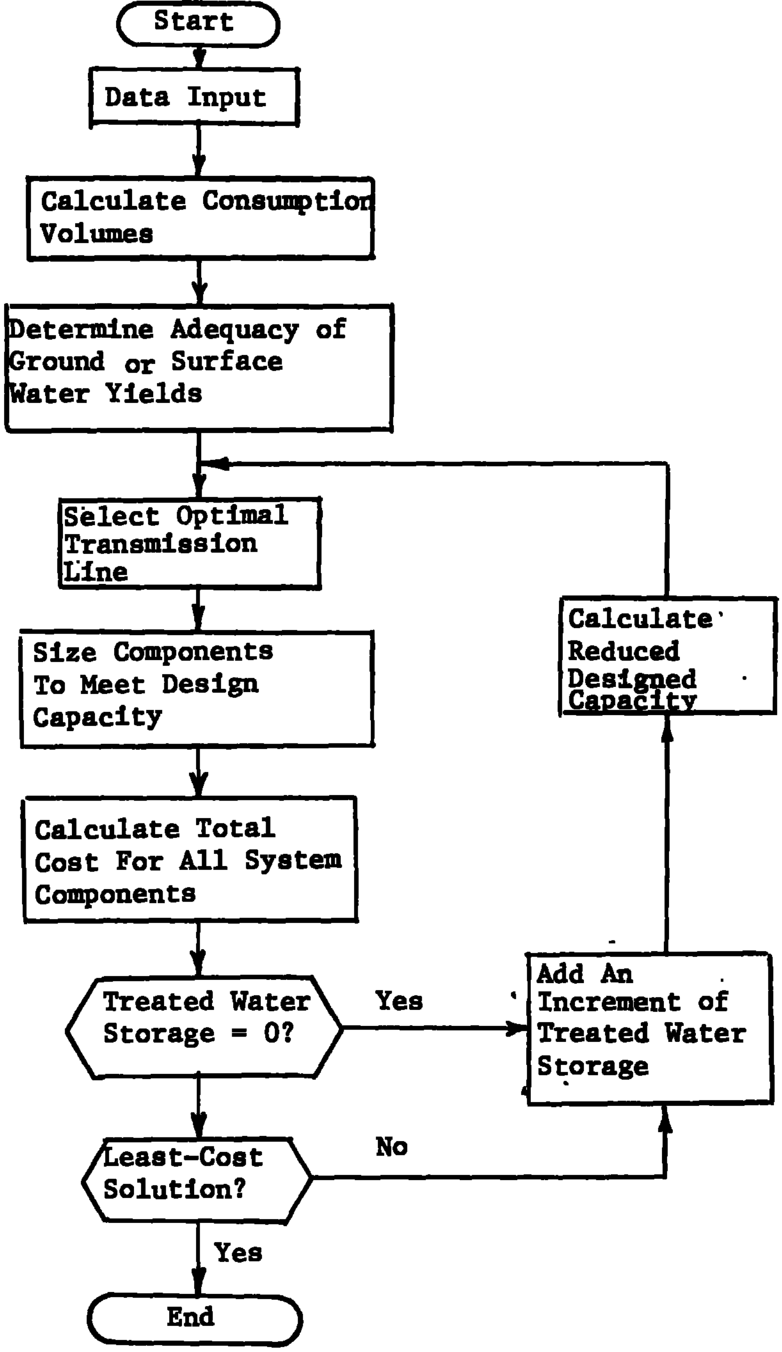

Fig. B.12 - Schematic of Treated Water Storage Capacity and Cost Calculation Procedure

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The report attempts to examine and quantify the role of ground water in future allocations of surface water storage for water supply. It identifies the circumstances under which integrated use of ground and surface water may be desirable in humid regions such as the Appalachian. The report develops a methodology for analysis and includes a case study.

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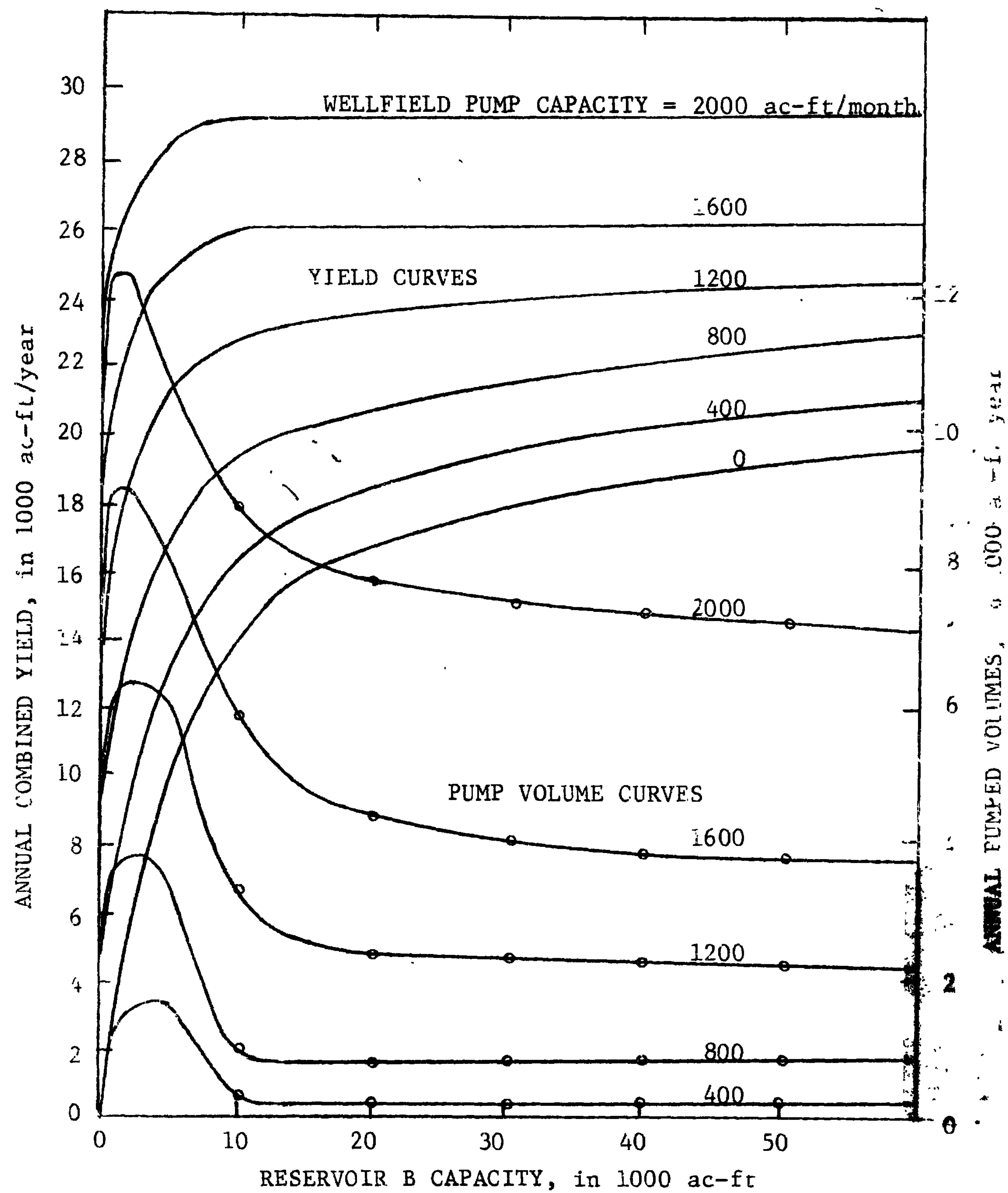
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RESERVOIR B CAPACITY, in 1000 ac-ft
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