ECONOMIC CONCEPTS AND TECHNIQUES

PERTAINING TO WATER SUPPLY, WATER ALLOCATION, AND WATER QUALITY

Compiled from IWR Contract Reports and Related Studies

A Report Submitted to the
U. S. Army Engineer Institute for Water Resources
Kingman Building
Fort Belvoir, Virginia 22060

Under Contract No. DACW31-76-M-0435

Approved for public release; distribution unlimited

December 1975

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E C O N O M I C C O N C E P T S A N D T E C H N I Q U E S

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by

Eric D. Bovet

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Presents in a systematic manner economic concepts and techniques helpful in analyzing alternative solutions to problems commonly found in planning for water supply and water quality. Seven economic studies published by the US Army Engineer Institute for Water Resources between 1971 and 1974, and related reports, were selected as source material for the reference manual. These do not cover in detail all facets of water supply, water allocation

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and water quality. The contents of the IWR sponsored studies fall under three main headings: Water Supply, Water Allocation and Water Quality. Waste Water Renovation for Reuse, which provides an alternative source of supply, was treated under water quality because of the problems raised by varying user tolerance to water of varying degrees of purity.

PREFACE

A need has been sensed among planners in the U.S. Army Corps of Engineers for a reference manual in which would be distilled the essence of a number of water supply, water allocation, and water quality reports sponsored in recent years by the Center for Economic Studies, Institute for Water Resources.

These subjects have become so complex and the literature so abundant that project designers cannot be expected to be familiar with all the developments or memorize all details germane to their work. A concise volume was called for that would present in a systematic manner economic concepts and techniques helpful in analyzing alternative solutions to knotty water resource problems.

Seven economic studies published by IWR between 1971 and 1974, and some related reports, were selected as source material for the reference manual. They do not cover in detail all facets of the water-supply, water-allocation, water-quality field. An effort was made by IWR, of course, in selecting and sponsoring research projects within funding limitations, gradually to round out the subject matter. New aspects were added to the research program from time to time to supplement findings already acquired. Nonetheless, an attempt at systematic coverage in this reference volume was limited by sources available and selected for inclusion.

The seven studies are listed here in chronological order of their publication dates. They are identified in the manual by their IWR Contract Report designations. They will be found covered in the numbered sections of the manual as indicated below:

Publication Date	Title	Author	IWR Contract Report	Section of Manual
August 1971	Costs and Effects of a Water Quality Program for a Small Strip Mining Company	G. Richard Dreese, Department of Economics, W.Va. University, Morgantown, W.Va., and Harold L. Bryant, Department of Economics and Finance, Xavier University, Cincinnati, Ohio	71-7	IX
May 1972	A Methodology for Assessing Economic Risk of Water Supply Shortages	G. K. Young, R. S. Taylor, and J. J. Hanks, Water Resources Engineers, Inc., Springfield, Va.	72-6	IV
February 1974	A Method for Integrating Surface and Ground Water Use in Humid Regions	Gert Aron, Thomas Rachford, John Borrelli, and Walter Stottmann, The Pennsylvania State University, University Park, Pa. 16802	74-3	I, II, III
July 1974	Interregional Planning of Water Resources Allocations by Systems Analysis Approach -- A Summary Report Four reports and four professional papers on related subjects	John E. Keith, Jay C. Andersen, Alton B. King, Mark H. Anderson, Thomas C. Anderson, Calvin C. Clyde, and Daniel H. Hoggan, Utah Water Research Laboratory, College of Engineering, Utah State University, Logan, Utah 84322	74-4	VI + 5 APPENDICES

Publication Date	Title	Author	IWR Contract Report	Section of Manual
October 1974	Changing Water Use in Selected Manufacturing Industries	Robert A. Leone, J. Royce Ginn, and An-Loh Lin, National Bureau of Economic Research, New York, N.Y. 10016, and the Graduate School of Business Administration, Harvard University, Boston, Mass. 02163	74-10	V
December 1974	An Evaluation of Water Reuse for Municipal Supply	Daniel Dworkin, Holcomb Research Institute, Butler University, Indianapolis, Ind., and Duanne D. Baumann, Department of Geography, Southern Illinois University, Carbondale, Ill.	74-11	VIII
December 1974	Evaluation of Quality Parameters in Water Resource Planning	Eric D. Bovet, Consultant, Alexandria, Va. 22308	74-13	VII

The contents of the above research reports were structured under three main captions: Water Supply, Water Allocation, and Water Quality. Waste water renovation for reuse, which provides an alternate source of water supply, was placed under Water Quality because of the problem raised by varying user tolerances to water of varying degrees of purity.

The resulting structure comprises the following sections:

ECONOMIC CONCEPTS AND TECHNIQUES PERTAINING TO WATER SUPPLY, WATER ALLOCATION, AND WATER QUALITY

Compiled from IWR Contract Reports and Related Studies

Section		IWR Contract Report
	FIRST PART. WATER SUPPLY	
I.	Planning for Surface Water Supply A. Estimating Water Demand B. Sizing Water Supply	74-3
II.	Planning for Ground Water Supply and Mixed Water Use	74-3
III.	Water Supply Costs	74-3
IV.	The Risk of Water Supply Shortages	72-6
V.	Industry's Response to Changing Water Costs	74-10
	SECOND PART. WATER ALLOCATION	
VI.	Interregional Planning of Water Resource Allocation Appendix A. Water Allocation in an Urbanizing Agricultural Region of Utah Appendix B. State-Wide Water Resource Allocation Model Appendix C. Inter-Basin Water Transfers for Utah's Agriculture Appendix D. Demand for Agricultural Water Appendix E. Professional Papers by Members of the Utah Water Research Laboratory	74-4
	THIRD PART. WATER QUALITY	
VII.	Planning for Water Quality A. Water Quality Concepts B. Water Quality Enhancement Technology and Cost C. Water Quality Benefits D. Water Quality Optimization	74-13
VIII.	Waste Water Renovation for Reuse	74-11
IX.	Control of Acid Mine Drainage	71-7

The manual is intended, through distillation of the several IWR Contract Reports and related studies, to provide, within the bounds of a modest volume, the essence of the findings deemed of value to the water resource planner. An effort was made to facilitate access to relevant topics by presenting the gist in systematic manner. The reader will be guided to passages of interest to him by consulting the subject index at the end of the manual.

Each section is preceded by a brief statement describing the object, scope, and applicability of the source report. The derivation of all data is carefully noted, so the reader can always refer back to the original IWR Contract Report or related study for more detailed information or clarifications.

Thus, the manual may serve the planner as a road sign to available economic concepts and techniques developed under IWR sponsorship, pertaining to various aspects of water supply, water allocation, and water quality.

TABLE OF CONTENTS

Page

FIRST PART. WATER SUPPLY

I. PLANNING FOR SURFACE WATER SUPPLY

A. Estimating Water Demand 1

1. Annual Water Demand 1
2. Monthly Water Demand 1
3. Daily Water Demand 2
4. Hourly Water Demand 2
5. Forecasting Population Growth 2
6. Effects of Errors in Population Forecasts 3
7. Decision Functions 3

B. Sizing Water Supply 5

1. Water Shortage Index 5
2. Water Yield 6
3. Safe Design Yield 6
4. Reservoir Routing Procedure 7
5. Technique for Selecting a Synthetic Flow Sequence 8
6. Design Yields from Surface Reservoirs 9
7. Determination of Ultimate Capacities for Several Supply Alternatives 9

II. PLANNING FOR GROUND WATER SUPPLY AND MIXED WATER USE

1. Joint Ground and Surface Water Use 1
2. Regional Development and Environmental Quality 1
3. Surface and Ground Water Quality 1
4. Methodology for Achieving an Optimal Water Supply 2
5. Determination of Design Yield 3
6. Ground Water Pumping Regimen 3
7. Ground Water Pumping as Emergency Back-Up 4
8. Preventive Pumping to Forestall Water Supply Shortages 6
9. Yield from Aquifer as Exclusive Source 7
10. Surface and Ground Water Supply Integration 9
11. General Description of Supply Component Quantification and Cost Calculation Procedures 10
12. Sensitivity Study of Integrated Surface and Ground Water Use 11

III. WATER SUPPLY COSTS

1a. Construction Costs for Surface Reservoir 1
1b. Operation and Maintenance Costs for Surface Reservoirs .. 1
2. Construction Costs for Intake Structure 2
3a. Construction Costs for Transmission Line 2
3b. Operation and Maintenance Costs for Transmission Line ... 2
4a. Construction Costs for Pumping Station 3
4b. Operation and Maintenance Costs for Pumping Station 3
5. Electrical Power Costs 4
6a. Construction Costs for Treatment Plant 4
6b. Operation and Maintenance Costs for Treatment Plant 5
7a. Construction and Equipment Costs for Chlorination System 5
7b. Operation and Maintenance Costs for Chlorination System 6
8a. Construction Costs for Coagulation, Flocculation, and Rapid Sand Filtration 6
8b. Operation and Maintenance Costs for Coagulation, Flocculation, and Rapid Sand Filtration 6
9a. Construction Costs for Treated Water Storage Reservoir .. 7
9b. Operation and Maintenance Costs for Treated Water Storage Reservoir 8
10a. Construction Costs for Well 8
11a. Installation Costs for Well Pump 9
12a. Installation Costs for Booster Pump 10
10b, 11b, 12b. Operation and Maintenance Costs for Well Field 10
13a. Construction Costs for Lime-Soda Softening 10
13b. Operation and Maintenance Costs for Lime-Soda Softening 11

IV. THE RISK OF WATER SUPPLY SHORTAGES

1. Methodology 1
2. Loss Functions 4
3. Water Shortage Risk Determination 9

V. INDUSTRY's RESPONSE TO CHANGING WATER COSTS

1. The Theoretical Framework 1
2. Price Elasticities of Water 7

SECOND PART. WATER ALLOCATION

VI. INTERREGIONAL PLANNING OF WATER RESOURCE ALLOCATION

1. Introduction 1
2. Hydrology and Study Areas 2
3. The Model 4
4. Supply Coefficients and Schedules 7
5. Demand Coefficients and Schedules 8
6. Allocation Solutions 12
7. The Cost of Inefficient Allocations 14
8. The Cost of Policy Decisions 15
9. Conclusions 16

Appendix A. Water Allocation in an Urbanizing Agricultural Region of Utah

1. Introductory Remarks 1
2. The Demand for Irrigation Water 3
3. The Supply of Water 4
4. Optimal Resource Allocation 5

Appendix B. State-Wide Water Resource Allocation Model

1. Systems Analysis Applications to Water Resource Planning 1
2. Economic Analysis Relevant to Resource Allocation 3
3. Systems Analysis Model for Optimal Allocation of Water Resources in Utah 14
4. Optimal Solution from the Model 18
5. Post-Optimization Analysis 22
6. Conclusions 23

Appendix C. Inter-Basin Water Transfers for Utah's Agriculture

1. Inter-Basin Allocation Theory 1
2. Water Transfer Costs 6
3. Mathematical Programming Model 7
4. Simplifying Assumptions 8
5. Water Allocation Optimization 12
6. Results from the Model 14

Appendix D. Demand for Agricultural Water

1. Theory 1
2. Assumptions 2
3. Rotation, Costs, Yields, Water Use 3
4. Results 5

Appendix E. Professional Papers by Members of the Utah Water Research Laboratory

1. Optimal Allocation of Water Resources in Utah 1
2. Jordan River Basin Regional Water Resource Allocations: A Systems Analysis Approach 4
3. A Systems Analysis Approach to Water Resource Allocations in Utah 5
4. Determining Economic Costs of Economically Non-Optimal Public Policy 6

THIRD PART. WATER QUALITY

VII. PLANNING FOR WATER QUALITY

A. Water Quality Concepts 1

1. Water Quality Classifications 1
2. Water Quality Indices 3
3. List of Water Uses 4
4. Contaminants Added Through Utilization of Public Water Supply 5
5. Raw Water Quality Criteria 6
6. Treated Water Quality Standards and Goals 8
7. Water Quality Models 11a

B. Water Quality Enhancement Technology and Cost 11b

1. Natural Instream Reoxygenation 11b
2. Artificial Instream Reaeration 12
3. Gravity Quality Storage Reservoirs 14
4. Costs of Impounding Reservoirs 14
5. Pumped Quality Storage Reservoirs 18
6. Well Supply Costs 20
7. Artificial Ground Water Recharge 23
8. Intake and Pumping Stations 23
9. Blending Proportions for Water from Several Sources ... 25
10. Transmission Pipelines 27
11. Raw Water Storage 32
12. Raw Water Treatment Plant and Storage 33
13. Treated Water Pumping Stations 37
14. Water Allocation to Meet Quantity and Quality Needs ... 38
15. Selection of a Suitable Desalination Process..... 42
16. Desalination Costs 42
17. Waste Water Outfall Management 44
18. Sewage Treatment for Disposal 45
19. Sewage Treatment for Reuse 48
20. Trade-Offs Between Waste Water Disposal and Reuse 51

21. Storm Water Treatment 51
22. Industrial Waste Water Treatment 52
23. By-Product Recovery from Industrial Effluents 55
24. Collective Waste Water Treatment 56
25. Land Disposal of Waste Water 60
26. River Bed and Lake Purification 60
27. Excess Vegetation Control 61
28. Thermal Pollution Control 61
29. Radioactivity Control 63
30. Municipal Treatment Charges to Industry 64
31. Waste Treatment Cost Allocation 64

C. Water Quality Benefits 66

1. The With-Without Criterion 66
2. Willingness to Pay 66
3. Alternative Cost 67
4. Damages Avoided 68
5. Land Values 68
6. Stochastic Nature of Water Quality 69
7. Time Discounting of Benefits 70
8. Benefits of Preserving Irreplaceable Resources 70
9. Benefits Accruing to M + I Water Customers 71
10. Benefits Accruing to Patrons of Water-Based Recreation Facilities 71
11. Benefits Accruing to Users of the Waste-Assimilative Capacity of Water Bodies 72
12. Marginal Costs and Marginal Benefits 72

D. Water Quality Optimization 75

1. Optimal Water Supply Augmentation Involving Desalination 75
2. Optimal Water Supply Protection Through Reaeration ... 76
3. Optimal Quality Storage Reservoir Releases 77
4. Optimal Water Supply Allocation from Multiple Sources with a Single Water Quality Parameter .. 78
5. Optimal Water Supply Allocation from Multiple Sources with Multiple Quality Parameters 87
6. Optimal Effluent Charges 91
7. Instream Water Quality Management 92
8. Optimal Waste Treatment 93
9. Optimal Instream Water Quality Maintenance 96
10. Typical Planning Problems Involving Water Quality Considerations 97

VIII. WASTE WATER RENOVATION FOR REUSE

1. Rationale for Waste Water Reuse as Potable Supply 1
2. Planning for Dependable Water Supply 2
3. Projecting Water Demand 4
4. Safety of Waste Water Reuse as Potable Supply 4
5. Economic Efficiency of Waste Water Reuse 5
6. Acceptability of Waste Water Reuse 6
7. The Future of Renovated Waste Water 7
8. A Model for Waste Water Reuse 8
9. Results of Waste Water Reuse Simulation 10
10. General Principles Governing Waste Water Reuse 12

IX. CONTROL OF ACID MINE DRAINAGE

1. Methods and Techniques 1
2. The Demand Function 2
3. The Supply Function 4
4. Economic Feasibility of Mine Drainage Control and Land Reclamation in Strip Mining 5

SUBJECT INDEX

LIST OF FIGURES

Fig. 1. Relationship Between Design Yield and Reservoir Capacity Without Ground Water Supplementation I-10

Fig. 2. Annual Combined Yield for a Stream and an Aquifer Used as Back-Up Source ..... II- 5

Fig. 3. Annual Combined Yield and Ground Water Volume Pumped for a Reservoir and an Aquifer Under the Preventive Pumping Rule ..... II- 8

Fig. 4. Present Worth Analysis: Total Yearly Risks Plus Reservoir Construction Cost IV-11

Fig. 5. Determination of the Choice of Inputs ..... APP.B- 7

Fig. 6. Determination of the Choice of Outputs APP.B-10

Fig. 7. Determination of the Relationship Between Output and Input APP.B-12

Fig. 8. Flow Diagram of Allocation Model APP.B-17

Fig. 9. Water Allocation for the Year 1965 ..... APP.B-20

Fig. 10. Water Allocation for the Year 2020 ..... APP.B-21

Fig. 11. Demand and Supply Between Competing Uses ..... APP.C- 3

Fig. 12. Excess Demand in HSU-1 ..... APP.C- 3

Fig. 13. Water Allocation in HSU-2 ..... APP.C- 3

Fig. 14. Efficient Allocation in HSU-1 ..... APP.C- 5

Fig. 15. Excess Supply in HSU-2 ..... APP.C- 5

Fig. 16. Efficient Allocation in HSU-1 Using Excess Supply .... APP.C- 5

Fig. 17. Industrial Water Use Systems VII-54

FIRST PART

W A T E R S U P P L Y

I

PLANNING FOR SURFACE WATER SUPPLY

I. PLANNING FOR SURFACE WATER SUPPLY

Source:

A Method for Integrating Surface and Ground Water Use in Humid Regions, by Gert Aron, Thomas Rachford, John Borrelli, and Walter Stottmann, The Pennsylvania State University, University Park, Pa. 16802.

IWR Contract Report 74-3 February 1974

Scope and Use

The primary objective of the study 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.

The fact that water is relatively abundant does not assure a reliable water supply. The risk of shortages is not dependent on the absolute quantity of average annual yield, but is more a function of hydrological 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 of 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 any more assure a reliable water supply for their service region without infringing on competing multi-purpose 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.

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 source(s) 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 operating scheme should both sources be managed?

To answer the above four questions, 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. The second phase emphasized analytical and numerical verification of the concepts, and a case study test of their applicability. The city of Elmira, New York was selected for the case study.

The passages considered particularly relevant to Section I were found essentially in chapters 3 (Description of a Generalized Water Supply System), 4 (Methodology for the Determination of a Least-Cost Water Supply Alternative), and 6 (Subsidiary System Design Considerations). In those chapters, a methodology is provided for determining, for any given water demand center, the economically most efficient development schedule and management regimen for surface and ground water resources. Concepts and techniques relating to surface water supply development were selected for Section I.

The concepts, principles, and methods seem generally applicable. Figures were selected by the authors within realistic ranges of possibilities so as to lend credibility to the concepts, but were not necessarily based on documented empirical findings. However, a substantial bibliography is listed.

I. PLANNING FOR SURFACE WATER SUPPLY

A. Estimating Water Demand

1. Annual Water Demand

When sufficient information is available, demand curves can be used to develop a schedule of yearly demand rates. Or, annual water demand can be based on the price of water and the elasticity of residential per capita water demand, and on fixed ratios of commercial and industrial demands to residential demand. The average residential per capita use may be assumed to increase at a rate of 0.25% per year, subject to responses to price changes. In humid regions, demand may be considered inelastic to price changes.

74-3: p. 3-5.

2. Monthly Water Demand

Average monthly demand rates, expressed as a percentage of annual use, may be developed from data reported in the following publications:

Babbitt, Harold E. and Doland, James J.: Water Supply Engineering, McGraw-Hill Book Co, New York, 1955.

California Water Resources Department: Planned Utilization of Ground Water Basins: Coastal Plain of Los Angeles County, Appendix C -- Operation and Economics, Bulletin No. 104, State of California, Sacramento, California, December 1966.

74-3: p. 3-7.

3. Daily Water Demand

The maximum day demand rate is equivalent to the largest daily volume of water used during a given year. Ratios of maximum to average day demand range from 1.50 to 2.90. A ratio of 2.20 is representative of typical water utilities.

74-3: p. 3-6.

4. Hourly Water Demand

Hourly demand rates are likewise compared with average day rates. The ratio of the maximum hourly rate to the average day rate is much higher than that of maximum day to average day rate. It ranges between 2.0 and 7.0. But, because of built-in treated water storage, the maximum hourly demand rate does not affect the design capacity of the water plant itself. Only the distribution system needs to accommodate the highest instantaneous demand rate.

74-3: p. 3-6.

5. Forecasting Population Growth

Water demand is a function of population. Population rarely remains static. The planning horizon for water supply may span 25 to 50 years. Forecasts for such durations are subject to error. The forecast error, in percent, can be expressed as 100 times the absolute value of the predicted minus the actual population count, divided by the predicted figure.

It has been claimed that there is approximately a 1% error in forecasting for each year of prediction.

74-3: p. 6-13 to 6-15.

6. Effects of Errors in Population Forecasts

When population growth is incorrectly forecast, the normal pattern of costs for installing and operating a water supply system is distorted. If the system is too small due to an underpredicted population, the O + M cost will generally be greater than for a system designed for the actual population. If the population is overpredicted, the initial investment will normally 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.

74-3: p. 6-15.

7. Decision Functions

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 of 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.

In most situations, uncertainty or at least risk is involved. Decision criteria that can be used for decision making under uncertainty include those of pessimism (minimax solution) and optimism (minimum solution). Since these decision criteria select one extreme or the other, a more conservative criterion may be desired. The equal-likelihood criterion can be used for both uncertainty and risk. In the latter case, a probability distribution function of forecast errors must be 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:

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

where: EC_i = expected cost of the ith alternative

d_j = probability of the jth outcome

C_{ij} = total cost of the ith alternative for the jth outcome

n = number of possible outcomes

Using the 1% error in forecasting for each year of prediction, a 50-year forecast is subject to a + or - 50% error. An initial population of 20,000 is estimated to increase to 44,500 within 50 years. Forecast errors of -50%, -25%, 0%, +25%, and +50% would result in outcomes of 22,250, 33,375, 44,500, 55,625, and 66,750, with each outcome being equi-probable.

74-3: p. 6-16 to 6-21.

B. Sizing Water Supply

1. Water Shortage Index

The fact that water is relatively abundant does not assure a reliable water supply, as proved by recent instances of water shortages in the humid Eastern U.S. The risk of shortages is a function of the hydrologic variability of water yield, a condition more pronounced in surface than in ground water sources. There is always a small chance of surpassing the worst drought generated. Thus, the risk of rare shortages must be incorporated in any study.

A monthly shortage index, which can be related to economic losses resulting from water supply deficits, may read as follows:

$$MSI_i = \left(\frac{B \cdot SH_i}{D_i}\right)^A$$

where: MSI_i = shortage index for month

SH_i = volume of water shortage during month

D_i = target draft during month

A = parameter whose magnitude reflects the relative severity of rare major shortages versus more frequent minor shortages. When A is set at 1.6, a 10% shortage occurring once in a given time span is equally severe as a 5% shortage accurring three times.

B = multiplier. When B is set at 2, the monthly index value is unity for a shortage equaling 50% of target draft.

An optimal shortage index can be obtained by balancing: marginal costs of increased system capacity resulting from a lowered shortage index, against marginal losses incurred when the index is increased. In the absence of an optimal index, a maximum shortage index can be set as a goal by the water utility. For example, an arbitrary value of 0.05 can be chosen for the maximum allowable average annual shortage index. The annual shortage index is the sum of the monthly shortage indices.

74-3: p. 1-1, 1-2, 4-8, 4-9, 4-11.

2. Water Yield

The concept of water yield makes use of a water shortage index which quantifies the level of occasional shortages. Surface water yield is calculated from a series of synthetically generated stream flows. This yield is compared with water usage based on projected population trends, and the shortage index computed accordingly.

74-3: p. 2-1.

3. Safe Design Yield

Time distribution of run-off from natural watersheds is a stochastic phenomenon. The greatest problem in estimating water yields from surface streams is the uncertainty of the representativeness of historical droughts as indicators of future drought potential. In recognition of this uncertainty, 1000 years of continuous streamflow may be synthesized for each stream on the basis of the mean, standard deviation, and skew of the historical sample.

74-3: p. 4-8.

4. Reservoir Routing Procedure

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

$$S_i = S_{i-1} + Q_i - LF_i - D_i,$$

subject to the constraint:

$$0 < S_i < S_{max}$$

where: S_i = reservoir storage at end of month

Q_i = volume of reservoir inflow during month

D_i = volume of target draft during month

LF_i = volume of required low flow release during month

S_{max} = storage capacity of reservoir

If, at the end of a month, the volume of water in the reservoir exceeds reservoir capacity, the excess is assumed spilled and the storage volume is set at 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 at zero.

Some assumptions pertaining to reservoir routing may be useful:

(1) Highest seasonal run-off occurs in early spring; therefore, the 12-month period May 1 to April 30 represents the most advantageous routing year. The reservoir will be nearly filled at the end of each routing year, and successive routing years will be quasi-independent of prior shortages.

(2) Evaporation from reservoir surface, in humid areas, can be assumed to be balanced by direct precipitation on the lake surface.

(3) Only during periods when the reservoir inflow exceeds the predetermined 10-year frequency 1-month low flow limit can inflows be retained in the reservoir.

(4) The conservation storage space in the reservoir is unaffected by sedimentation or allocation to multi-purpose 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 computed by dividing the sum of yearly shortage indices by the sequence length N.

74-3: p. 4-11 to 4-14.

5. Technique for Selecting a Synthetic Flow Sequence

The generation of 1000 years of synthetic streamflow, while reducing the uncertainty of drought prediction, raises two questions:

(1) How long a synthetic flow sequence should be used for the design yield determination?

(2) Could a particular subsequence be chosen as representative of the entire 1000-year flow sequence generated?

To save computer time, the 1000 years of synthetic flow can be divided successively into two 500-year, four 250-year, ten 100-year, twenty 50-year, and fifty 20-year subsequences. The design yields of a given reservoir can be computed for all of these subsequences. The coefficients of variation of the yield (standard deviation divided by mean yield) will exhibit a continuous decrease with increasing sequence length, but the mean yield will show a pronounced decrease as the sequence length rises, then level off markedly after a particular duration.

In the interest of conserving computer time, it can be assumed adequate to use sequences of the duration after which the mean yield exhibits a leveling off. Individual design yields of those sequences should then be compared for the purpose of choosing one sequence that will most closely represent the entire sample. Curves should be constructed for various reservoir sizes within the range of possible requirements. Deviations from the mean of the yields obtained for the sequences selected should be compared, and the sequence closest to the mean yield of the reservoir may be chosen for all subsequent yield determinations.

74-3: p. 4-15 to 4-18.

6. Design Yields from Surface Reservoirs

By means of an iterative technique, of the selected synthetic flow sequence, and of an average annual shortage index of 0.05 for example, a series of relationships between reservoir capacity and design yield can be established. Figure 1 on page I-10 shows such relationships.

74-3: p. 4-19 to 4-20.

7. Determination of Ultimate Capacities for Several Supply Alternatives

Ultimate capacities are those needed to satisfy the utility's water requirements during the last year of the planning horizon. The time-of-supplementation is the latest date on which additional facilities must be available if requirements are to be met. Present population, the annual population growth rate, the average annual demand, and the maximum day demand, are all needed to establish ultimate capacities of several potential water sources. 74-3: p. 4-32.

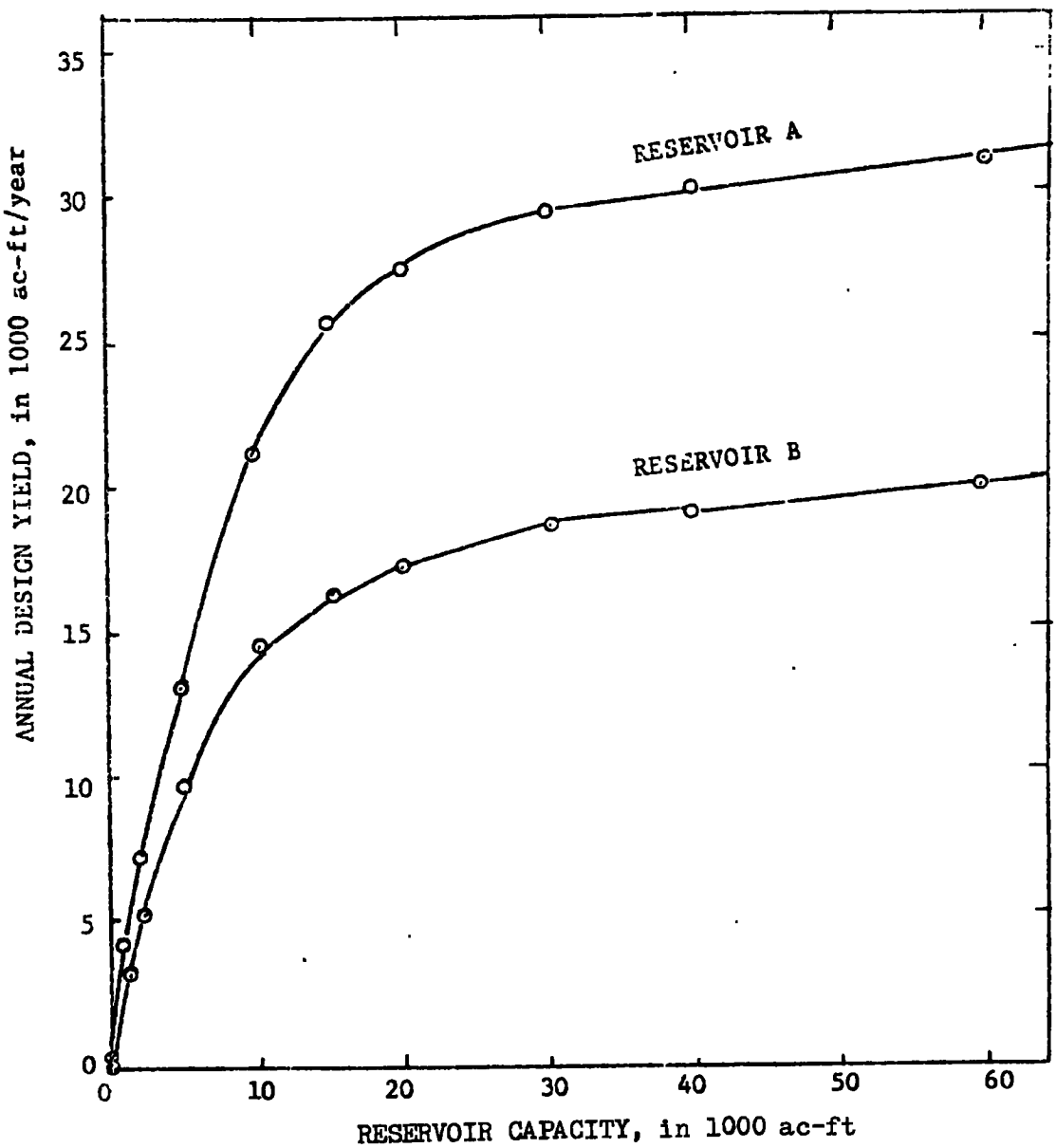


Figure 1 -- Relationship Between Design Yield and Reservoir Capacity Without Ground Water Supplementation

II

PLANNING FOR GROUND WATER SUPPLY AND MIXED WATER USE

II. PLANNING FOR GROUND WATER SUPPLY AND MIXED WATER USE

Source:

See Section I

IWR Contract Report 74-3 February 1974

Scope and Use

The justification, objective, and scope of IWR Contract Report 74-3 were presented at the head of Section I.

Passages deemed germane to Section II were found essentially in chapters 4 (Methodology for the Determination of a Least-Cost Water Supply Alternative), and 7 (Feasibility of Integrated Use -- Sensitivity Study). Concepts and techniques relating to ground water supply development, as well as to integrated surface and ground water use, were chosen for Section II.

It is recognized that the Corps of Engineers has traditionally concentrated on developing surface water sources and supplies. Whether ground water development will some day become a part of a Corps water project is a matter of conjecture. But the techniques available for evaluating the feasibility of aquifer development can at least be used by the Corps to determine alternative costs and benefits.

II. PLANNING FOR GROUND WATER SUPPLY AND MIXED WATER USE

1. Joint Ground and Surface Water Use

The term Conjunctive Water Use has been applied to semi-arid regions where artificial recharge of aquifers is essential for effective water conservation. For humid regions where recharge is not considered necessary, the term Integrated Water Use is preferred.

74-3: p. 1-1.

2. Regional Development and Environmental Quality

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

74-3: p. 1-2.

3. Surface and Ground Water Quality

Representative values for average yearly turbidity and hardness in typical surface and ground water sources may be, for example, 50 ppm of SiO_2 and 40 ppm of hardness as $CaCO_3$ in surface water, and negligible amounts of SiO_2 but 200 ppm of hardness as $CaCO_3$ in ground water.

74-3: p. 3-11.

4. Methodology for Achieving an Optimal Water Supply

A step-by-step procedure is needed for designing an approximately optimal water supply system from a set of feasible alternatives. Optimality is assured by a system that approaches least cost while meeting a specified water requirement.

Step 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 well field capacities. The relative contribution of each source may vary between 0 and 100%. Considering the stochastic nature of yield from systems containing surface water sources, the yield determination is based on simultaneous reservoir-aquifer routing with a series of synthetic monthly streamflow requences. The final result is a set of curves representing combined yield as a function of reservoir size and aquifer pump capacity.

Step 2. Determination of Feasible Source Combinations

The combined yield curves and the annual water demand schedule circumscribe the range of combinations of well field and reservoir capacities that can satisfy the requirements in the last year of the planning horizon. The information can be presented by means of a time-of-supplementation curve, which indicates the point in time when the initial source, a reservoir, requires supplementation from a second source, another reservoir or an aquifer. A yield isoquant curve shows the possible trade-offs between supplemental reservoir and aquifer development.

Step 3. Determination of Annual Schedules of Design Flow Rates

Several water supply combinations, uniformly distributed over the feasible range of isoquants, are chosen for testing. Annual schedules of maximum day flow rates and of monthly low volumes, pertaining to surface and ground water transportation, pumping, and treatment, are established. Design flow schedules of maximum day flow rates specify the capacities which each supply source, as well as associated pumps, pipelines, and treatment facilities, must equal or exceed during each year of the planning horizon. A schedule of use volumes is developed for each month of every year during the planning horizon.

Step 4. Calculation of Costs

Cost calculations require an input of primary decision variables, design flow schedules, and physical input parameters. They produce values for secondary decision variables and individual as well as total costs.

Step 5. Identification of Least-Cost Combinations

Costs of the source combinations selected earlier are compared. The combination with the lowest cost constitutes the optimal combination. Its cost and resulting decision variables are retained as representative for the alternative under consideration. Since the response surface may not be concave, a judicious examination of alternatives is required.

74-3: p. 4-2 to 4-7.

5. Determination of Design Yield

Design yield is computed in an iterative procedure, routing inflows and outflows through reservoirs and aquifers, and registering spills and shortages. The following factors must be specified: Synthetic flow sequence, low flow requirement, permissible shortage index, reservoir capacity, ground water storage capacity, and ground water pumping capacity. A first trial value for annual target draft is selected. If the average annual index matches the specified limiting index within the given tolerance, the design yield has been established; otherwise, the trial target draft rate must be adjusted.

74-3: p. 4-11 and 4-12.

6. Ground Water Pumping Regimen

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 back-up resource to be called on whenever surface water shortages arise has been found highly ineffective. Under such circumstances, the maximum rate of ground water delivery, rather than the extent of aquifer storage, tends to be the limiting factor. There is high likelihood that ground water reserves would be subject to demands beyond well field capacity. Ground water pumping should, therefore, start well ahead, and in anticipation, of shortages.

74-3: p. 2-7.

7. Ground Water Pumping as Emergency Back-up

The use of ground water to fill shortages in surface water supply when they arise constitutes a back-up system that is effective when the well pump capacity is rather large relative to the projected withdrawal rate. For reservoir capacities larger than ten times the monthly well field capacity, the yield increase due to the back-up source becomes relatively minor, and a schedule of preventive pumping is recommended. Figure 2 shows the annual combined yield of a reservoir and back-up aquifer without preventive pumping. Back-up aquifers are effective only in combination with very small reservoirs or where annual well field capacities approximate the annual demand volume.

74-3: p. 4-19 to 4-21.

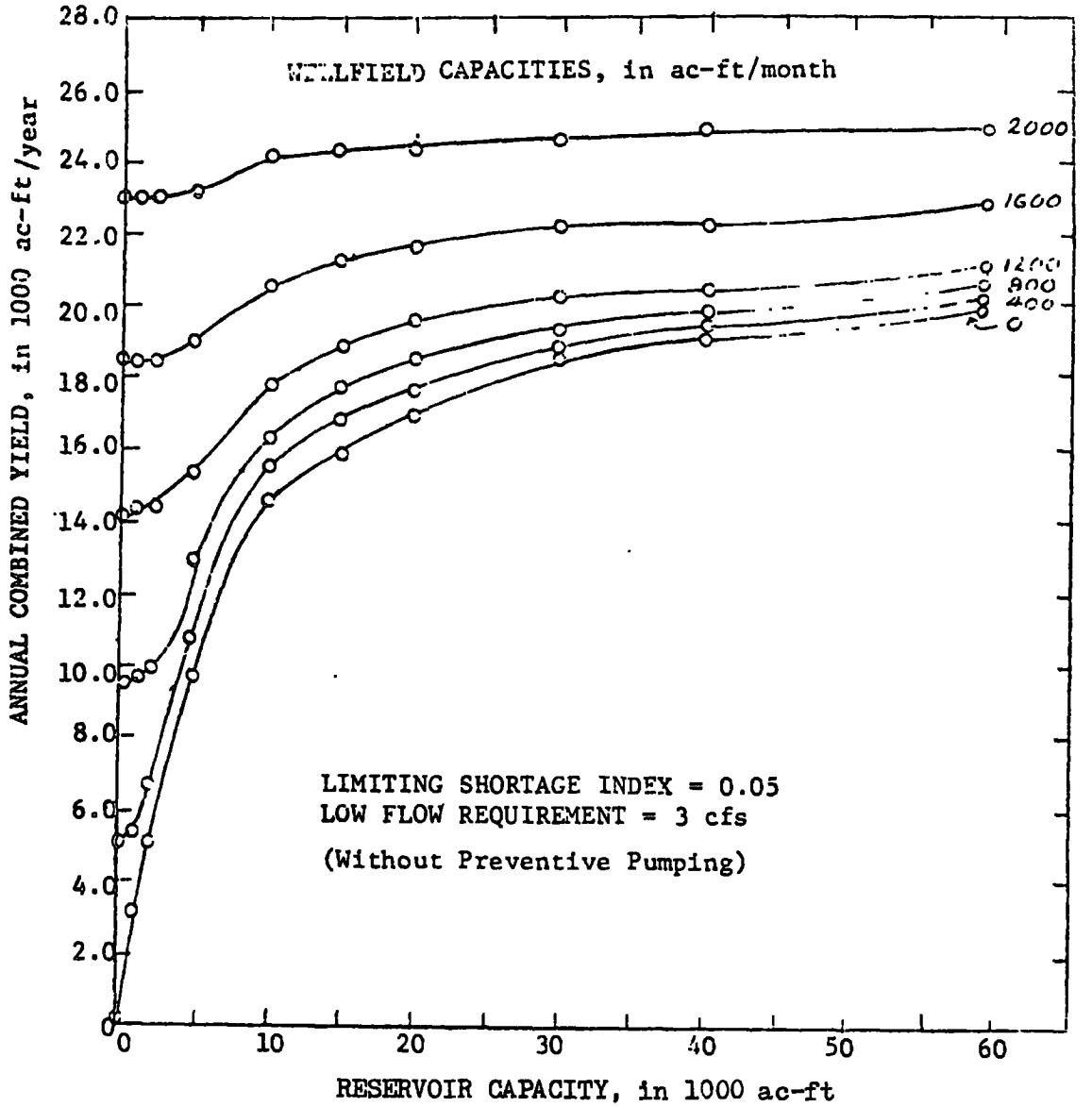


Figure 2 -- Annual Combined Yield for a Stream and an Aquifer Used as Back-Up Source

8. Preventive Pumping to Forestall Water Supply Shortages

Prudent water supply management would suggest beginning to pump ground water several months ahead of potential supply shortages, and thus providing for a carry-over storage in the reservoir which, in conjunction with a moderate but steady ground water supply, will carry the utility over the drought period.

Using the routing year beginning May 1, a target carry-over storage can be computed for every month of the relatively dry season between June (month 2) and December (month 8) by the equation:

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

where: CS_i = target carry-over storage at the beginning of month i

D_n = target draft during month n, including low flow releases

EXQ_n = expected reservoir inflow during month n

PMPC = monthly well field pump capacity

Whenever the water volume in a reservoir at the beginning of a month is less than the target storage, the decision should be made to supply the difference by ground water pumping, up to the limiting pump capacity. The expected reservoir inflow is the variable which determines the degree of back-up service the ground water source will provide. The <u>degree of preventive pumping</u> is defined as the ratio between long-term average and expected reservoir inflows. Comparative curves showing the relationship between total yield or average pumping volume and the degree of preventive pumping indicate that the combined yields or the pumping volume increase

drastically as the degree of preventive pumping is raised from 1 to 5, but level off after that point. Figure 3 shows the advantage of preventive pumping over the use of the aquifer merely as a back-up source. Each increment in pumping capacity adds substantially to the combined yield. The yield when maximum capacity (2000 acre-feet/month) is used equals 25,000 acre-feet/year under the back-up rule, versus over 29,000 acre-feet/year under the preventive pumping rule.

74-3: p. 4-22 to 4-27.

9. Yield from Aquifer as Exclusive Source

If well field pumping capacity has been sized according to sound engineering design practice to supply the maximum day demand, and if annual withdrawals do not exceed average annual aquifer replenishment, month-by-month routing of stochastic inflows and scheduled releases is not deemed necessary. The design yield equals the average daily demand times the ratio, RMA, of maximum to average day demand. When RMA is assigned a value of 2.2, the annual aquifer design yield can be expressed as:

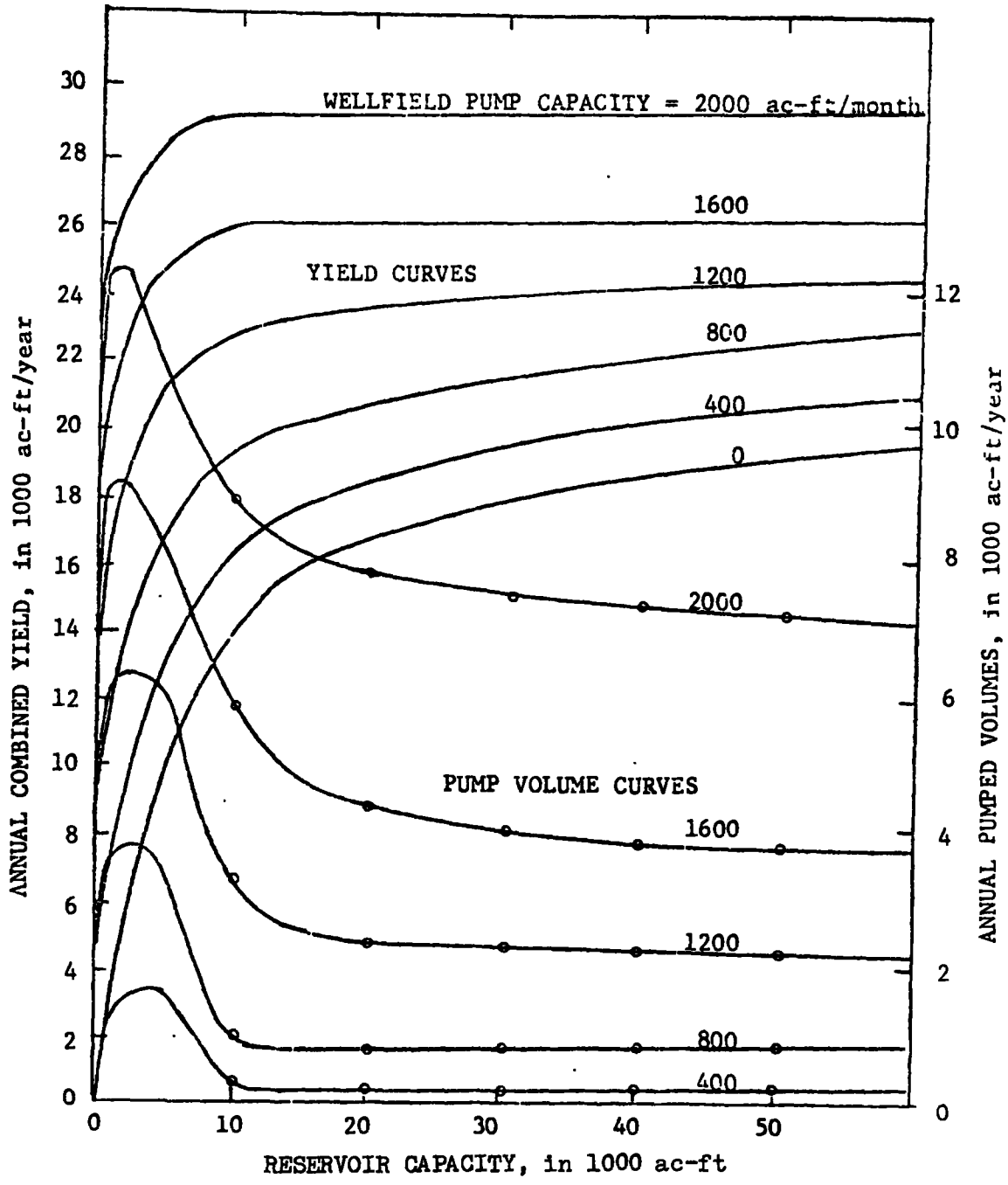
$$AY = \frac{12\ PMPC}{RMA} = 5.45\ PMPC$$

where: AY = annual aquifer design yield

PMPC = monthly well field pump capacity

RMA = ratio of maximum to average day demand

If the annual demand rate approaches the average annual aquifer replenishment rate, a routing procedure should be used with all surface components set to zero. Where a possible critical depletion of aquifer reserves becomes a threat, it should also be attempted to determine a



Conditions: Limiting Shortage Index = 0.05
Low Flow Requirement = 3 cfs
Degree of Preventive Pumping = 5.0

Figure 3 -- Annual Combined Yield and Ground Water Volume Pumped for a Reservoir and an Aquifer Under the Preventive Pumping Rule

relationship between annual rainfall and replenishment rates as well as average water table elevation. The relation is difficult to estimate without an excellent data base. If such detailed data are unavailable, the use of the average annual recharge plus the safety factor associated with the limitation of a maximum permissible draw down should be adequate.

74-3: p. 4-26 to 4-28.

10. Surface and Ground Water Supply Integration

The effectiveness of integrated versus separate water supply operation can be compared, first, under the back-up rule according to which the aquifer is considered merely a standby source and is called into operation only on the verge of a water shortage.

Let the yield of a storage reservoir ranging in capacity from 5,000 to 40,000 acre-feet, operated by itself, be compared with the yield of the reservoir when operated in combination with a standby well field ranging in pumping capacity from zero to 2,000 acre-feet/month. The combination of small reservoirs with large well field pumping capacities is the most productive in annual yield. For large reservoirs and small pumping capacities, annual yields are higher if the sources are operated individually. This is because, under the back-up rule, the aquifer is pumped at a much lower load factor than the RMA, the load factor of the separately operated well field.

A different result is obtained if the comparison is made under the preventive pumping rule. No matter how large the reservoir or how small the pumping capability, integrated operation provides a larger yield than

separate operation. Conversely, given a required yield, the integrated system can provide it with lower design capacities of both the reservoir and the well field pumps.

The preventive pumping rule permits the lowest total capital investment. However, if well field costs are a major factor and/or total water sources are scarce, it is wiser 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 reduces the well field's load factor and consequently imposes larger investment costs.

74-3: p. 4-28 to 4-32.

11. General Description of Supply Component Quantification and Cost Calculation Procedures

Design flow schedules consist of year-by-year schedules of maximum-day demands and annual schedules of supply volumes. Maximum-day demands specify the daily capacity a water utility must have in any one year during the planning horizon. They also pinpoint future dates when supply must be augmented under an optimal stage construction scheme.

Design schedules of supply volumes specify the flow quantities of each supply source during each month of the planning horizon. They are needed in computing O + M costs.

The total cost for each water supply source alternative is the sum of four major cost items: Reservoirs; surface water transportation; aquifer development including pumping and ground water transportation;

and raw water treatment. The boundaries of each of these cost items and the components within the boundaries should be defined so that design and economic considerations are quasi-independent of those in other cost items. For example, within the item of surface water transportation, the design capacity and pumping station cost are related to the diameter of the conveying pipeline. The least-cost overall combination of pipeline and pumps involves trade-offs between these two components.

Generalized planning equations and assumptions for determining design capacities and costs are needed to minimize system costs. If a computer program is used, it makes possible a detailed sensitivity analysis.

74-3: p. 5-1 to 5-3.

12. Sensitivity Study of Integrated Surface and Ground Water Use

The goal of a sensitivity analysis is to investigate the conditions under which integrated use schemes are more advantageous than single source developments. Both aquifers and reservoirs have the potential to satisfy total water requirements for any use; the feasibility of integrated use depends on their relative economic competitiveness.

A large number of factors affect the cost competitiveness of aquifers and surface water sources. Among these, water quality is particularly important. Four water quality combinations are here selected for analysis:

(1) Surface water needs full treatment, ground water requires chlorination only.

(2) Surface water needs full treatment, ground water requires hardness removal and chlorination.

(3) Surface water needs chlorination only, ground water requires hardness removal and chlorination.

(4) Surface and ground water require the same degree of treatment.

With Water Quality Combination (1), initial aquifer development is the most economical alternative. But, as the distance from well field to the treatment plant approaches 26 miles, the cost of the well water approaches that of the exclusive surface supply. The cost of alternatives involving integrated use schemes also increases with distance from the well field, but at a reduced rate because of the lower relative use of ground water.

When initial water table depth is related to well field distance, an increase of 100 feet in water table elevation is shown to be equivalent in its cost effects to a 0.9 mile increase in well field distance. As for the discount rate, a change of 1%, from 4% to 5% for example, decreases the present worth of costs for well water by about 10%. Conditions for integrated use also become more favorable with higher discount rates. Smaller well field pumping capabilities are called for, but the time of supplementation by a reservoir is moved closer to the present.

With Water Quality Combination (2), when the hardness of evenly blended surface (e.g., 40 ppm) and ground water (200 ppm) is below the maximum permissible standard (120 ppm), and when the distance of reservoir and well field to treatment plant is identical, integrated operation is by far the most advantageous water supply scheme. The reason is that, while softening would be required were the aquifer used as the sole source, and turbidity removal treatment were surface water the only source, integrated use makes both softening and turbidity control unnecessary. The initial surface-to-

ground water mixing ratio will be higher than one-to-one, but will gradually decrease to that proportion as new wells are added. Under these conditions, integrated use is the least-cost solution until the distance of reservoir to treatment plant exceeds 23 miles, beyond which exclusive softened ground water use becomes cheaper. When, instead, the aquifer approaches a distance of 18 miles from the treatment plant, the exclusive use of surface water becomes more attractive.

As the ground water hardness decreases, the mixing ratio of surface to ground water use will decrease until the integrated operation merges with exclusive aquifer use. As the ground water hardness increases beyond 200 ppm, either large mixing ratios or softening at $75/ppm/mgd is required, which adds substantially to costs. The result is a shift toward increased use of surface water until it becomes the exclusive use.

With Water Quality Combination (3), integrated use is likewise recommended. The optimal relative contribution of surface water varies with the competitive position of the two sources. Its lower limit is set by the surface-to-ground water blending ratio made necessary by the hardness of well water.

With Water Quality Combination (4), in which treatment costs of the two sources cancel one another, so long as the distances of the two sources are not vastly different, integrated use tends to be more advantageous than single source use.

Under constraint conditions, where neither the surface water nor the ground water source is able to meet total water supply requirements during the entire span of the planning horizon, integrated use becomes a necessity.

Most effective water conservation has a higher priority than least cost. Therefore, an integrated water use scheme relying primarily on surface water will effectively obviate mining of the aquifer.

The above conclusions are based on a number of assumptions which vary substantially with location and requirements. They show, however, that the planner will find it advantageous to explore the possibility of integrated ground and surface water use whenever this is feasible.

74-3: p. 7-1 to 7-23.

III

WATER SUPPLY COSTS

III. WATER SUPPLY COSTS

Source:

See Section I

IWR Contract Report 74-3 February 1974

Scope and Use

The justification, objective, and scope of IWR Contract Report 74-3 were presented at the head of Section I.

The costs shown in Section III are extracted from Appendix A of the contract report. They were utilized by the authors in developing a rationale for integrated use of surface and ground water sources. The costs had to be gathered within the budgetary limitations of the project.

Cost indexes were selected as follows: For construction, and operation and maintenance of surface reservoirs, the Engineering News Record Construction Cost Index (ENR-CCI) was thought to be the most representative; investment and O + M costs are adjusted to a common base of 1727, which corresponds to a point in time of mid-year 1972. For all other cost centers, the Engineering News Record Building Cost Index (ENR-BCI) was believed best applicable; construction and O + M costs are adjusted to a common base of 1039, related to the same point in time. For all cost centers, the ratio of the 1972 index to the index existing when the costs were developed is the factor used in the equations to raise the costs to 1972 cost levels. This ratio, called the "index ratio" (IR), is given for each cost equation presented in Section III. By the same token, the cost equations can be brought up-to-date for a reasonable period into the future by dividing the index ratio (IR) by the common base (1727 or 1039, respectively), and multiplying it by the current Engineering News Record Construction Cost Index or Building Cost Index, respectively.

The construction cost equations do not include the contractor's profit, engineering fees, or contingency allowances. These are commonly added at the rate of 10% of the construction or building costs. With regard to the constants shown in all equations, their source is not given in Appendix A.

The equations, say the authors, "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." They may well be utilized by planners who seek first approximations.

Another set of construction and O + M costs for water supplies, as well as for waste treatment facilities, is presented in Section VII (Planning for Water Quality). See the head of that section for qualifications.

III. WATER SUPPLY COSTS

1a. Construction Costs for Surface Reservoir

The costs shown below apply to an earth fill reservoir, and encompass: Land clearing, spillway construction, relocations, engineering services, contingencies, and land.

$$C_r = (IR)(9{,}160)(V_r)^{0.54} + (0.49)(L)(V_r)^{0.87}$$

where: C_r = construction costs of reservoir in dollars

IR = index ratio equal to 1.84

V_r = reservoir volume in acre-feet

L = cost of land at $500/acre

Estimated life expectancy: 50 years.

1b. Operation and Maintenance Costs for Surface Reservoir

O + M costs for reservoir are a function of capacity.

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

where: C_{or} = O + M costs for reservoir in dollars per year

V_r = reservoir volume in acre-feet

IR = index ratio equal to 1.98

2. Construction Costs for Intake Structure

For large lakes, reservoirs, and rivers, the cost of an intake is 2 to 5 times greater than that of a similar pipeline of the same length. The factor of 5 may be used, and the diameter of the intake may be assumed to be one commercial pipe size larger than the transmission line. Life expectancy is estimated at 50 years.

3a. Construction Costs for Transmission Line

Construction costs per mile of pipeline are a function of pipe diameter.

$$C_{pl} = (IR)(2,160)(D_p)^{1.2}$$

where: C_{pl} = construction cost of pipeline in dollars per mile

IR = index ratio equal to 1.70

D_p = nominal diameter of pipeline in inches

Estimated life expectancy: 50 years

The right-of-way cost, estimated at $569 per mile, should be added to the value of C_{pl}.

3b. Operation and Maintenance Costs for Transmission Line

Pipeline O + M costs are directly proportional to pipeline construction costs excluding the right-of-way.

$$C_{opl} = (0.0025)(C_{pl})$$

where: C_{opl} = O + M costs of pipeline in dollars/year

The index ratio is not included, since the cost level is adjusted in the construction cost equation.

4a. Construction Costs for Pumping Station

Included are costs of pump and building.

For volumes of 0.2 to 2.0 mgd:

$$C_{ps} = (IR)(HP)(0.29)(Q_t)^{-0.50}$$

For volumes of 2.0 to 200 mgd:

$$C_{ps} = (IR)(HP)(4.19)(Q_t)^{-0.12}$$

where: C_{ps} = construction costs of pumping station in dollars

IR = index ratio equal to 1.70

HP = installed horsepower

Q_t = volume in gpm

Estimated life expectancy: 25 years.

4b. Operation and Maintenance Costs for Pumping Station

Costs apply to pumping station ranging between 150 and 15,000 HP.

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

where: C_{ops} = annual O + M costs per pump station, in dollars

IR = index ratio equal to 1.79

Q_t = volume in cfs

H = maximum expected pumping head in feet

T = annual hours of plant operation equivalent to hours at full design capacity

A = age of plant in years

5. Electrical Power Costs

Power costs are a step function of both installed HP and the amount of power used.

$$C_{pw} = (IR)(1.0525)(215 + (1.61)(KWD - 100) + 190 + (0.007)(KWH - 20,000)) + (IR)(0.00023)(KWD)$$

where: C_{pw} = cost of electricity per year

IR = index ratio equal to 1.00

KWD = kilowatt demand installed

KWH = kilowatt hours per month of energy usage

6a. Construction Costs for Treatment Plant

The following cost formula is borrowed from the construction of waste water treatment plants, and may not be accurate.

$$C_b = (IR)(40,000)(Q_{dc})^{0.70}$$

where: C_b = construction cost of treatment plant in dollars

IR = index ratio equal to 1.88

Q_{dc} = design capacity in mgd

Estimated life expectancy: 50 years.

6b. Operation and Maintenance Costs for Treatment Plant

The cost of chlorination at the rate of 28.8 lbs of chlorine per mg is included. The cost formula is not applicable where chlorination is the only water treatment process.

$$C_{og} = (0.02)(C_b) + (IR)(28.8)(0.13)(VOL)$$

where: C_{og} = annual general O + M costs for water treatment plant in dollars

C_b = construction cost of treatment plant in dollars

IR = index ratio equal to 1.24

VOL = amount of water processed in mg

7a. Construction and Equipment Costs for Chlorination System

Costs are hypothesized.

$$C_c = (IR)(8,700)(Q_{dc})^{0.6}$$

where: C_c = construction costs in dollars

IR = index ratio equal to 1.00

Q_{dc} = design capacity in mgd

Estimated life expectancy: 50 years.

7b. Operation and Maintenance Costs for Chlorination System

O + M costs for building and equipment are 2% per year of original construction cost; labor cost is 50 cents per mg; chemicals consist of 28.8 lbs of chlorine per mg.

$$C_{oc} = (IR)((28.8)(0.13)(VOL) + (0.50)(VOL)) + (0.02)(C_c)$$

where: C_{oc} = annual chlorination system O + M cost in dollars

IR = index ratio equal to 1.24

VOL = volume of water treated in mg

C_c = construction cost in dollars

8a. Construction Costs for Coagulation, Flocculation, and Rapid Sand Filtration

$$C_{cf} = (IR)(330,000)(Q_{dc})^{0.678}$$

where: C_{cf} = construction cost of the coagulation-filtration equipment in dollars

IR = index ratio equal to 1.62

Q_{dc} = design capacity in mgd

Estimated life expectancy: 50 years.

8b. Operation and Maintenance Costs for Coagulation, Flocculation, and Rapid Sand Filtration

Costs depend on average yearly turbidity and average yearly flow rate.

$$C_{ocf} = (IR)(365)(COE)(Q_{ay})^{0.620}$$

where: C_{ocf} = O + M cost in dollars per year

IR = index ratio equal to 1.62

COE = coefficient related to average annual turbidity (see Table below)

Q_{ay} = average yearly flow rate in mgd

Coefficients (COE) for Above Cost Formula

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

9a. Construction Costs for Treated Water Storage Reservoir

Costs are for a conventional buried concrete storage reservoir.

$$C_{bc} = (IR)(67,000)(VOL)^{0.606}$$

where: C_{bc} = construction cost of buried concrete reservoir in dollars

IR = index ratio equal to 2.03

VOL = storage capacity in mg

Estimated life expectancy: 50 years.

9b. Operation and Maintenance Costs for Treated Water Storage Reservoir

O + M costs are a function of capacity.

$$C_{obc} = (IR)(860)(VOL)^{0.211}$$

where: C_{obc} = O + M cost for the treated water storage reservoir

IR = index ratio equal to 2.03

VOL = storage capacity in mg

10a. Construction Costs for Well

Well construction costs are a function of depth, bore diameter, geological formation, and expected flow from well. Included in the formula are a test hole costing $2,000 and a well house costing $2,500.

$$C_w = (IR)(COE)(D_w)^{EXP} + 4,500$$

where: C_w = well construction cost in dollars

IR = index ratio equal to 1.60

COE = coefficient whose value is a function of well diameter and geologic formation. See Table below.

D_w = depth of well in feet

EXP = exponent whose value is a function of well diameter and geologic formation. See Table below.

Estimated life expectancy: 40 years

The well diameter is a function of the pumping rate.

Coefficients (COE) and Exponents (EXP) for Well Costs

Geologic Formation	Pumping Rate (gpm)	Bore Hole Diam. (in)	COE	EXP
Finished tubular wells in sand and gravel	140- 700	6 - 10	800	0.299
	1,400- 2,100	12 - 16	850	0.373
Finished gravel-packed wells in sand and gravel	2,100- 3,500	16 - 20	680	0.408
	7,000-17,500	24 - 34	680	0.482
	21,000-31,500	36 - 42	890	0.583
Shallow sandstone, limestone, or Dolomite bedrock wells	0- 140	6	0.578	1.413
	350- 1,400	8 - 12	0.839	1.450
	2,100- 7,000	16 - 24	1.781	1.471
Deep sandstone wells	350- 1,400	8 - 12	0.029	1.870
	2,100- 3,500	16 - 20	1.314	1.429

11a. Installation Costs for Well Pump

Installed vertical turbine pump costs are a function of the maximum pumping head and the maximum flow rate.

$$C_{pu} = (IR)(7.31)(Q)^{0.453} \quad (H)^{0.642}$$

where: C_{pu} = cost of installed pump in dollars

IR = index ratio equal to 1.60

Q = pump capacity in gpm

H = maximum expected pumping head in feet

Estimated life expectancy: 20 years.

12a. Installation Costs for Booster Pump

Close-couple booster pumps are installed in the main pipeline where pumping heads exceed 700 feet.

$$C_{bp} = (IR)((67.8)(Q_t) - (4.04)(Q_t)^2 + (0.123)(Q_t)^3)(H_e)$$

where: C_{bp} = cost of installed booster pump in dollars

IR = index ratio equal to 1.70

Q_t = maximum discharge in gpm

H_e = pumping head in excess of 700 feet

Estimated life expectancy: 20 years.

10b, 11b, 12b. Operation and Maintenance Costs for Well Field

Well, well pump, and booster pump O + M costs are lumped together here; they include labor and vehicles, but exclude electric power costs (see item 5).

$$C_{wf} = (IR)(7.6)(VOL)$$

where: C_{wf} = well field O + M costs in dollars per year

IR = index ratio equal to 1.70

VOL = volume pumped per year in mg

13a. Construction Costs for Lime-Soda Softening

Costs exclude the water treatment plant building.

For volumes between 1 and 10 mgd:

$$C_{sf} = (IR)(310,000)(Q_{dc})^{0.550}$$

For volumes in excess of 10 mgd:

$$C_{sf} = (IR)(151,000)(Q_{dc})^{0.862}$$

where: C_{sf} = construction costs of the equipment used for lime-soda softening, in dollars

IR = index ratio equal to 1.62

Q_{dc} = design capacity in mgd

Estimated life expectancy: 50 years.

13b. Operation and Maintenance Costs for Lime-Soda Softening

O + M costs are a function of hardness removed and the average yearly flow rate.

$$C_{osf} = (IR)(365)(COE)(Q_{ay})^{EXP}$$

where: C_{osf} = yearly O + M costs in dollars

IR = index ratio equal to 1.62

COE = coefficient varying with hardness reduction and flow rate (see Table below)

Q_{ay} = average yearly flow rate in mgd

EXP = exponent varying with hardness reduction and flow rate (see Table below)

Coefficients (COE) and Exponents (EXP) for Lime-Soda Softening O + M Cost Equation

Hardness Reduction (mg/l)	Average Yearly Flow Rates (mgd)	COE	EXP
300	1 - 10	174.0	0.632
300	10 - 100	87.0	0.919
200	1 - 10	162.0	0.585
200	10 - 100	71.4	0.919
100	1 - 10	147.0	0.530
100	10 - 100	53.0	0.919

74-3: p. A-1 to A-18.

IV

THE RISK OF WATER SUPPLY SHORTAGES

IV. THE RISK OF WATER SUPPLY SHORTAGES

Source:

A Methodology for Assessing Economic Risk of Water Supply Shortages, by G.K. Young, R.S. Taylor, and J.J. Hanks, Water Resources Engineers, Inc., Springfield, Va.

IWR Contract Report 72-6 May 1972

Scope and Use

The project was devoted to developing a procedure for estimating the frequency-damage relationship arising from a water shortage in an urban area. This implied estimating income losses associated with varying degrees of water shortage which, when combined with the frequency of shortage, resulted in the frequency-damage relationship.

An empirical test of the procedure was developed for the York, Pa. Water Service Area.

The concepts and case study may be valuable to planners working on urban water supply problems, as they show an alternative method of assessing the benefits of adequate municipal water supplies. They distinguish between short-run and long-run plans, and determine the role which each type plays in resource development planning.

IV. THE RISK OF WATER SUPPLY SHORTAGES

1. Methodology

A water supply shortage may be defined as prevailing at any time the water purveyor chooses, or is forced into, a position where not all of the total demand for water is met. Some events which may lead to a shortage are:

1. Inadequacy of the present water supply.
2. Population and industrial growth.
3. Increases in water use resulting from a larger per capita demand.
4. Irregular periods of meteorological drought or seasonal deficiencies in supply.
5. Inability to meet peaks in the demand, including seasonal, monthly, weekly, daily, or even hourly drafts.
6. Additional hook-ups to the system.
7. Hedging during a period when water is on hand, but forecasts indicate insufficient supply from principal source.
8. The water manager's operating policies.
9. Improper setting of water rates.

A water supply shortage generally results in:

1. Short-term economic losses to customers and the water supplier.
2. The need for long-term system improvements to meet future demand.

Water supply shortages and their economic impacts can be reproduced by a computer simulation model. Basic elements in the model are a stream, which furnishes the main water supply to the community; a reservoir; a water company

manager who runs the system; and alternate water supply sources such as rivers and quarries. Data for the simulation consist of projected population trends over the period considered; the water demand expected; the hydrology of the supply stream; the reservoir storage; and information on system management.

Risk analysis as it applies to water supply shortages involves the following steps. Consider a set of drought situations of variable severity and span. Each drought situation has an annual probability of occurrence and an economic loss associated with it. Risk is the expected value of the loss, and equals the sum of the products of the probabilities and respective losses. Risk avoidance measures the benefits associated with a reduction in supply fluctuations.

Demands are assumed given. If demands are not satisfied, losses are assumed to occur. The risk of such losses is their expected value. In a surface water system, storage reservoirs provide the reserves which mitigate shortages. As the amount of storage in a surface system increases, the risks decrease. On the other hand, the level of economic activity is reflected in water demands. As water requirements increase, economic losses associated with not meeting the requirements increase. Thus, as demands increase, so do risks.

Risk measurement is predicated on expected economic losses incident to hydrological behavior of the water supply, demand-restrictive measures, and supply-expansive actions of the water manager.

Hydrology implies the estimation of daily streamflow inputs to the supply system. In the eastern United States, the June through September period is normally the period with lowest streamflows. The lowest mean flow of record for the June through September period is selected. Daily streamflow data for that period are related to means having known probabilities.

Restriction of the demand, during water shortages, is decreed by the water manager. A five-step cut-back scheme based on logic, experience, and a sense of values is as follows:

Directed at All Users

1. Voluntary cut-backs accomplished by public appeals.
2. Mandatory cut-backs eliminating the outdoor use of hose.

Directed at Industrial and Commercial Users

3. Mandatory 50% cut-back of water used inside each establishment.
4. Mandatory shut-down of all industrial and commercial usage.

Directed at Domestic Users

5. Mandatory severe (but not total) restrictions of indoor water use.

The policies of the water manager regulate the system and dominate the entire day-by-day analysis. The manager monitors streamflow and reserves, makes supply allocations, and can also purchase water from other sources.

The combined effects of hydrology, demand restrictions, and actions of the water manager cause economic losses to occur during a shortage. The daily impact of economic losses is determined for four sectors of the area: the municipal government, the commercial, the industrial, and the domestic sector. The municipal sector may incur a loss of revenue from water sales plus the

cost of water imports; the commercial sector may have stock losses, such as plants in a nursery; the industrial sector may lose profits and incur fixed costs during a shut-down; and the domestic sector may receive a reduced payroll and lose consumer surplus.

The simulation model is used to determine the following items:

1. As daily streamflows are routed through the demand sectors in accordance with the manager's decision schedule, and as the water supply becomes critical, compute daily water allocations resulting from demand restrictions and the tapping of emergency sources.

2. Compute the aggregate area loss and risk from losses incurred by individual sectors of water users.

To generate these results, inputs to the model must include:

1. The physical characteristics of the reservoir and distribution system.

2. The water demand of each sector as a function of time.

3. The cost of emergency water supply.

4. The water manager's decision schedule.

5. Domestic price-demand functions.

6. Industrial loss functions.

72-6, p. 1-2 to 3-16.

2. Loss Functions

The boundaries of the water company's service district serve to circumscribe the area within which economic losses are to be estimated. Economic losses are sustained by one or more sectors of water customers:

1. Cost of importing water (municipal sector).
2. Loss of water company revenue (municipal sector).
3. Stock losses (mostly commercial sector).
4. Profit losses and unproductive fixed costs (industrial sector).
5. Payroll losses (domestic sector).
6. Consumer surplus losses (domestic sector).

Two cost alternatives are available to the water supplier considering the importation of emergency supplies. One is a low-capital-cost but high-operating-expense procedure, while the other has a high capital cost with fairly low operating expenses. The duration of the emergency being unknown, the logical sequence of these alternatives is as listed above. The first type includes the importation of water by truck or railroad car -- a strictly temporary arrangement; the second type, construction of a transmission pipeline. Truck or railroad conveyance costs are straight-forward.

Pipeline transmission costs, over and above the initial investment, are calculated from the equation:

$$C_T = 1.08 \left(\frac{1.66 \cdot 10^{-2} (L_1 S_1 + L_f S_f) P}{E} \right) ,$$

where:

C_T = transmission cost including maintenance ($/Kgal/mi)

E = efficiency factor (fraction)

L_1, L_f = loading factors (fraction of total capacity)

P = cost of energy ($/KWH)

S_1 = slope of the pipe (ft/1000 ft)

S_f = friction loss (ft/1000 ft)

An estimate of the friction losses is given by:

$$S_f = Q^{1.85} \cdot 10^3 / (405 \cdot 10^{-6} \cdot C \cdot D^{2.63})^{1.85}$$

where: C = Hazen-Williams coefficient

D = pipe diameter (inches)

Q = pipe flow (mgd)

The loss of water company revenue is easily ascertainable from records. Stock losses in the commercial sector are estimated by individual merchants.

Profit losses and unproductive fixed costs incurred by the industrial sector may be estimated on the basis of data from state statistics, from the Census of Manufactures for the particular state involved, U.S. Department of Commerce, Bureau of the Census, and from the Economic Report to the President. The following steps are required:

Each industrial high-water user is identified with its appropriate SIC classification; the average number of employees is obtained for each class; the estimated number of production workers is obtained by multiplying that number by the percent of production employees to all employees; the number of production workers is multiplied by the average annual wage per production worker to yield the estimated average annual production payroll; the latter is divided by the percent that the production payroll represents of the value of shipments to estimate the value of shipments; profits after taxes per dollar of sales are obtained from the Economic Report to the President; this percent is multiplied by the estimated value of shipments to determine estimated profits after taxes; estimated fixed costs are computed by multiplying the estimated value of shipments by the ratio of value added by manufacture (less production payroll) to value of shipments; profits and fixed costs can now be aggregated and divided by the number of production workers to yield profits plus fixed costs per production worker.

From interviews with company officials is obtained the anticipated production cut-back in case of water supply restrictions, such as for example a 33% production cut-back in case of a 50% water supply reduction. Thus, industry's economic loss can be related to various levels of restriction in water use.

In the domestic sector, payroll losses are much the most important. To estimate these losses, the average annual wage per production worker, determined for the industrial sector, is first reduced to a weekly wage and then multiplied by the number of production workers to equal the estimated average weekly production payroll. In case of water supply restrictions, payroll reductions may average 75% of production cut-backs, which themselves vary widely with water use restrictions. The loss in weekly production payrolls thus equals 75% of the average weekly production payroll multiplied by the anticipated production cut-back in percent.

Still in the domestic sector, another loss is the consumer surplus which disappears with the revenue lost to the water utility whenever a shortage occurs. Total utility can be obtained by integrating the demand curve to the equilibrium point. Total cost is the price at equilibrium times the volume used. The difference is the consumer surplus. When faced with a forced shortage, one can integrate the demand curve from the forced lower limit to the equilibrium point, and subtract the lost revenue to determine lost consumer surplus.

In developing the aggregate demand curve for residential water, the equations formulated by Howe and Linaweaver may be used. If an increase in consumption is prevalent in the summer months, these authors suggest

that the demand curve be disaggregated into two portions, the sprinkling demand and the domestic demand. The sprinkling demand curve is calculated by using Howe and Linaweaver's equation:

$$q_s = A \cdot b^{-0.793} (w_s - 0.6r_s)^{2.93} \; V^{1.45} \; p_s^{-1.57},$$

where:

A = coefficient to fit curve to specific area

b = irrigable area per dwelling unit (acres)

p_s = marginal charge for water (cents/Kgal)

q_s = average summer sprinkling demand (gal/day/dwelling unit)

V = market value of dwelling unit ($1000)

r_s = summer precipitation (inches)

w_s = summer potential evapotranspiration

The market value V of a dwelling unit may be calculated from data in the U.S. Census information for the area in question. V may be taken to be a weighted average house value, where the numbers averaged are the midpoints of the various value intervals reported in the census information, and the weights are the number of dwellings in each interval.

The domestic portion of the consumer surplus loss is a factor only in an extreme situation. The last use of water to be restricted is assumed to be the domestic demand. Howe and Linaweaver's work may again be used to determine the loss of domestic consumer surplus. The equation governing the price-demand relationship for domestic use is:

$$q_d = C + (3.47 \cdot V) - 1.30 \, p_s ,$$

where: q_d = domestic demand (gal/day/unit)

C = coefficient to fit area

V = market value of housing ($1000)

p_s = price of water (¢/Kgal)

The winter average monthly demand is assumed to represent the domestic demand. When all other values have been determined, the value of C can be computed. This yields the price-demand function. From it, and allowing a certain reduction in domestic demand, the consumer surplus may be calculated and expressed in ¢/day/unit. The simulation model uses the unit loss value, the number of dwelling units, and the total number of days the restriction is applicable, to determine the domestic loss.

Totaling the values of the lost consumer surplus due to sprinkling and domestic use restrictions yields the total consumer surplus lost as a result of drought restrictions.

72-6, p. 6-1 to 6-24.

3. Water Shortage Risk Determination

The simulation logic has a relatively simple structure. For a given reservoir storage, a set of hydrologic conditions with their associated probabilities are used to compute the daily supply available for municipal withdrawal. The demand is calculated in five-year increments for a fifty-year planning period. Next, a water balance is performed on the supply and demand for a given year for each hydrologic condition, and the reservoir releases required to meet the demand are tallied. The daily reservoir level is also calculated. When the reservoir reaches certain critical levels, a systems operating rule is instigated which either increases supply by importing

water, or decreases demands by imposing restrictions on water use. The economic portion of the simulation determines the regional dollar losses incurred in the study area as a result of the water supply importations or restrictions. Each hydrologic condition results in different losses. Each loss is multiplied by the probability of the associated hydrologic condition to determine the risk. The risks associated with the complete set of hydrologic events are accumulated to determine the total risk for the given year of a water supply shortage. The simulation repeats this process in five-year steps for the increasing demands until it reaches the year 2020.

In order to reduce the information gained from this technique to a single number, the model calculates the present worth of the total yearly risks over the fifty-year planning period. Each reservoir storage level has an associated present value of risk. The model then determines the cost of construction of a reservoir of the size selected. By allowing the simulation to consider various size reservoirs, a curve such as that shown in Figure 4 can be prepared. It should prove useful to a planner.

72-6: p. 7-1 to 7-5.

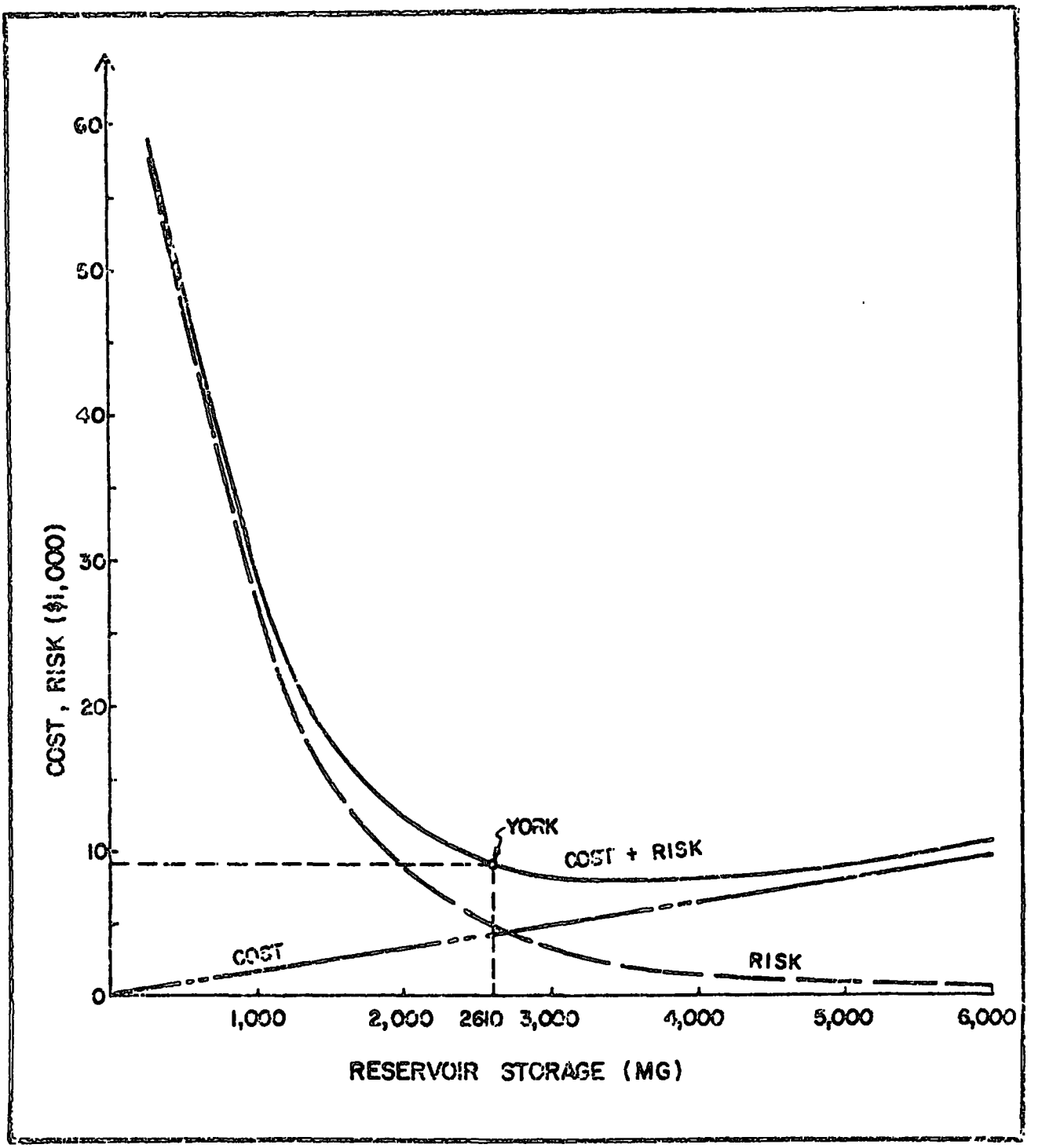


Figure 4. Present-Worth Analysis:
Total Yearly Risks Plus Reservoir Construction Cost

V

INDUSTRY's RESPONSE TO CHANGING WATER COSTS

V. INDUSTRY's RESPONSE TO CHANGING WATER COSTS

Source:

Changing Water Use in Selected Manufacturing Industries, by Robert A. Leone, J. Royce Ginn, and An-Loh Lin, National Bureau of Economic Research, New York, N.Y. 10016, and the Graduate School of Business Administration, Harvard University, Boston, Mass. 02163

IWR Contract Report 74-10 October 1974

Scope and Use

The objective of this report is to present findings of a study of the effects of changes in the price and quality of water, as well as in water pollution control regulations, on technological changes in process design and plant location decisions. The heart of the study is found in chapter IV, entitled "A Cross-Section Model of Industry Response to Water Costs." The purpose of that chapter is to specify demand equations for gross water and intake water, and then estimate them by using cross-section plant data for the paper, chemical, petroleum, and steel industries.

From chapter IV were extracted a series of equations the purpose of which is to compute:

(1) The level of gross water use
(2) The level of intake water use
(3) The relationship between the level of gross and intake water use
(4) The price elasticity of water.

This methodology for establishing various mathematical relations is applicable to problems involving industrial water supply and use/cost ratios. The authors concluded that the demand for industrial water is highly elastic to water cost changes. This may not be primarily because water

cost is such a major item of expense, but because it is so easy to recycle industrial water and recycling is long overdue.

With regard to the particular application of the methodology to the high-water-using industries, the following warnings seem appropriate: "The data provided by these applications are subject to great error. Even after obviously erroneous values have been eliminated, it is noted that two-thirds of the observations will have serious errors or omissions. Roughly half of these errors can be corrected by seeking the original application, while the other half originates with the plants themselves. We can only suggest, therefore, that analysts make use of this information only after extremely close scrutiny, and that it be avoided as a data source for general analytical purposes."

The main contribution here to the planner is the methodology rather than its particular application to the high-water-using industries. For that reason also, the statistics used in the report were omitted from Section V.

V. INDUSTRY's RESPONSE TO CHANGING WATER COSTS

The purpose of the model is to evaluate industry's demand for gross water and intake water by means of plant data for the paper, chemical, petroleum, and steel industries.

1. The Theoretical Framework

The causes of changes in the water-output ratio and the intake-gross water ratio may be studied by conceptually treating intake water as an input to the production of gross water, and gross water as an input to the production of final output. Dealing first with the demand for gross water, consider a plant whose technology of producing final output (Q) is characterized by a Cobb-Douglas production function. The inputs used include gross water (GW), materials (M), labor (L), and capital (K). The production function is given by

$$Q = A\,(e^{mt})\,(GW^{a_1})\,(M_Q^{a_2})\,(L_Q^{a_3})\,(K_Q^{a_4}) \qquad (1)$$

where A is a scaler, m stands for the proportional rate of disembodied technical progress, and t is time. The sum of the coefficients a_1, a_2, a_3, and a_4 measures the returns to scale.

Total production cost equals the sum of the quantities of inputs multiplied by their respective prices:

$$C_Q = (pGW) + (gM_Q) + (wL_Q) + (rK_Q) \qquad (2)$$

where g is the unit price of materials, w the wage rate, and r the gross rate of return to capital. The price of gross water p is equal to the average cost of materials, labor, and capital incurred in the production of the final output for withdrawing intake water, recirculating water, and discharging effluent water.

Minimization of total cost with respect to all inputs subject to the production function gives the least-cost condition:

$$u = \frac{\left(\frac{P}{a_1 Q}\right)}{GW} = \frac{\left(\frac{g}{a_2 Q}\right)}{M_Q} = \frac{\left(\frac{W}{a_3 Q}\right)}{L_Q} = \frac{\left(\frac{r}{a_4 Q}\right)}{K_Q} \qquad (3)$$

where u is the total derivation of cost with respect to output or the marginal cost of output. Equations (3) and (1), in turn, yield the level of gross water use:

$$GW = \left(\frac{Q}{Ae^{mt}} \left(\frac{a_1}{a_2} \cdot \frac{g}{p}\right)^{a_2} \left(\frac{a_1}{a_3} \cdot \frac{w}{p}\right)^{a_3} \left(\frac{a_1}{a_4} \cdot \frac{r}{p}\right)^{a_4} \right)^{\left(\frac{1}{a_1+a_2+a_3+a_4}\right)} \qquad (4)$$

The quantity of gross water demanded thus depends negatively on its own price and on technical progress. It depends positively on the prices of other inputs and on the output of the final product.

Dividing both sides of (4) by Q, we obtain:

$$\frac{GW}{Q} = \left(\frac{Q^s}{Ae^{mt}} \left(\frac{a_1}{a_2} \cdot \frac{g}{p}\right)^{a_2} \left(\frac{a_1}{a_3} \cdot \frac{w}{p}\right)^{a_3} \left(\frac{a_1}{a_4} \cdot \frac{r}{p}\right)^{a_4} \right)^{\left(\frac{1}{a_1+a_2+a_3+a_4}\right)} \qquad (5)$$

where $s = 1 - a_1 - a_2 - a_3 - a_4$. Thus, when there are increasing or decreasing returns to scale, the output of the final product will also affect the productivity of gross water. If there are constant returns to scale, the output variable will drop out and will not be a determinant of gross water per unit of output.

In similar fashion, the demand for intake water can be derived from the demand for gross water. Assume that the production function for gross water is:

$$GW = B\,(e^{nt})\,(IW^{b_1})\,(M_R^{\,b_2})\,(L_R^{\,b_3})\,(K_R^{\,b_4}) \qquad (6)$$

where B is a scaler, n stands for the proportional rate of disembodied technical progress, and $b_1+b_2+b_3+b_4$ measures the returns to scale for the production of water.

Let total water cost be:

$$C_w = (q \cdot IW) + (h \cdot M_R) + (w \cdot L_R) + (r \cdot K_R) \qquad (7)$$

where h, w, and r are unit prices of materials, labor, and capital used for treating recycled water. Since water withdrawn and not consumed or lost in the production processes will be eventually discharged, the price of intake water must include the price of discharging effluent water. Thus, q is defined as the average cost of withdrawing and discharging intake water.

Minimizing (7) subject to (6), we obtain:

$$v = \frac{\left(\frac{q}{b_1 GW}\right)}{IW} = \frac{\left(\frac{h}{b_2 GW}\right)}{M_R} = \frac{\left(\frac{w}{b_3 GW}\right)}{L_R} = \frac{\left(\frac{r}{b_4 GW}\right)}{K_R} \qquad (8)$$

where v is the marginal cost of gross water. The level of intake water use is thus given by:

$$IW = \left(\frac{GW}{Be^{nt}} \left(\frac{b_1}{b_2} \cdot \frac{h}{q}\right)^{b_2} \left(\frac{b_1}{b_3} \cdot \frac{w}{q}\right)^{b_3} \left(\frac{b_1}{b_4} \cdot \frac{r}{q}\right)^{b_4} \right)^{\left(\frac{1}{b_1+b_2+b_3+b_4}\right)} \qquad (9)$$

The quantity demanded for intake water therefore depends negatively on the price of intake water and on technical progress in connection with reuse of water. It depends positively on the prices of other inputs and on gross water required.

Dividing both sides of (9) by GW, we obtain:

$$\frac{IW}{GW} = \left(\frac{GW^{s'}}{Be^{nt}} \left(\frac{b_1}{b_2} \cdot \frac{h}{q}\right)^{b_2} \left(\frac{b_1}{b_3} \cdot \frac{w}{q}\right)^{b_3} \left(\frac{b_1}{b_4} \cdot \frac{r}{q}\right)^{b_4} \right)^{\left(\frac{1}{b_1+b_2+b_3+b_4}\right)} \qquad (10)$$

where $s' = 1 - b_1 - b_2 - b_3 - b_4$. It is clear that gross water will affect intake water per unit of gross water unless there are constant returns to scale in the production of gross water.

Making use of (7) and (3), the price or average cost of gross water will be given by:

$$p = \frac{C_w}{GW} = v\,(b_1 + b_2 + b_3 + b_4) \qquad (11)$$

where v is the marginal cost of gross water. Let MPP be the marginal physical product of intake water, and from (8) MPP $= b_1 GW/IW$ in equilibrium.

Then, (11) can be written as:

$$p = \frac{q}{MPP} (b_1 + b_2 + b_3 + b_4) \tag{12}$$

The price of gross water will therefore be equal to the price of intake water in equilibrium if $(b_1 + b_2 + b_3 + b_4)/MPP = 1$, for example when there are constant returns to scale and MPP = 1.

These results can be summarized as follows:

$$\frac{IW}{Q} = \frac{GW}{Q} \cdot \frac{IW}{GW} \tag{13}$$

$$\frac{GW}{Q} = G\left(Q, \frac{g}{p}, \frac{w}{p}, \frac{r}{p}, \frac{1}{\hat{T}}\right) \tag{14}$$

$$\frac{IW}{GW} = I\left(GW, \frac{h}{q}, \frac{w}{q}, \frac{r}{q}, \frac{1}{\hat{T}}\right) \tag{15}$$

where T $(= e^{nt})$ accounts for technical change and has a reducing effect. The relative prices of inputs have positive effects. The effect of final output or gross water will be negative (positive) if the returns to scale are increasing (decreasing), and will be zero if there are constant returns to scale.

To complete the model, we assume that the output of the final product (Q) is determined under the condition of profit maximization, i.e., the price of output being equal to its marginal cost. From this condition and relations (1), (2), and (3), we obtain:

$$Q = Q\left(\frac{o}{p}, \frac{o}{g}, \frac{o}{w}, \frac{o}{r}, \hat{T}\right) \tag{16}$$

where o is the price of the final product. Thus, the output is positively related to its market price and to technical progress, but negatively related to the input prices.

The system described above assumes that Q, GW, and IW are endogenous variables, and o, p, g, w, r, q, and t exogenous variables. Since the price of intake water is the average cost of materials, labor, and capital incurred in obtaining and discharging water, it may depend on the amount of intake water withdrawn unless the average cost remains unchanged over a relevant range of production. Similarly, the price of gross water may depend on the amount of gross water used. Therefore, to the extent that p and q are indeed endogenous, the above model will be incomplete.

If p and q are exogenous, then the model, as it stands, is step-wise deterministic. The causal direction runs from exogenous variables to output and to gross water and finally to intake water. Because of this independence, each equation can be estimated by the ordinary least-squares method.

The unit manufacturing cost of gross water is the weighted average of the unit costs of intake water and recycled water. If separate price information on intake and recycled water is not available, these costs can be estimated on the basis of the following identity:

$$P_W \cdot GW = (AC_{IW} \cdot IW) + (AC_{RW} \cdot RW) \qquad (17)$$

where $P_W \cdot GW$ is the total cost of gross water, AC_{IW} the average cost of intake water (including effluent costs), and AC_{RW} the average cost of recycled water. The average cost of a unit of intake water is likely to vary with the total volume of water withdrawn (IW), the scale of the plant (S) and the prevailing level of technology (T). Similarly, the cost of recycled water is likely to vary with these variables, as well as with the intensity of water reuse (GW/IW). Assuming linear relationships between

average costs and these variables yields the following relation estimated by applying the ordinary least-squares method:

$$P_w \cdot GW = (a_1 + a_2IW + a_3S + a_4T)(IW) + (b_1 + b_2RW + b_3S + b_4T + b_5GW/IW)(RW) \quad (18)$$

Two water use relations are estimated as shown below:

$$\ln(GW/Q) = a_1 + a_2\ln(P_w) + a_3\ln(Q) + a_4\ln(I) + a_5S + a_6T + u \quad (19)$$

$$\ln(IW/GW) = b_1 + b_2\ln(P_i) + b_3\ln(P_r) + b_4\ln(GW) + b_5\ln(C) + b_6S + b_7T + v \quad (20)$$

where ln is the natural log, and u and v are the disturbance terms. The two specifications are based on (14) and (15). The variables S and T are dummy variables, taking the values of 0, 1, or 1.5 for small, medium, or large plants, and for old, average, or advanced technology, respectively.

74-10, p. 78-89.

2. Price Elasticities of Water

There are three price elasticities which can be computed from the model. They are the elasticities of intake water per unit of output with respect to the prices of gross water, intake water, and recycled water, respectively. Let $F = IW/Q$, $G = GW/Q$, and $H = IW/GW$. The identity:

$$\frac{IW}{Q} = \frac{GW}{Q} \cdot \frac{IW}{GW} \quad (21)$$

can be written as:

$$F(P_w, P_i, P_r, \ldots) = G(P_w, \ldots) \cdot H(P_i, P_r, \ldots) \quad (22)$$

From (22) can be derived the following relations:

$$E_{F \cdot P_w} = (E_{G \cdot P_w}) + (E_{H \cdot P_i} \cdot E_{P_i \cdot P_w}) + (E_{H \cdot P_r} \cdot E_{P_r \cdot P_w}) \quad (23)$$

$$E_{F \cdot P_i} = (E_{G \cdot P_w} \cdot E_{P_w \cdot P_i}) + (E_{H \cdot P_i}) \quad (24)$$

$$E_{F \cdot P_r} = (E_{G \cdot P_w} \cdot E_{P_w \cdot P_r}) + (E_{H \cdot P_r}) \quad (25)$$

where $E_{A \cdot B}$ is the elasticity of A with respect to B.

Since $P_w = (P_i - P_r) \cdot (H + P_r)$, the following equations can further be derived:

$$E_{P_w \cdot P_i} = k + ((1 - \frac{P_r}{P_w}) \cdot E_{H \cdot P_i}) \quad (26)$$

$$E_{P_w \cdot P_r} = (1 - k) + ((1 - \frac{P_r}{P_w}) \cdot E_{H \cdot P_r}) \quad (27)$$

where k is the total cost of intake water as a percent of the total cost of gross water.

The following assumptions are made: (1) any percentage change in the price of gross water results in an equal-percentage change in the prices of both intake water and recycled water; and (2) $E_{P_w \cdot P_i}$ and $E_{P_w \cdot P_r}$ are approximated by k and (1 - k), respectively. These assumptions assure that:

$$E_{F \cdot P_w} = E_{F \cdot P_i} + E_{F \cdot P_r}$$

74-10, p. 93-96.

SECOND PART

W A T E R A L L O C A T I O N

VI

INTERREGIONAL PLANNING OF WATER RESOURCE ALLOCATION

VI. INTERREGIONAL PLANNING OF WATER RESOURCE ALLOCATION

Source:

Interregional Planning of Water Resources Allocations by Systems Analysis Approach -- A Summary Report, by John E. Keith, Jay C. Andersen, Alton B. King, Mark H. Anderson, Thomas C. Anderson, Calvin G. Clyde, and Daniel H. Hoggan, Utah Water Research Laboratory, College of Engineering, Utah State University, Logan, Utah 84322

IWR Contract Report 74-4 July 1974

Scope and Use

This report, called a "Summary Report," consolidates the contents of four Utah Water Research Laboratory publications and of four professional papers. Excerpts from these eight documents are presented in the five Appendices to Section VI.

IWR Contract Report 74-4 addresses the problem of water resource allocation in the State of Utah. Because annual precipitation varies from 5" to over 60" in Utah, interregional water transfers may be desirable, as from the Colorado River Basin to the Great Basin. Hence, interregional planning for optimum water allocation appears essential.

The study had for its objective to develop a methodology for determining optimal allocations of water in Utah, given alternative assumptions and constraints. Specific objectives included determination of:

1. The hydrologic characteristics and cost of water from various sources in each of ten hydrologic study units of the state.

2. Supply functions for water in each hydrologic study unit of the state, given the hydrology and cost of water.

3. The value of the marginal product of water in agricultural uses from crop production, considering productivities of land classes, costs of crop production, and other data.

4. Demand functions for water in each hydrologic study unit of the state, from available agricultural, municipal, and industrial data.

5. The present economically efficient allocation of water among hydrologic study units in Utah, given the profit generating objective function, and the physical and economic constraints of the supply and demand relationships.

6. Changes in these efficient allocations, given alternative projections of demographic changes in Utah.

Initially, water sources were inventoried from state-wide hydrological data. The marginal cost of using water from each source was determined. Existing sources require O + M costs; potential sources require, in addition, development costs. These data were then translated into supply functions.

The next step was to determine the productivity of agricultural land in each hydrologic study unit, by land class, for all the crops which would be grown in significant amounts. Then, costs of production, except water, were subtracted from the revenues produced by each crop to yield a net return per acre. Fixing water inputs at alternative levels allowed demand functions to be developed for water, based upon shadow prices and quantities (value of a unit of water).

The supply and demand relationships were then included in one programming model, which maximized net returns to water in agriculture, given municipal, industrial, and wetland requirements.

The methodology was expressly developed for applicability in any state in the Union. Allocation problems exist in numerous areas, and although watersheds and river basins are convenient units for which to plan allocation solutions, state boundaries have the advantage of coinciding with authorized political jurisdictions capable of implementing proposed water allocation schemes. Existing water supply systems, alternative demands for water, and alternative water salvage, reuse, and transfers are considered by the model. Economically efficient water allocation is the objective function, with maximum net economic returns to water as the optimization criterion.

Among the conclusions of the study were the following:

1. The inclusion of demand and supply analyses as separate components avoids the problems involved in least-cost planning for projected demands. The writers suggest inclusion of demand studies in all planning and feasibility studies. The "requirements" approach to water planning lacks consideration of one-half the problem.

2. Costs of policies which deviate from efficient (or optimal) allocations can be determined using supply functions, demand functions, or both, from mathematical programming. From these costs, public decision-makers can readily and clearly analyze probable results of alternative decisions.

3. Models similar to that developed for Utah can be constructed for other areas, states, or regions. These models can effectively provide analyses of resource allocation decisions which involve costs of much greater magnitude than the cost of developing the model. This approach is believed a reasonable compromise between the high cost of planning and the need for detailed information.

4. Once the model is constructed, changes in structure or coefficients can be carried out at little cost relative to their usefulness in planning.

VI. INTERREGIONAL PLANNING OF WATER RESOURCE ALLOCATION

IWR Contract Report 74-4 and other reports and papers originating from the Utah Water Research Laboratory at Utah State University are primarily concerned with the problem of optimal regional, interregional, and state-wide water resource allocation for municipal, industrial, and agricultural use.

IWR's Summary Report consolidates the Laboratory's series of reports and papers. An effort was made, in the Summary, to fill in missing elements, integrate various aspects of a state-wide water allocation problem, and present in concise and logical fashion an over-all optimal solution, as a tool for interregional water resource planning.

1. Introduction

Optimal water allocation is needed in Utah, where precipitation ranges from 5" to 60" per year. Fortunately, overall water availability is adequate for the foreseeable future. The solution to the problem involves the determination of cost schedules and supply functions for water from various sources, and of value schedules and demand functions for water in various uses. By means of a linear programming model, the economically efficient water allocation can be computed for present and projected demographic conditions.

The marginal cost of using water from any source is the sum of O + M costs of existing facilities, plus development and O + M costs of projected facilities. From such costs can be derived supply functions.

The productivity of agricultural land in each hydrologic study unit, by land class, is determined for all potential crops; production costs (not including water costs) are subtracted from revenues for each crop, to yield a net return per acre. Shadow prices for various quantities of water used allow demand functions to be developed. Supply and demand relationships are then used to maximize net returns to water in various uses. Future requirements modify demand functions and optimum allocations.

74-4 (PRWG 100-5), p. 1-4.

2. Hydrology and Study Areas

The State of Utah has been divided into ten watersheds or hydrologic study units (HSU's). About three-quarters of the total water supply is consumed on grazing land and wasteland. Less than 5% is consumed on irrigated land, and two-tenths of 1% in M + I uses. In addition, 9.5% of the water supply evaporates from lakes, reservoirs, and streams; and 4.3% must be released as an outflow of interstate streams under compact agreements. Inflows to the Great Salt Lake are governed by policies and legal commitments. The manageable portion of the total water supply consists of precipitation, surface run-off, and ground water.

A number of inefficiencies in the present allocation of water in Utah are evident. There are over two million acres of swamp land, marshes, mud flats, and valley bottoms currently saturated, while over three million acres could be added to agricultural production if water were available. More M + I water is required while a major share of Utah's portion of Colorado River water continues to flow out of the state unused, and about $1\frac{1}{2}$ million ac-ft/yr of water are evaporated from the Great Salt Lake. Water table levels are falling as groundwater is mined. Even if a sizable amount of Upper Colorado River Basin water is transferred to the Great Basin by the Central Utah Project, a large-scale water transfer proposed by the U.S. Bureau of Reclamation, approximately one-third of Utah's share of this water will still be unused.

Agricultural return flows find their way into surface and groundwater. M + I wastewater, after appropriate treatment, is available for some form of reuse. Return flows and treated wastewater are computed as the product of coefficients and withdrawals.

Storage requirements to adjust seasonal fluctuations and ensure long-term reserves are estimated. Long-term carry-over storage requirements are derived from the results of frequency mass-curve analyses conducted for 76 streams. A frequency mass curve is obtained by plotting, for any selected probability of occurrence, the expected values of accumulated volumes of runoff during each of many sequences of consecutive months against the carry-over period in months. Separate frequency mass curves are obtained for each probability of occurrence selected.

Since the volume of required storage can be considered a function of (1) probability of not experiencing a shortage, (2) carry-over period, and

(3) demand level, frequency mass-curve analysis provides information necessary for plotting draft vs. storage curves. Draft vs. storage can be computed for the 76 streams by means of a computer program. Draft is expressed in percent of mean annual flow for values of 50, 65, 80, 95, and 110 percent. Storage is given in inches over the watershed. Probability values (probability of not experiencing a shortage) range between 75% and 95%. The long-term storage required corresponds to the maximum values of storage as a function of the carry-over period. These values are determined for each stream at each of the five draft values and probability levels.

The seasonal storage is determined for each HSU by calculating the difference between the supply curve on a monthly basis and the draft requirement for each of the five draft values.

Groundwater recharge potential is assessed in each HSU in order to define the recharge constraint. The problem is to designate areas where artificial recharge to groundwater aquifers is practicable, the water table being low enough to permit recharge, and to estimate for each area the amount of water that could be put underground in basins and/or through wells.

74-4 (PRWG 100-5), p. 5-18.

3. The Model

The model uses supply and demand analysis to determine efficient water allocations. Since both supply of and demand for water are complex, and numerous variables enter into these relationships, mathematical programming is chosen as the analytical technique.

The general statement of the mathematical program is:

Maximize (minimize): $Z = CX$

Subject to the constraints: $AX \gtreqless B$, and $X \geq 0$, where:

Z = the value of the objective function

C = a $(1 \times N)$ vector of returns (costs)

X = an $(N \times 1)$ vector of variables

A = an $(M \times N)$ array (matrix) of coefficients of N variables in M equations, and

B = an $(N \times 1)$ vector of right-hand-side values.

This technique can be used to generate optimum values for the variables, as well as shadow prices (equivalent to Lagrange multipliers) which represent marginal cost or value of those variables. Non-linear relationships and stochastic parameters are difficult to handle.

The allocation model is designed to maximize net profits from the use of water. While the cost of providing M + I customers and wetlands with water is included, efficient allocation is dependent on the agricultural sector. Since there is little change in the amount of water applied to M + I uses, the value of M + I water is constant. The value of water is much higher in M + I than in agricultural uses, so that M + I water use is found at the starting point of the demand curve, high above the cost curve. Also, quantities required for M + I use are minimal as compared with those needed for irrigation. Thus, M + I needs will not be threatened by rising water costs for generations to come. Agricultural requirements, on the other hand, are highly elastic and sensitive to water costs.

The only change in the water allocation pattern over time is due to water added to agricultural pursuits. Therefore, the marginal value of the extra, newly developed water results from its application to agriculture. The optimal water allocation problem is solved by maximizing the difference between returns to agriculture (net of non-water-related production costs) and costs of water use for agricultural production. Maximizing net returns is equivalent to equating marginal supply (cost) with marginal demand (value of marginal product). The equality ensures economically efficient water allocation for all uses.

Variables included in the supply part of the model are water sources, availability from those sources, losses, requirements for various transportation and distribution systems, various outflows, and reuse capabilities. Supply variables are expressed per acre-foot.

Variables included in the agricultural demand part of the model are productivities, input requirements, and rotation schedules for each crop, land class, and county in each HSU. Demand variables are measured per acre. In linking agricultural supply and demand, a factor relating acres of production to acre-feet of water use is provided to make the two model parts compatible. Maximum and/or minimum bounds are established for each variable as appropriate.

Coefficients in the constraint matrix fall into 3 categories: (1) technical relationships of development of water sources and distribution systems; (2) productivity relationships between inputs and outputs in agriculture, including rotation requirements, and (3) water, land, labor, and

other input availabilities. Limits on each constraint (termed right-hand-side values) are determined.

Values of outputs (demand coefficients) and costs of inputs (supply coefficients) are summed up for inclusion in the objective function.

Included in the model are: the production of each crop (by land class, county, and HSU), profit, agricultural water by source, sources of water used by M + I customers and wetlands. Shadow prices for each variable allow construction of both supply and demand curves. Solutions are generated for changed conditions and future projections by altering coefficients and requirements appropriately.

74-4 (PRWG 100-5), p. 19-23.

4. Supply Coefficients and Schedules

The components of water cost are those costs associated with a particular function or process. As the sources are aggregated from the lowest to highest cost (in accordance with the programming algorithm), an upward-sloping, stepped function is determined. It approximates a normal supply curve. Cost components include costs of:

1. Existing transfer facilities (for M + I and agricultural use, and to surface water pool).
2. New transfer facilities to move Colorado River water to the Great Basin (Central Utah Project; Uintah to Sevier transfer).
3. Other new transfer facilities.
4. Existing storage facilities.
5. New storage facilities.

6. Agricultural distribution facilities (present and planned).

7. M + I distribution facilities (present and planned).

8. M + I raw water treatment facilities (present and planned).

9. Wastewater reclamation facilities.

10. Artificial aquifer recharge facilities.

Note: For existing facilities, O + M costs only are included; for new, planned facilities, capital and O + M costs are both included.

Supply schedules are developed through the use of shadow prices of agricultural water for alternative levels of M + I water withdrawals. For any constant level of M + I withdrawal, the remainder of total water available can be used for agriculture, with each source costing a given amount per ac-ft. As M + I withdrawals increase, water is available to agriculture only from higher cost sources, the supply function being shifted upward.

74-4 (PRWG 100-5), p. 23-32.

5. Demand Coefficients and Schedules

Several assumptions are made:

1. M + I water requirements are fixed.

2. Agricultural productivity is fixed at 1980 projections for an average manager.

3. Agricultural prices rise at the same relative rates as input costs. And

4. Timing of water delivery is irrelevant to water value.

Water demand consists of three sectors: the demand by M + I customers, wetlands, and agriculture.

The information on value of water in M + I uses is sketchy when it exists at all. Therefore, M + I uses are entered into the model as alternative fixed requirements, projected for given years to 2020. The fixity of M + I water use is equivalent to a perfectly inelastic demand curve. The total demand curve retains the slope of the sum of the remaining demand curves, but is rightward of it by the amount of M + I requirements.

The value of water for production of recreation, including provision of habitats for wildlife, and other wetland uses, is not readily obtainable. Therefore, wetland consumption is entered into the model likewise at fixed alternative levels,as a perfectly inelastic demand curve. The total demand curve is shifted further rightward.

Agricultural water demand is governed by the productivity of irrigated land. Inasmuch as M + I and wetland demands are assumed fixed, only the value of the marginal product in the agricultural sector in each HSU determines efficient water allocations throughout the state. Because of the large number of variables, simplifying assumptions are helpful:

1. Yields on a given class of land of a given crop in a given HSU are the HSU average for that class of land as projected for 1980.

2. Inputs per unit of a given crop production are the average for the given HSU and land class, and are utilized in fixed proportions as projected for 1980. Variable input costs are identified with quantities of crops grown, and fixed input costs with acreages of land in production. Present and potential land developments are specified by class, county, and HSU.

3. Crop rotations are the normal rotations for the HSU.

4. Prices of agricultural products and costs of production inputs are changing at the same relative rate.

As a result of these simplifying assumptions, unit profitability, and therefore the value of the marginal product, is constant for each crop on a given land class in a given county in a given HSU. Each county has a stepped marginal demand curve including segments for crop rotation patterns by land classes. Because expanding agricultural production mobilizes less and less productive land classes, the stepped marginal demand curves are sloping downward.

Demand coefficients include those of three major demand sectors. The M + I sector experiences a demand based on population projections and industrial development projections. A medium population projection can be derived from available estimates by public and private agencies. Potential large developments of the extractive oil shale industry complicate projections in certain areas. The recent past indicates oil shale may not be developed until other sources from which petroleum can be obtained with less ecological disturbance are exhausted. A slow rate of development, wherein full production of one million barrels of oil a day will not be attained until after 2020, is most probable. A moderate rate of development from which about $1\frac{1}{2}$ million barrels a day will be produced by 2000 is assumed and incorporated in the median projection. Fossil fuel power generation plants are presently under construction and in partial production in certain areas. Further expansion may be severely slowed by environmental considerations. However, the study assumes a moderate development rate of full power generation capabilities and corresponding population increases.

Wetland requirements equal such inflows as will maintain current water levels by offsetting present evaporation of water bodies plus evapotranspira-

tion by phreatophytes and other plants. Some water salvage upstream seems possible without seriously affecting recreation or aesthetics of the wetlands.

Inflows into the Great Salt Lake, while similar in nature to wetland requirements, are treated separately since these inflows are of a large magnitude and play a critical role in water use along the Wasatch front. Three alternative volumes of inflows are used in the model.

The model is parameterized by introducing changes in M + I requirements representative of water demands in each decade to 2020. Optimal solutions are generated for three alternative inflows into the Great Salt Lake, with and without wetland inflow salvage.

The determination of agricultural productivity coefficients is a delicate problem. All information must be broken down by counties within HSU's. All numbers in the demand portion of the model are on a per-acre basis. Presently irrigated and potentially irrigable land class acreages are estimated. The land class percentage breakdown, county by county, is calculated and applied to presently irrigated acreage estimates and, in altered form, to potentially irrigable areas.

Crops included in the study are barley, corn silage, sugar beets, alfalfa hay, irrigated pasture, and dry-land wheat. The latter crop is the only one that can be grown alone. All others must be grown in rotation. Five rotation constraints are expressed as inequalities. Alfalfa is limited to a maximum of 5 to 8 years in succession. The rotation constraints allow numerous crop combinations; however, only five combinations are economically feasible. Other constraints deal with the length of the growing season, labor requirements, and the availability of sugar refineries. Barley is used as a nurse

crop prior to the production of alfalfa hay, or as a cash crop. Irrigation water consumption is determined for every crop in every county in every HSU. Alfalfa may be cut two or three times per season, and requires corresponding water levels. Different crops require different numbers of hours of irrigation and different periodicities.

Demand schedules are constructed by means of the parameterization of water availability. The water variables are incremented, and at each change in productivity of water, shadow prices fall in accord with reduced profitability. Shadow prices are equivalent to marginal values of the product, and the trace of change is a "stepped" demand curve.

After both the supply and the demand portion of the model are completed, the two portions are linked, using the agricultural water consumption/water withdrawal equations. Possible solutions generated from the model indicate the economically efficient solution.

74-4 (PRWG 100-5), p. 32-53.

6. Allocation Solutions

The allocation of water, within and between HSU's, depends upon maximizing net agricultural returns for the entire state. Transfers occur when the value of the marginal productivity is sufficiently high to pay the cost of water transfer and earn an equal or higher net profit in the receiving HSU than in the providing HSU. As long as water is available for agricultural use and a positive net profit is earned, water is allocated to agriculture. If water availability is restricted, it is allocated to that agricultural use and HSU from which the highest net profit can be earned.

A solution for any given level of M + I or wetland requirement is achieved by making appropriate changes in coefficients, right-hand-sides, or bounds. Series of changes are simulated by parameterizations of appropriate variables.

An analysis of the need for the Central Utah Project, using the model, shows that the Ute Indian portion is not required until some time after 2020, unless use of alternative water sources is restricted. The Sevier Area water transfer system, however, appears efficient from the present on for its full transfer capability. The transfer can be made at lower cost than developing new locally available water sources. The following results are provided by manipulation of the model:

1. If salvage of water and groundwater mining is allowed, the Bonneville Unit of the Central Utah Project is not economically efficient until 2010 for inflows to Great Salt Lake of less than 850,000 ac-ft/yr. For inflows up to 1,014,000 ac-ft/yr, 1995 is the indicated timing. With no salvage, low levels of importation are indicated immediately.

2. Development of the Bonneville Unit to full capacity, with water salvage and inflows to Great Salt Lake of 850,000 ac-ft/yr, is not justified prior to 2020. For inflows of 1,014,000 ac-ft/yr without salvage, the date is moved to 1995.

3. Bonneville Unit water is sufficiently costly to be inefficient for new agricultural land development. Only available groundwater is sufficiently cheap as a source of new irrigation water. Low-cost recharge is used for M + I needs, and residual natural groundwater is used for new agriculture. When M + I requirements exceed the low-cost recharge potential, allocation

to new agricultural development is reduced by the amount of M + I requirements above the recharge potential.

4. For inflows to the Great Salt Lake of 850,000 ac-ft/yr or more, if present groundwater aquifer levels must be maintained, full development of the Bonneville Unit is efficient by 2000 with salvage, by 1990 without.

74-4 (PRWG 100-5), p. 54-62.

7. The Cost of Inefficient Allocations

Costs of inefficiency are calculated from either foregone returns to investment or the higher costs of supply. In this calculation, the lag between investment and operation resulting from construction time of large projects, or about 15 years, must be taken into account. To determine the economic cost of inefficient investment prior to the optimal time, all alternatives to transferred water are assumed to be unrestricted. The alternatives include full groundwater development, inflows to the Great Salt Lake of a minimum of 850,000 ac-ft/yr, and a maximum water salvage. Seventy-five percent of full transfer occurs in 2020, setting the investment date at 2005. Total returns foregone to idle facilities if investment occurs in 1973 are the discounted sum of annual returns to the investment funds up to 2005, or for 32 years.

A conservative estimate of investment costs for the Bonneville Unit attributable to M + I and agricultural water users is $130 million. Annual returns are: at 5%, $6.5 million; at 7%, $9.1 million; at 12%, $15.6 million. An annual income of $1,125,000 must be deducted from foregone returns, representing that portion of water importation which is indicated as being efficient.

Net annual foregone returns are, then, $5,375,000 at 5% with present value of $84,936,000; $7,975,000 at 7% with a present value of $100,860,000; and $14,475,000 at 12% with a present value of $117,407,000.

Smaller annual returns are foregone if salvage is permitted and lesser inflows to Great Salt Lake are assumed. In any event, the magnitude of the returns foregone by investing in idle Bonneville Unit facilities is sufficient to offset much of the investment costs. Mistiming of the investments may cause a considerable loss of revenue to the public.

74-4 (PRWG 100-5), p. 64-66.

8. The Cost of Policy Decisions

The cost of institutional constraints can be illustrated by the restriction on groundwater pumping. Two kinds of losses are incurred as a result of the curtailment of groundwater pumping. First, water users suffer higher costs, or losses in consumers' surplus; second, returns to new agricultural development are foregone. The losses in surplus occur in three different forms depending on the type of recharge or transport.

The calculation of losses of producers' surplus to M + I uses involves the summation of three products, as follows: (1) the difference between marginal costs of low-cost recharge and new groundwater, times quantities recharged to replace new groundwater; (2) the difference between marginal costs of high-cost and low-cost recharges, times quantities recharged; and (3) the difference between marginal costs of water transferred to replace recharge and high-cost recharge, times quantities transferred.

Benefits foregone by agriculture in the form of producers' and consumers' surplus are estimated minimally by gross returns less the cost of new groundwater pumped for irrigation, multiplied by the quantity of new groundwater applied to the land. Restrictions on salvage increase losses of returns, since salvage releases additional groundwater for use in new agricultural production. The present value of losses is the sum of the discounted values of annual costs or losses over appropriate periods.

Relaxing institutional constraints on use of locally available water provides benefits to society of a magnitude sufficient to pay off significant amounts of the investment costs in the Bonneville Unit. If public policy is both to limit the development of locally available water, and to invest now so that returns are zero until 1985, economic costs are even higher. Loss of returns and loss of benefits must both be taken into account. At 5%, the present value of the annual loss approximates $30 million, and the value of foregone returns is about $60 million. Total loss of $90 million equals about 70% of the cost of the Bonneville Unit project.

74-4 (PRWG 100-5), p. 66-74.

9. Conclusions

1. The inclusion of demand and supply analyses as separate components of the model avoids the problems involved in least-cost planning for projected demands. In this study, the marginal, or least productive, activity turns out to be the agricultural use, which is highly sensitive to rising costs.

2. Multiple demands can be usefully included in a mathematical model, so that efficient allocations among uses can be determined directly.

3. Costs of policies deviating from efficient allocations can be determined using supply functions, demand functions, or both. From these costs, public decision-makers can readily analyze results of alternative decisions.

4. Hydrologic modeling can be effectively included in a mathematical programming allocation model.

5. Models similar to that developed for Utah can be constructed for other areas, states, or regions.

6. Once the model is constructed, changes in structure or coefficients can be carried out at little cost relative to their usefulness in planning.

74-4 (PRWG 100-5), p. 75-76.

APPENDICES TO SECTION VI

A number of studies and documents pertaining to water resource allocation in the State of Utah preceded, and were consolidated in, IWR Contract Report 74-4. They are independent of the series of IWR-sponsored research projects. Because of their potential interest to Corps of Engineers planners, they are briefly digested in the following five Appendices. Included are four Utah Water Research Laboratory publications identified as PRWG 100-1, 100-2, 100-3, and 100-4, which are covered in Appendices A, B, C, and D, respectively; and four professional papers by members of the Utah Water Research Laboratory, reviewed in Appendix E.

APPENDIX A. WATER ALLOCATION IN AN URBANIZING AGRICULTURAL REGION OF UTAH

Source:

Water Resources Planning to Satisfy Growing Demand in an Urbanizing Agricutural Region, by Thomas C. Anderson, Utah Water Research Laboratory, College of Engineering, Utah State University, Logan Utah 84322.

PRWG 100-1 April 1972

Scope and Use

Despite the author's negative conclusion with regard to the economic justification for inter-basin transfers of water to the Jordan River Basin of Utah for M + I use, the methodology developed for arriving at this conclusion is worthy of attention. The value of water is determined by estimating its supply and demand. The optimal allocation of resources is estimated by combining the supply and demand models into a single linear programming problem. Municipal and industrial water withdrawals are set at estimated 1965 levels and parametrically increased. Additional M + I withdrawals are met through a slight reduction in irrigation withdrawals, and by developing and recharging ground water aquifers. The model indicates that more than double current M + I withdrawals can be supplied in this manner with little or no reduction in irrigation use.

It is concluded that there is sufficient water within the Jordan River Basin to satisfy M + I needs through at least 1990 if not 2010, even if irrigation flows are maintained at 1965 levels. Transfers from irrigation and development within the Jordan River Basin appear to be a much cheaper solution to the basin's impending water problem than inter-basin transfers.

The technique seems to have merit for planning water allocations potentially involving inter-basin transfer, in regions outside the State of Utah.

VI. WATER ALLOCATION IN AN URBANIZING AGRICULTURAL REGION OF UTAH

1. Introductory Remarks

A linear programming model is developed for determining the economically most efficient allocation of surface and groundwater for irrigation and municipal-industrial (M + I) purposes in a defined region of Utah.

The model employed is an application of one of the most elementary of economic concepts: the determination of value by supply and demand. Separate consideration is given to the supply of, and demand for, water in order to estimate its economic value. A supply function used in this way considers precipitation, natural flow, and existing developments, as well as the cost of augmenting existing supplies through groundwater developments, seasonal water transfers, and spatial transfers of water. Economic demand considers the relationship between the quantity of water and its value in each of several complementary and competitive uses. Due regard must be given to the externalities of water use (including secondary benefits and costs when they have a significant impact), as well as its social value for recreation and aesthetic purposes to properly assess a situation.

Virtually all economic activity may be considered as the interrelationship of supply and demand. Exchange should occur whenever the (market or shadow) price of a commodity is such that both buyer and seller gain. Market prices are efficient in allocating resources in competitive markets. Resource allocation of goods for which markets are limited would necessitate bidding

by a governmental agency to account for public goods, as well as internalizing the externalities. The term "shadow price" is used here to depict the marginal value that would arise if all demanders and suppliers could cast their vote whether through the market or a public body.

The buyer-seller relationship can be extended to geographical areas of various sizes. For example, whether a national economy should export or import a particular commodity depends on the domestic supply of, and demand for, the commodity, and the supply and demand in the rest of the world. Domestic supply and demand are functions of supply of and demand for the commodity in the various regions within a country, and eventually the supply and demand of economic agents within the regions.

Analogously, whether a region, be it a state, river basin, or subbasin, should export or import water depends on the supply of, and demand for, water in the region, and the supply and demand in neighboring regions. If the marginal value or shadow price of water in two contiguous regions and the costs of transporting water are known, the economic justification of interbasin transfers is determined. Water should be imported into a region if the shadow price there is above the export shadow price including costs of transportation. Similarly, it should be exported if the marginal value of water in an adjacent region exceeds the marginal value in the region under consideration by more than the costs of transportation.

A critical point in water resource planning in the west is the demand for irrigation water. Irrigation is by far the largest consumptive water use. As the demand for water for uses other than irrigation grows, a

possible way of meeting at least part of this demand is through transfers from irrigation. If the demand for irrigation water is known, the costs of altering the irrigation water supply can be determined.

The problem facing water resource planners concerned with the area studied is mainly one of growing municipal and industrial (M + I) demand. The ways of meeting this growing demand include intra- and inter-basin transfers including transfer between users. The costs of restricting transfers from agriculture can be derived from the model developed for the study.

PRWG 100-1, p. 1-2.

2. The Demand for Irrigation Water

What water is worth to agriculture, in ac-ft/yr. per acre, is determined by net returns in dollars per acre from growing rotating combinations of crops. Only consumptive water use is charged to the operation. Agricultural land is rated as to class according to its suitability and yield in specific uses. Dry-land wheat requires no irrigation; all other crops, including pasture, need irrigation.

The model takes into account, within a defined hydrological area in Utah:

1. Requirements versus availability of water and land;
2. Presently irrigated arable land (in acres) in each of four counties and each of five land productivity classes;
3. Variable and average production costs for each operation and each crop (hours of work and dollar cost per acre);
4. Annual yield per acre of each crop in each land class and each county;

5. Annual consumptive water use (in feet/acre), number of yearly cuttings (for alfalfa), and irrigation hours for each crop in each county;

6. Alternative proportions of acreage planted to each crop, with aggregate net return in dollars per acre for each crop combination, aggregate water use in feet/acre for each crop combination, and shadow prices of water ($/ac-ft) and land ($/acre) for successive increments of water and land use in each crop combination;

7. Crop rotation constraints;

8. Selling price of each crop;

9. Net return, in dollars per acre, of each crop taken separately;

10. Net value of total and marginal product per acre, by level of water application to each crop.

The objective function is resource allocation to utilize available water and land supplies so as to maximize net return. The dual linear programming problem assigns shadow prices to the resources. Parametric solutions of the dual at varying levels of water availability estimate the relationship between the quantity of water and its economic value, or a demand function.

PRWG 100-1, p. 7-18.

3. The Supply of Water

A second linear programming model is used to minimize the cost of water supply. The problem is how to allocate water and related resources so as to meet specified water requirements for various water uses as cheaply as possible.

Alternate water resources include: existing and potential surface and groundwater resources, renovated wastewater, agricultural return flows,

intra-basin and inter-basin water transfers. Costs include: surface and groundwater development, M + I wastewater treatment, groundwater recharge, additional reservoir storage.

Competing uses for the water include: M + I, irrigation, and wetlands. Water outflow is desirable at a level sufficient to maintain a salt balance in the irrigated soil.

Available water includes: surface water, groundwater, and imported water.

The objective function is the least cost method of supplying water. After the linear programming problem has been solved, particular water uses can be varied to determine the cost of meeting the changed requirement. The dual to this problem is one of resource valuation. Its solution estimates the values of the resources and constraints. Parametric solutions to the dual estimate a supply function.

PRWG 100-1, p. 19-22.

4. Optimal Resource Allocation

The optimal allocation of resources is estimated by combining the supply model and the demand model into a single linear programming problem. The combined model indicates that more than double current M + I water withdrawals can be supplied, with little or no reduction in irrigation use, by developing groundwater and recharging groundwater aquifers.

It is concluded that there is sufficient water within the defined region to satisfy M + I needs through at least 1990 if not 2010, even if irrigation withdrawals are maintained at 1965 levels. Additional inter-basin transfers are not an economical source of water now or in the foreseeable future.

PRWG 100-1, p. 23-27.

APPENDIX B. STATE-WIDE WATER RESOURCE ALLOCATION MODEL

Source:

Development of Regional Supply Functions and a Least-Cost Model for Allocating Water Resources in Utah: A Parametric Linear Programming Approach, by Alton B. King, Jay C. Andersen, Calvin G. Clyde, and Daniel H. Hoggan, Utah Water Research Laboratory, College of Engineering, Utah State University, Logan, Utah 84322.

PRWG 100-2 June 1972

Scope and Use

The development and allocation of the water resources within a state require water planners to prepare plans far in advance of the actual time new facilities are needed. Water resources development entails modification of a natural hydrologic system the better to meet man's needs. The interrelationships among elements of the hydrologic system are relatively simple in comparison with the social, legal, economic, and institutional interdependencies involved. The relationships are so complex as to require that planning of water resource development be accomplished on a systems basis. It has become apparent that water resource planning must consider mass transfer of water encompassing areas which have potential for economic growth competing with other areas already highly developed economically. The wisest political decisions and the greatest benefit to the public will result if a method is used to explore the probable consequences of alternative water resources development and management policies and plans. The objective of this study was to extend the capability of systems analysis and operations research to the problem of interregional planning of water resource allocation in the State of Utah.

A state-wide linear programming model was developed to determine a least-cost allocation of water resources to meet projected requirements, subject to constraints such as hydrologic characteristics, limits on inter-basin transfers, restrictions on artificial ground water recharge, and existing water requirements. Data from hydrologic inventories were used to determine availability, reservoir storage-draft relationships, evaporation losses, agricultural return flows, and M + I effluents. Cost data were developed for storage facilities, diversion and canal works, artificial recharge facilities, raw water and waste water treatment. Two sets of supply functions were developed: one for agricultural use and one for M + I use. Parametric linear programming was employed to develop a functional map of the shadow price (margin 1 cost) of water for each of the two uses. The shadow price of imported water (value) to each region within the state was also determined to show the possible economic consequence of inter-basin transfers.

This state-wide water resource allocation model is applicable to the 49 other states.

APPENDIX B. STATE-WIDE WATER RESOURCE ALLOCATION MODEL

1. Systems Analysis Applications to Water Resource Planning

In recent years, systems analysis has become increasingly useful as a tool in water resource planning, design and development, operating procedures, and management.

Systems analysis is a strategy for problem solving which relies heavily on mathematical modeling to assess the technical and economic optimality of alternative systems designs, policies, operating procedures, etc., for performing various functions and meeting various needs with limited resources. Systems analysis may be employed as a decision aid in assessing the technical and economic consequences of alternative courses of action.

A mathematical model is a set of equations which describe some physical, biological, or chemical process and can be classified by three methods: (1) performance versus optimization models; (2) deterministic versus stochastic models; (3) analytical versus simulation models. The optimization model using analytical definitions of the function to be optimized and based on deterministic technology has proven highly useful for water resource planning.

A mathematical programming problem occurs when one seeks to maximize or minimize an analytical function (called an objective function) of one or more variables, subject to certain relationships involving the variables (called constraints). Under certain limited conditions, a solution to this problem

can be found using classical differential calculus, including Lagrangian multipliers and the calculus of variations. The complex engineering and economic aspects of today's water resource problems are far beyond the computational adequacy of the classical methods, and have motivated a keen interest in programming models. Several programming models have been developed, and computational algorithms exist for some of their solutions. These are: linear programming, non-linear programming (including quadratic programming and geometric programming), and dynamic programming.

Linear programming is one of the most widely used of all systems analysis techniques. A linear programming problem might be stated as follows: Given a set of m linear inequalities or equations in r variables ($r \gtreqless m$), non-negative values of these variables are sought which will satisfy the constraints and maximize or minimize some linear function of the variables.

Many applications have been made of the linear programming model to solve problems in water resources. Some of these are: (1) Least-cost plan for waste treatment; (2) Optimum operation of large dams, considering benefits from hydropower and irrigation; (3) Sewage treatment plant design; (4) Conjunctive use of surface water and groundwater.

Non-linear programming is similar to linear programming except the objective function and constraints are not required to be linear functions of the decision variables. One form of non-linear problem for which numerical computation techniques have been developed is known as quadratic programming in which the objective function has quadratic terms subject to linear constraints. A more general, and consequently harder to solve, form of non-linearity occurs with an objective function that is non-linear to a higher

degree than quadratic. This form is known as geometric programming. Geometric programming is just in its infancy in water resources use, but has been used successfully in other applications.

A tool that has been used quite successfully to solve sequential decision problems is dynamic programming. A sequential decision problem is a problem in which a sequence of decisions (termed a policy) must be made and in which each decision affects future decisions. Unlike linear programming, there exists no standard mathematical model format according to which a problem may be structured for solution by dynamic programming. Rather, dynamic programming is an approach-oriented technique, and the particular equations to be used must be developed to fit the problem at hand.

Examples of its use are: (1) Design and operation of multi-reservoir systems; (2) Optimization of individual multi-purpose reservoirs; (3) Minimization of overall cost of waste treatment among discharges; (4) Optimal use of groundwater over time; (5) Optimization of conjunctive use of groundwater and surface water. -- A combination of dynamic programming with linear programming has been used to study the problem of optimal future operation of a water resource system with random streamflows.

PRWG 100-2, p. 3-4.

2. Economic Analysis Relevant to Resource Allocation

Social efficiency, the highest objective of resource allocation, would require that all the social goals of resource development be met. Since, however, it is not possible to describe all social goals in mathematical terms,

the next best objective is economic efficiency. The mathematical model describing economic efficiency can be constrained by social goals. The optimum project is one which is most effective in increasing net benefits subject to social constraints.

In terms of mathematical programming, the objective function of the project is net benefit, and the economic constraint, related to the technical feasibility of the project, is known as a production function. If an input vector of resources is designated as X with an associated output vector as Y, the problem can be stated: Maximize the objective function u(X,Y) subject to the constraint f(X,Y).

To provide insight into the relationships between inputs and outputs for the above statement of optimality, use can be made of calculus. A maximum can be found by differentiating the objective function with respect to each of the vector components, setting each differential to zero, and solving the resulting equations. An artificial unknown called the Lagrange multiplier is introduced. The resulting relationship between inputs and outputs and their respective prices is:

$$\frac{du/dx_i}{du/dy_j} = -\frac{dy_j}{dx_i} = \frac{p_i}{p_j} = \text{the marginal physical product (MPP)} \qquad (1)$$

$$i = 1, 2, \ldots h, \ldots$$

$$\frac{du/dx_i}{du/dx_h} = -\frac{dx_h}{dx_i} = \frac{p_i}{p_h} = \text{the marginal rate of substitution (MRS)} \qquad (2)$$

$$j = 1, 2, \ldots k, \ldots$$

$$\frac{du/dy_j}{du/dy_k} = -\frac{dy_k}{dy_j} = \frac{p_j}{p_k} = \text{the marginal rate of transformation (MRT)} \quad (3)$$

Analysis of these equations indicates:

du/dx_i = Marginal cost of input i = price of input i = p_i

du/dy_j = Marginal benefit of output j = price of output j = p_j

dy_j/dx_i = Marginal physical product (MPP), or the additional output which can be produced per unit of input

dx_h/dx_i = Marginal rate of substitution (MRS), or the marginal rate at which the h-th input can be substituted for the i-th input while holding production constant

dy_k/dy_j = Marginal rate of transformation (MRT), or the marginal rate at which production can be shifted from the j-th output to the k-th output.

The following set of rules applies:

Rule 1. The Optimum Allocation of Goods: Each consumer maximizes his satisfaction by ordering his consumption so that the marginal rate of distribution between any two goods is equal to the ratio of their prices.

Rule 2. The Optimum Degree of Specialization: Each firm maximizes its profits by making its marginal rate of transformation between any two outputs produced equal to the ratio of their prices (Equation 3).

Rule 3. The Optimum Relationship Between Input and Output: Each firm maximizes its profit by equating the marginal physical product of input in producing output with the ratio of their prices (Equation 1).

Rule 4. The Optimum Allocation of Inputs: Each firm maximizes its profit (minimizes its cost) by making its marginal rate of substitution between any two inputs used in production equal to the ratio of their prices (Equation 2).

The ideal market under conditions of pure competition would automatically achieve these optimum objectives. The allocation of resources to different tasks in different ways is a problem in the theory of production. It is a problem in linear economics since the restrictions on the problem are linear in that the total amount of any resource devoted to all tasks must not exceed the total amount available. Thus, each restriction is a simple sum.

Each of the three equations derived above has an equivalence in linear programming. Linear programming solves the problem of the choice of optimal production technique or process (Equation 2) and the problem of the choice of optimal product mix (Equation 3).

Consider the case of production possibilities for a product with only two variable inputs (X_1 and X_2) and three possible production techniques or processes (L, M, and N). As shown in Figure 1, lines of constant production rates (isoquants) are in straight line segments with slope changes occurring at the lines representing the process. Not only can the product be produced with any given process, but a combination of processes can be used. For example, production at point K of 200 units reflects a combination of 100 units from process L and 100 units from process M. Production processes are assumed to have constant returns to scale, i.e., each input requirement is proportional to the output level, and the processes can be operated in combination without altering the structure of input requirements. If the unit price of input X_1 is p_1, and of input X_2 is p_2, then

$$C = p_1X_1 + p_2X_2 \ldots\ldots \qquad (4)$$

represents production cost as a function of X_1 and X_2. It is shown in Figure 5

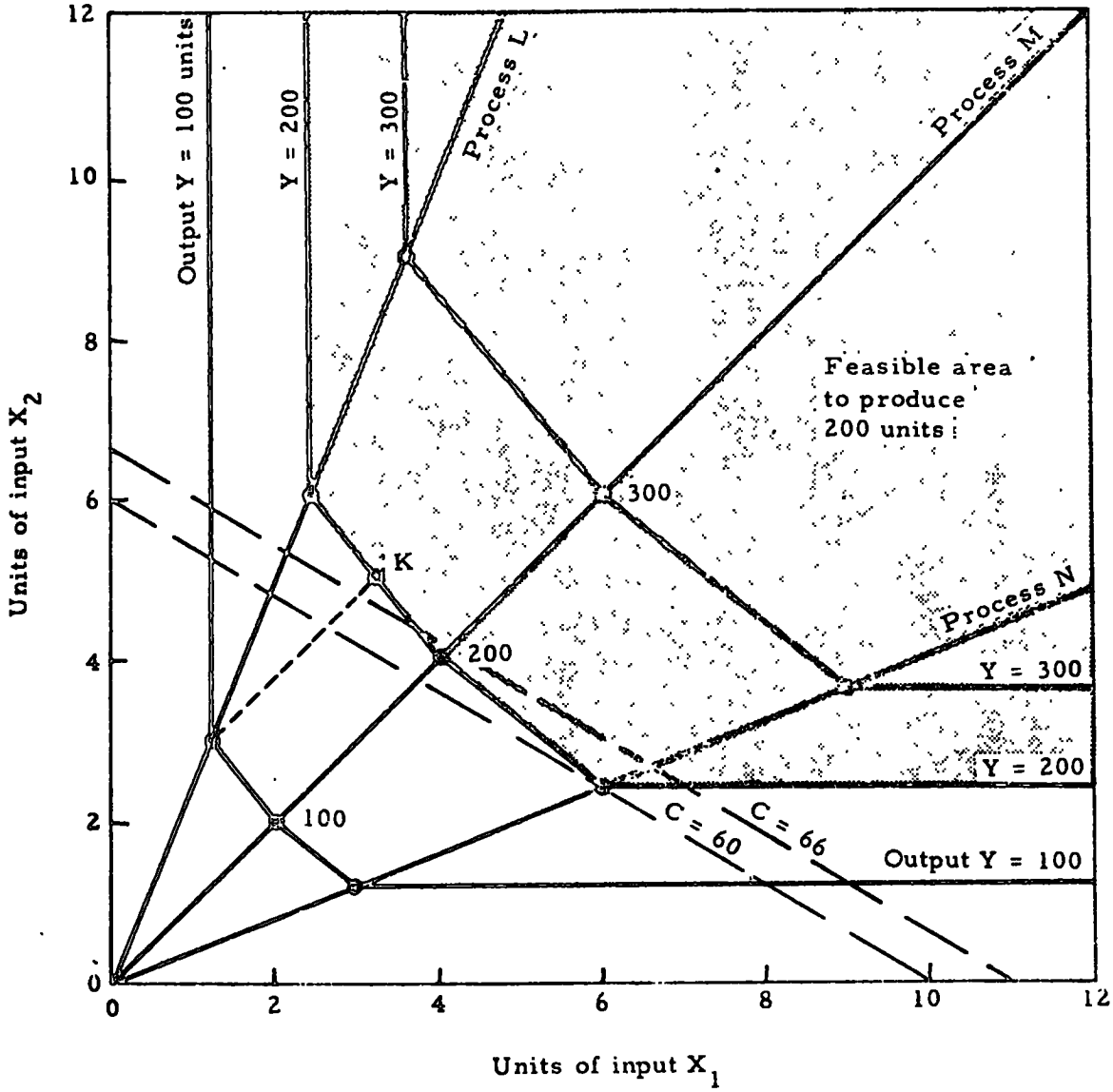


Figure 5. Determination of the Choice of Inputs

as constant cost (iso-cost) lines for $p_1 = 6$ and $p_2 = 10$. Note that the slope of the iso-cost line is the inverse ratio of the input prices, p_1/p_2. Suppose it is desired to produce 200 units at lowest cost. The shaded area is the feasible production area. The problem in linear programming format becomes:

Objective function: Minimize $C = qX_1 + rX_2$

Constraints: Lines representing the 200-unit isoquant

The solution can be found graphically to be:

1. Cost = 60
2. X_1 = 6.0 units
3. X_2 = 2.4 units
4. Process is N.

In marginal analysis, the isoquant would be a smooth curve, and the contact with the line of iso-cost would be a point of tangency. Thus, the marginal rate of substitution (tangent line to the isoquant) would equal the price ratio of the two inputs (slope of the iso-cost line). Equation 2 would be satisfied. In linear programming, the isoquant is a series of straight line segments resulting in its slope (MRS) being discontinuous and undefined at the intersection of two segments. Thus the price ratio at a condition of optimality can vary between the values given by the MRS on each side of the optimal point.

Now consider the case of production possibilities for a firm having two possible products (Y_1 and Y_2) and two inputs (X_1 and X_2). Assume the inputs could not exceed 150 units each and the two production functions are:

$$15Y_1 + 10Y_2 \leq 150 \quad \text{for input } X_1$$

$$10Y_1 + 15Y_2 \leq 150 \quad \text{for input } X_2$$

The shaded area in Figure 6 is the feasible production area common to both inputs. If the unit profit from output Y_1 is p_1 and from Y_2 is p_2, then:

$$P = p_1Y_1 + p_2Y_2 \ldots\ldots \qquad (5)$$

represents the profit as a function of Y_1 and Y_2. It is shown on Figure 6 as a constant profit (iso-profit) line for $p_1 = 12$ and $p_2 = 10$. The two lines shown represent a profit of 144 and of 132.

Suppose it is desired to find the combination of outputs which bring the greatest profit. The problem in linear programming format becomes:

Objective function: Maximize $P = 12Y_1 + 10Y_2$

Constraints: $15Y_1 + 10Y_2 \leq 150$

$10Y_1 + 15Y_2 \leq 150$

The solution is found to be:

1. Profit = 132
2. Y_1 = 6 units
3. Y_2 = 6 units
4. X_1 = 150 units
5. X_2 = 150 units.

An argument can be presented just as before to show that the price ratio is not equal to the marginal rate of transformation (tangent to production line) but can vary between the values given by the MRT on each side of the optimal point. This is the equivalent in linear programming to Equation 3.

It can also be shown how linear programming solves the problem of the optimal relationship between output and input (Equation 1). The production

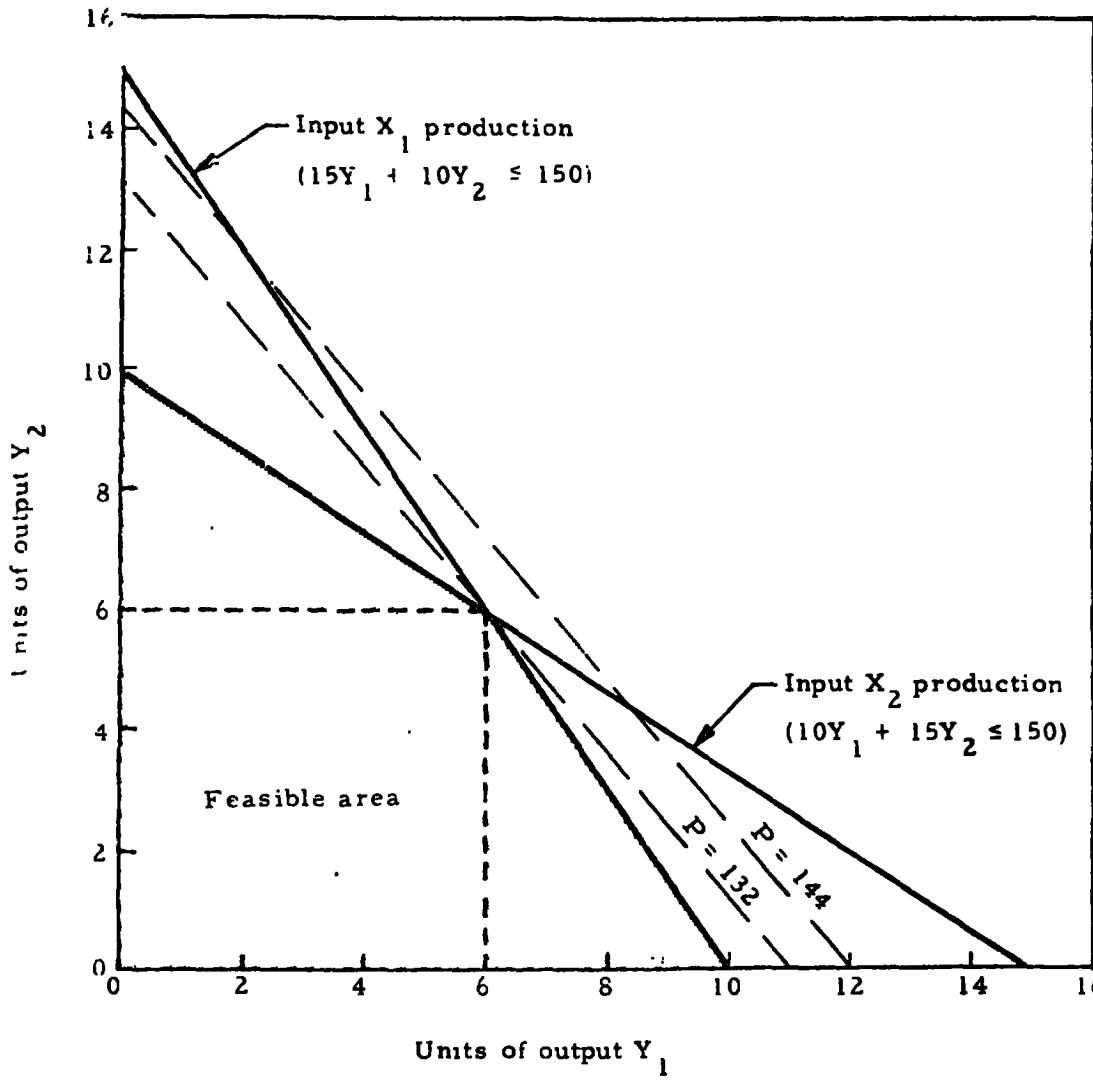


Figure 6. Determination of the Choice of Outputs

function for output Y and input X_1 can be determined from Figure 5 by relating the output at any given value of input X_2 to the input X_1 (found by taking horizontal cuts across the graph). After non-dimensionalizing with respect to X_2 (divide each term by X_2), the production function is shown in Figure 7. This curve holds for any value of X_1 due to the basic assumption of constant return to scale. If the unit price of input X_1 is p_1, of input X_2 is p_2, and of output Y is p, then:

$$N = pY - p_1X_1 - p_2X_2 \ldots\ldots \qquad (6)$$

represents the net profit. After non-dimensionalizing with respect to X_2, this function becomes:

$$N(X_2) = (\frac{N}{X_2} + p_2) = p(\frac{Y}{X_2}) - p_1(\frac{X_1}{X_2}) \qquad (7)$$

It is shown on Figure 7 as a constant net-profit-per-unit-of-input-X_2 iso-net line. If $p = 3$ and $p_1 = 6$, then the two lines shown represent $N(X_2)$ of 220 and 235. Suppose it is desired to find the combination of output Y and input X_1 which would bring the greatest net profit. The problem in linear programming format becomes:

Objective function: Maximize $N(X_2) = 3(\frac{Y}{X_2}) - 6(\frac{X_1}{X_2})$

Constraints: Lines representing the production function.

The solution is found to be:

1. $N(X_2) = 235$
2. $Y/X_2 = 83.3$
3. $X_1/X_2 = 2.5$

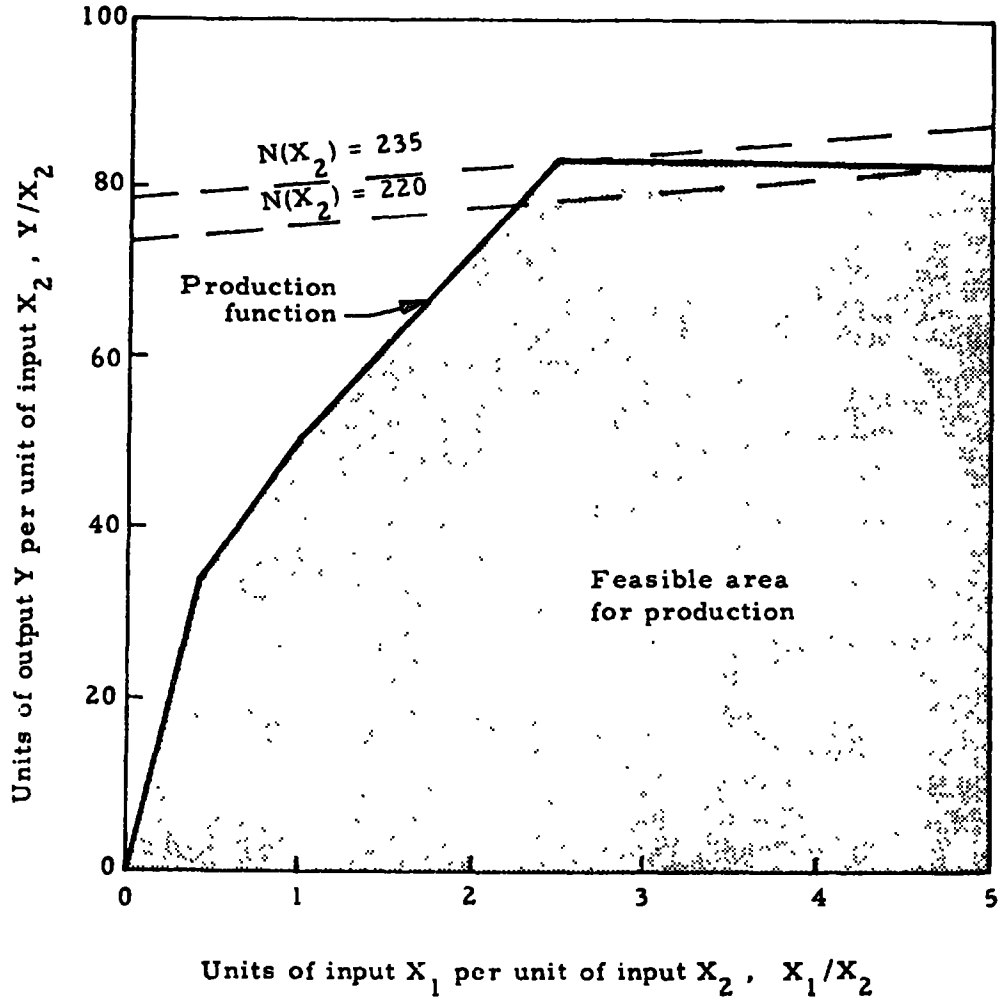


Figure 7. Determination of the Relationship Between Output and Input

Now the slope of the production function is the marginal physical product (MPP) of output Y with input X_1. Also, the slope of the iso-net lines is the inverse ratio of the prices, p_1/p. Using similar arguments as before, it can be shown that the price ratio is not equal to the MPP but can vary between the values given by the MPP on each side of the optimal point. This is the equivalent in linear programming to Equation 1.

This problem could have been solved including the second input X_2 by working with a three-dimensional problem rather than a two-dimensional problem. The production function would be a surface rather than a line; likewise, the iso-net function would be a plane. The optimum would occur at the point of tangency of the iso-net plane with the production function surface.

One of the additional benefits of the linear programming technique is called sensitivity analysis. Such an analysis performed for prices in the first equivalent linear problem discussed above indicates the range over which each of the prices p_1 and p_2 can vary (holding the other price fixed), such that the optimal combination of inputs remains unchanged. These ranges are determined from the range of the price ratio (slope of iso-cost line) for which the optimal point remains unchanged. Similar arguments can be made for the second and third problems.

PRWG 100-2, p. 4-9.

3. Systems Analysis Model for Optimal Allocation of Water Resources in Utah

The State of Utah is divided into ten watersheds called Hydrologic Study Units (HSU's), for the purpose of total water allocation at minimum overall cost. For each HSU, the following assessments are established, in ac-ft/yr:

Water availability from aquifers and surface sources;
Evaporation from lakes and reservoirs;
Existing storage capacity;
Groundwater use for irrigation;
Excess precipitation on irrigated croplands;
Transbasin diversions;
Withdrawals for M + I use;
Withdrawals for agricultural use;
Inter-HSU diversions;
Agricultural return flows;
M + I treated wastewater effluent;
Depletions (other than evaporation), including consumption by wetlands;
Outflow (negative in those HSU's in which groundwater is being mined).

In-basin water availability and gross supply are determined for each HSU. Storage requirements to regulate seasonal fluctuations and provide carry-over for long-term droughts, are estimated. Probability values of 75%, 80%, 85%, 90%, and 95% are used. Draft is expressed in percent of mean annual flow for values of 50%, 65%, 80%, 95%, and 110%. The seasonal storage is added to the long-term storage to determine the total storage required in each HSU. Storage-draft curves are established for each HSU which tell the reader the size of reservoirs needed (in ac-ft of capacity) to yield a given flow (in ac-ft/yr) representing a given percent of the mean annual flow at a given percent probability level.

The groundwater recharge potential is assessed for each HSU, on the basis of criteria establishing limits on the amount of water that can be artificially recharged each year.

The water resource allocation model for the State of Utah falls in the category of a general linear programming problem. The objective function is an expression for the total cost in dollars of meeting the demand for water resources in Utah. The variables are made up of all the alternatives of allocation which may combine to form the solution of the problem. These variables represent a quantity of water to be allocated to a given alternative use, in ac-ft/yr. Each variable has an associated cost coefficient which reflects the cost of allocating one ac-ft/yr to the given alternative or activity. The objective function thus represents the total cost for allocation of Utah's water resources in dollars per year.

The variables can be grouped into eight general categories:

1. Variables showing the amount of water which will be transferred from the Colorado River Basin to the Great Basin. This category is structured primarily on the basis of the proposed Central Utah Project of the U.S. Bureau of Reclamation, parts of which are now under construction (Bonneville Unit, Ute Indian Unit).
2. Variables showing the amount of local surface water used in the 10 HSU's.
3. Variables showing the amount of groundwater used in 8 HSU's; two of the Units have insufficient groundwater to make its use economically feasible.
4. Variables dealing with the amount of local surface water which must be stored.
5. Variables dealing with the evaporation loss from storage reservoirs.

6. Variables showing the amount of water that appears as return flow or treated wastewater effluent in each HSU.

7. Variables showing the amount of water used to recharge the groundwater basins.

8. Variables showing the amount of outflow from each of the 10 HSU's.

The linear constraints consist of six general categories of equations or inequalities:

1. Constraints dealing with the amount of water available for allocation within the HSU's.

2. Constraints dealing with the water demand to be met within each HSU.

3. Constraints dealing with the amount of water which must be stored in order that surface water which runs off in the spring can be available for use later in the year and for use in extended droughts. Included in this category are equations which allow for evaporation loss from reservoirs.

4. Constraints dealing with the amount of return flow and M + I effluents. More water is withdrawn for municipal, industrial, and agricultural use than is comsumptively used, and the remainder appears as effluents and return flow.

5. Constraints dealing with the amount of goundwater that is used freely by wetlands.

6. Constraints dealing with limitations imposed by groundwater recharge, inter-basin transfers, and surface and groundwater outflow.

A Flow Diagram of the allocation model is shown in Figure 8. The symbols used in the diagram are explained in a legend appearing on the page following the figure.

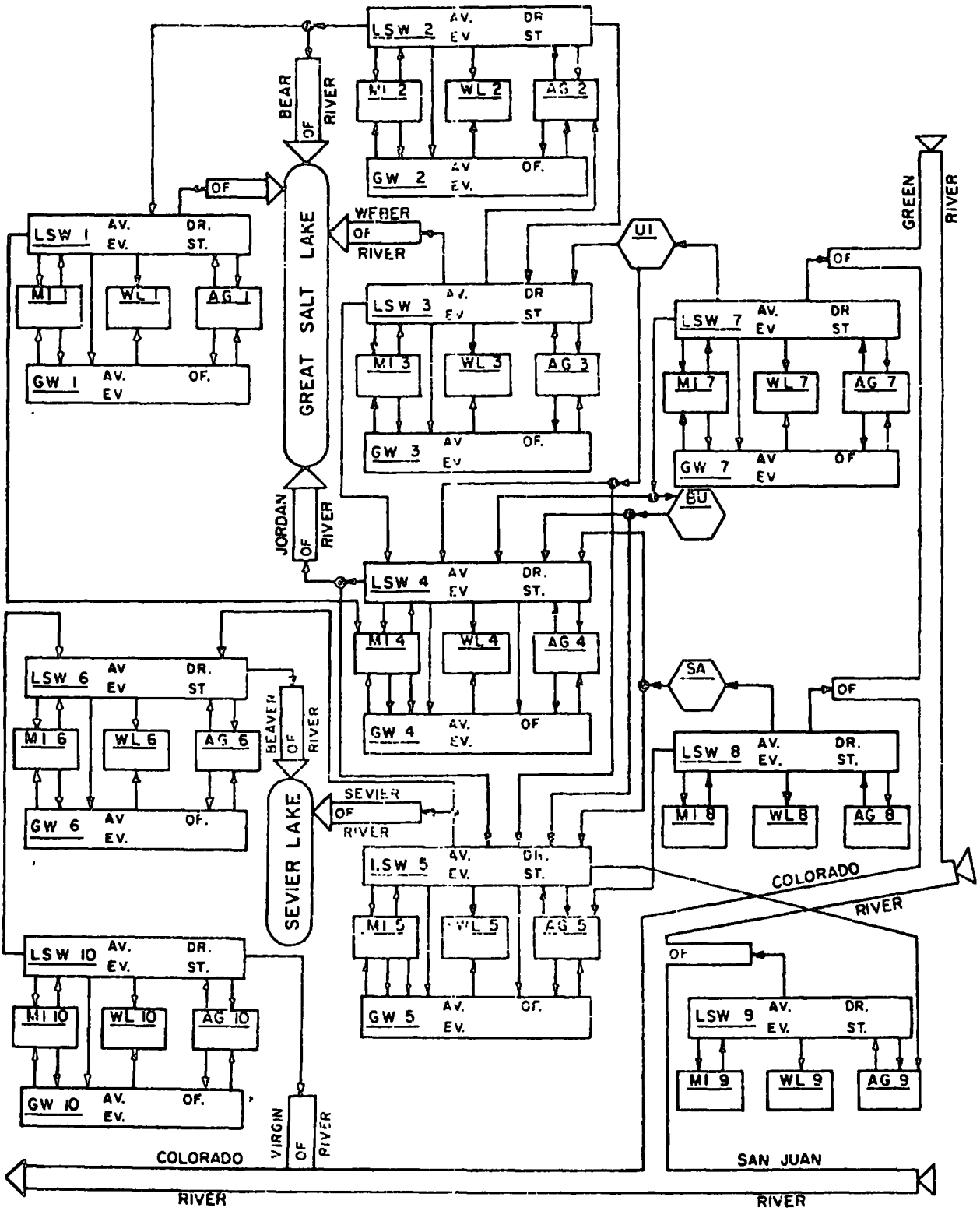


Figure 8. Flow Diagram of Allocation Model

LEGEND FOR FIGURE 8

LSW	=	available local surface water
AV	=	amount of water available for allocation
EV	=	net evaporation loss from surface storage
DR	=	required draft on stored water in surface reservoirs
ST	=	amount of surface water storage needed to satisfy the required draft
MI	=	demand for water by municipal and industrial users
WL	=	demand for water by wetlands
AG	=	demand for water by agricultural users
GW	=	available groundwater
OF	=	outflow from the HSU
UI	=	Ute Indian Unit of inter-basin transfer
BU	=	Bonneville Unit of inter-basin transfer
SA	=	Sevier Area suitable for inter-basin transfer

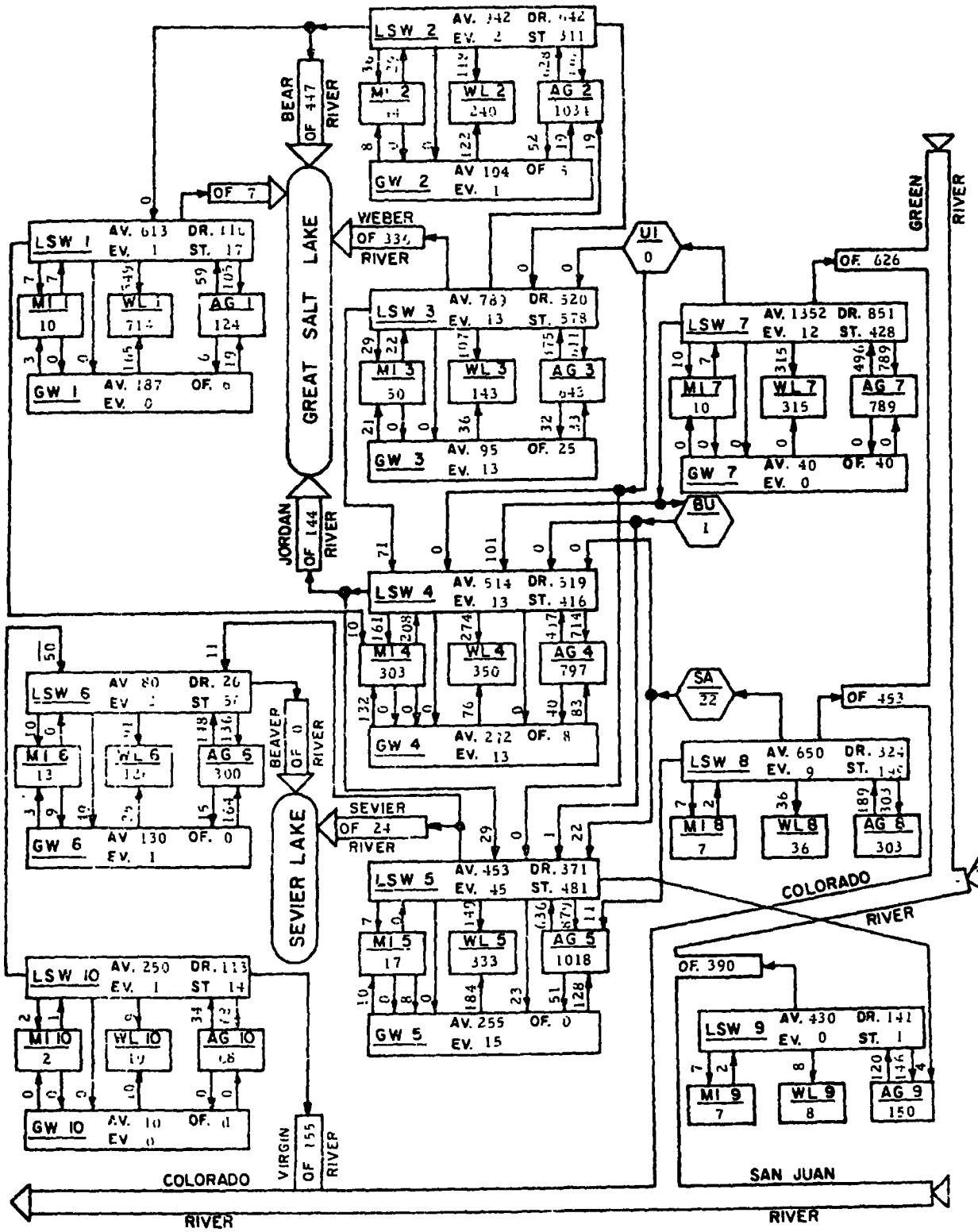
PRWG 100-2, p. 11-30.

4. Optimal Solution from the Model

Solution of the linear programming problem consists of several parts, including the optimum value of the objective function, the optimal activity levels or values of real and slack variables, and the solution of the dual to the linear programming problem.

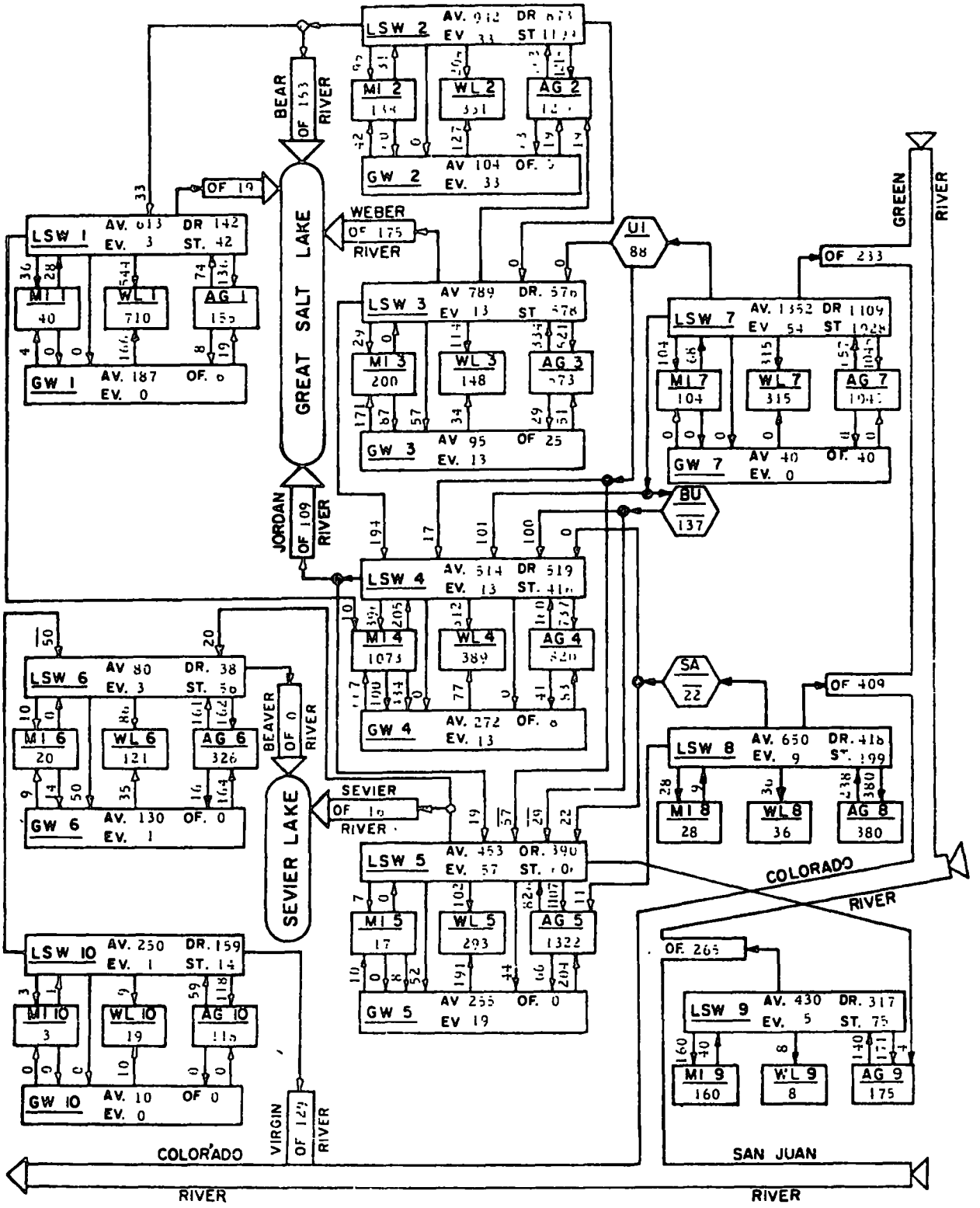
The optimum value of the objective function is used primarily to compare one optimum solution with another. Here, the optimum value, which equals $9,722,447, represents the minimum annual cost of development of new facilities to meet the specified demands for water under a particular set of assumptions. The solution is based on water demands for the year 1965, and the assumption is made that groundwater mining is not permitted. Since facilities existing in 1965 are in the model at zero cost, the value of the

objective function in this case represents the yearly cost of developing new facilities to eliminate groundwater mining in two of the HSU's. Cost projections over time are made by examining changes in the value of the objective function, as right-hand-side values of the demand constraints are changed. For example, under a given growth projection, the allocations for 1965 and 2020 are given by the two diagrams in Figures 9 and 10 below:



FLOW DIAGRAM FOR ALLOCATION MODEL

Figure 9. Water Allocation for the Year 1965



FLOW DIAGRAM FOR ALLOCATION MODEL

Figure 10. Water Allocation for the Year 2020

Solution of the dual to the linear programming problem determines resource shadow prices. The economic interpretation of the dual property of linear programming lies in the concept that resource allocation and pricing are two aspects of the same problem. The dual problem is formulated as follows:

(a) Transpose rows and columns of the constraint matrix;
(b) Transpose the right-hand-side of constraints with the objective function coefficients;
(c) Change the sense of the inequality signs in the constraints;
(d) Change the sense of the objective function (e.g. maximize instead of minimize).

The optimal solution of this dual problem gives the values of the dual variables which are referred to as shadow prices and indicate the rate at which costs increase or decrease for a corresponding increase or decrease in the amount of resource given by the right-hand-side value of the resource constraint. For example, the shadow price or value of the resource "available surface water" in one of the HSU's is $14 per ac-ft/yr. This says that the value of the objective function (which is new development cost) would change by $14 per year if the available surface water were changed one ac-ft/yr. Thus, the value of this resource is defined.

PRWG 100-2, p. 57-59.

5. Post-Optimization Analysis

It is usually desirable to conduct a sensitivity analysis to determine the effect on the optimal solution of changing certain coefficients or constants to other possible values. If such an analysis indicates the optimal solution is very sensitive to small changes in the coefficients or constants,

then special care should be taken in checking the values of these coefficients or constants. Thus, one of the greatest helps which can come from a sensitivity analysis is the identification of those coefficients or constants which are critical to the solution, thereby reducing the number that must be reexamined.

Parametric analysis is a procedure for generating new optimal solutions from an original optimal solution while allowing one or more parameters (constraints or coefficients) to vary systematically over a specified range of values. Either the objective function coefficients or the constraint right-hand-side values or both can be varied over a desired range, either singly or in any combination. Use is made of this procedure to vary the right-hand-side values of some of the constraint equations, in particular those showing the demand for water. Thus, projections of demand over time can be inserted in the model, and new optimal solutions generated quite easily.

PRWG 100-2, p. 59.

6. Conclusions

Based on an analysis of the results based on assumptions discussed earlier, it is concluded that parametric linear programming is an efficient tool to determine optimum allocation of water resources, either on a regional basis without inter-regional transfers of water or on a statewide basis with inter-regional transfers. Additional conclusions are reached as a result of a regional analysis:

1. Except for 3 of the HSU's, moderate increases in agricultural development are possible, without new water transfer or storage facilities, at a cost of less than $15 per ac-ft/yr.

2. Except for 3 of the HSU's, substantial increases in M + I development are possible, without new water transfer or storage facilities, at a cost of less than $75 per ac-ft/yr.

3. In the remaining 3 HSU's, any further development requires water transfer.

4. Elimination of groundwater mining can be accomplished either by reducing the present level of agricultural development or by water importation.

5. Imported water is of little value in large areas of the state, particularly for increased M + I development. It appears economical to import water only if development demands reach such high levels as to require construction of new surface water storage reservoirs.

Conclusions reached as a result of statewide analysis include:

1. The degree to which major importations of water from the Upper Colorado River Basin to the Great Basin are needed is directly related to the inflow requirement into the Great Salt Lake.

2. Reducing the inflow requirement to the Great Salt Lake to about one-half the present requirement allows development to continue to about the year 1990 with only the Bonneville Unit of the Central Utah Project required.

3. Elimination of groundwater mining is feasible through water transfer from Lower and Upper Colorado River Basins.

4. Development of groundwater resources is a key element in the determination of the least-cost allocation pattern.

5. Areas where it is essential to reexamine water supply costs are easily identifiable by sensitivity analysis.

6. Surface water storage and transfers are the prime development facilities limiting growth in the state.

One major conclusion can be made concerning development of new facilities in the Upper Colorado River Basin to export water to the Great Basin. The low value of imported water makes it imperative to determine the value of maintaining the current inflow to the Great Salt Lake. Without this knowledge it is impossible to state categorically that new importation facilities must be constructed to a certain timetable. Whether this value is expressed in monetary terms or in some socially related terms, it nevertheless is a key factor in judging which alternative should be chosen. -- Another factor in this judgment is the value to the state of laying solid claim to Utah's share of the Colorado River water. Developing facilities early and using this water would establish Utah's allocation on much more solid ground in any jurisdictional dispute which might develop in the future. -- Trade-offs between these two factors are a primary concern of the state's water planners. Use of operation research techniques to develop tools such as this model will be of great assistance to planners in performing trade-off studies.

PRWG 100-2, p. 69.

APPENDIX C. INTER-BASIN WATER TRANSFERS FOR UTAH's AGRICULTURE

Source:

The Economic Efficiency of Inter-Basin Agricultural Water Transfers in Utah: A Mathematical Programming Approach, by John E. Keith, Jay C. Andersen, and Calvin G. Clyde, Utah Water Research Laboratory, College of Engineering, Utah State University, Logan, Utah 84322.

PRWG 100-3 June 1973

Scope and Use

The economic efficiency of water development in Utah, including transfer systems, has seldom been examined, nor have the costs of public policies which result in deviations from efficient allocations. In order that public officials be better informed about water allocations, the present effort examines the efficient allocation of water in time frames up to 2020 under several alternative assumptions. and calculates the cost of alternative policies.

Using mathematical programming techniques, a computer model is developed to determine the supply (marginal cost) and demand (value of marginal product) relationships for agricultural, M + I, and wetland water. The model maximizes net profit per acre to an average farmer in each region of Utah. The optimal solution generated is an efficient allocation, since net profit maximization occurs only when value of marginal product equals marginal cost.

Using alternative assumptions about public policies, several alternative distributions are determined.

This study has applicability to other states faced with an inter-basin water allocation problem.

APPENDIX C. INTER-BASIN WATER TRANSFERS FOR UTAH's AGRICULTURE

1. Inter-Basin Allocation Theory

In perfectly competitive markets, assuming no externalities, the efficient allocation of any scarce resource to production of some product or group of products requires that the marginal cost (MC) or supply of that resource be equal to the value of its marginal product (VMP) or demand. Further, if that scarce resource is to be allocated optimally between competing uses, the VMP of the resource in production must be equal as among all competing uses. If a given resource is utilized in two pursuits, one of which has a lower VMP (disequilibrium in a competitive market), then efficiency would require a shift in use until the value of the marginal product is equal among the competing uses. If a resource is to be transported from one region to another, the same efficiency criteria apply, assuming constant and equal income and other prices between regions. The VMP of the resource must be sufficiently high in the receiving region to compensate for the marginal costs of production and transfer costs. Further, the VMP in the receiving region, net of transfer costs, must equal that in the supplying region. Otherwise, reallocations of the resource will yield increased returns. The demand curve is simply the VMP of various levels of a given input in the given use in a given area. The supply curve is simply the marginal cost (MC) of producing the resource or input, ceteribus paribus. The development efficiency conditions are met at the intersection of the supply and total demand curves. A movement along the supply or demand curve toward equilibrium is an efficient movement.

Figure 11 is a graphic illustration of the in-region efficient allocation of water in Utah with the following symbols:

P^i = the equilibrium price in the i-th HSU;

D^k_{ij} = the demand for water from the k-th HSU for the j-th use in the i-th HSU. D^k_{ij} can be used interchangeably with VMP^k_{ij};

Q^k_{ij} = quantity of water from the k-th HSU purchased by the j-th use in the i-th HSU;

S^k = supply of water from the k-th HSU; S^k can be used interchangeably with MC^k;

X = indicates excess supply or demand;

i = HSU number = 1, ..., 10;

k = HSU number = 1, ..., 10;

j = use (1 = agriculture, 2 = M + I, 3 = wetland).

Taking the demand curves for each use, and summing those demands into one total demand curve for a given HSU ($D^k_{i,TOT}$), when equated to the supply curve, will yield the price to be paid and the efficient allocations as between uses within the HSU of the total water ($Q^k_{i,TOT}$). Note that at every price less than P^1, there exists an excess demand for water, where:

$$D^1_{X,TOT} = D^1_{1,TOT} - MC^1.$$

Figure 12 illustrates that excess demand curve. $D^1_{X,TOT}$ is the relevant demand for transferred water in HSU 1, and it should be added to all other demands in the HSU from which water is to be transferred.

Any transfer costs (T) must be netted out from this excess demand to yield the demand for water from HSU 2 in HSU 1; mathematically,

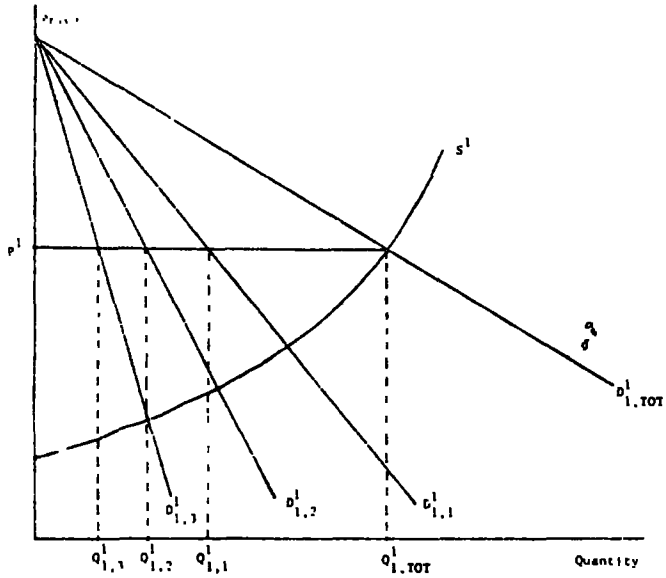


Figure 11. Demand and Supply Between Competing Uses

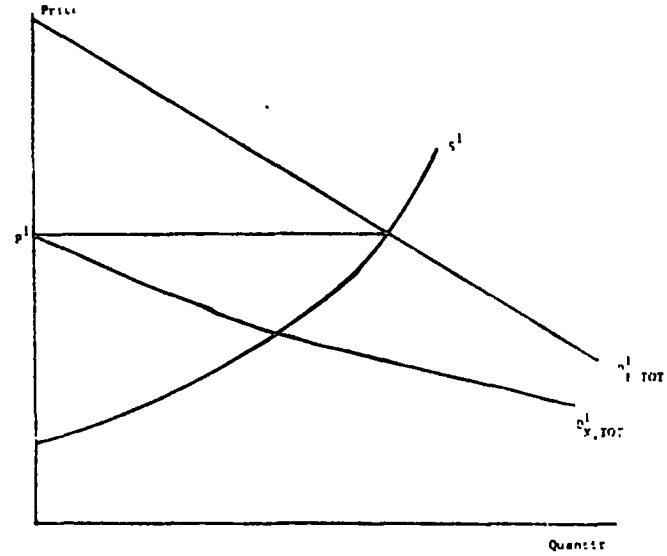


Figure 12. Excess Demand in HSU 1

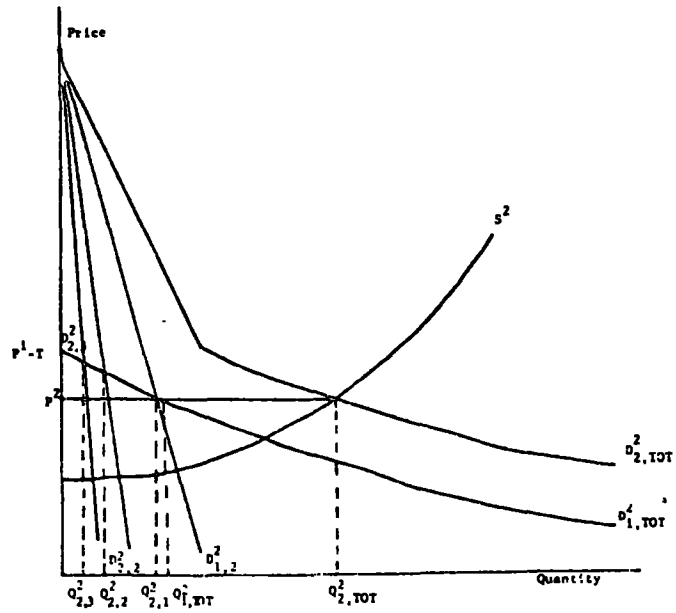


Figure 13. Water Allocation in HSU 2

$$D^2_{1,TOT} = D^1_{X,TOT} - T.$$

Figure 13 indicates the efficient allocation in HSU 2. The price established in HSU 2, P^2, plus the transfer cost, will be the equilibrium price in HSU 1, and will determine water allocations in HSU 1, as indicated in Figure 14.

$Q^2_{2,TOT}$ is the water allocated to HSU 2 among uses in HSU 2. $Q^2_{1,TOT}$ is the water transferred from HSU 1 to HSU 2. The total water in HSU 1, $Q^1_{1,TOT}$ + $Q^2_{1,TOT}$, is allocated among uses. The net VMP in HSU 1 ($D^1_{1,TOT}$ - T) is equal to the VMP's in all uses in HSU 2, and the VMP's of each use in HSU 1 are greater than those in HSU 2 by exactly the transfer costs. The extension of M uses and N regions is graphically laborious, but mathematically straightforward. The identical solutions as proposed on the demand side can be reached by using an excess supply curve ($S^2_X = S^2 - D^2_{2,TOT}$ above in the in-region equilibrium in HSU 2), summing it horizontally with S^1, and taking the equilibrium. Figures 15 and 16 illustrate the approach from the supply side.

If complementarity of outputs results from the resource transfers, then the measure of costs and benefits to all the products of the transfers must be included in the analysis. The analysis of joint products applies to this complementarity. The benefits and costs which accrue to other products are netted from those of the output in question. The residual cost (total cost net of joint costs) is considered as the relevant marginal cost for a given use.

PRWG 100-3, p. 6-8.

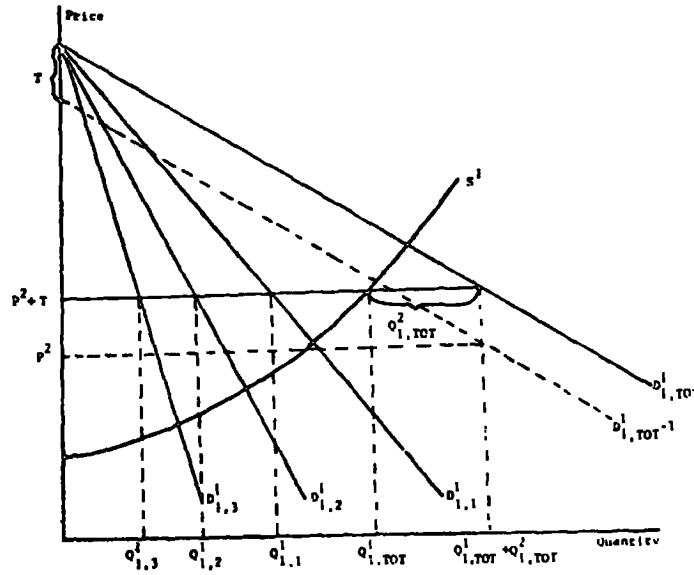


Figure 14. Efficient Allocation in HSU 1

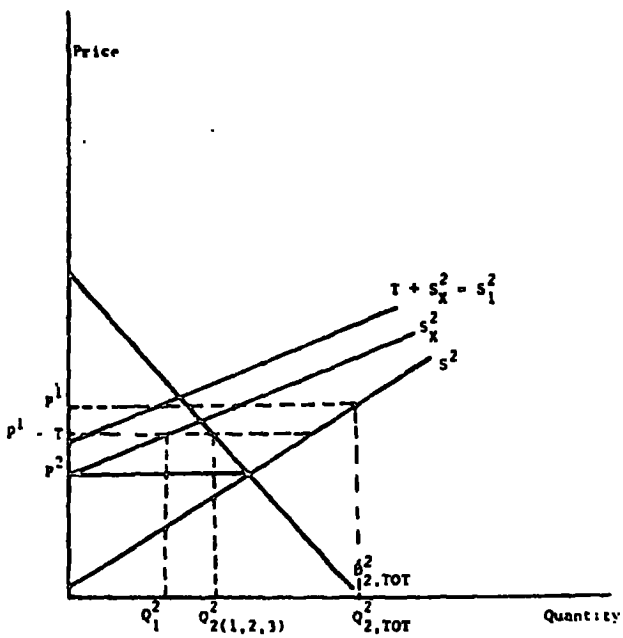


Figure 15. Excess Supply in HSU 2

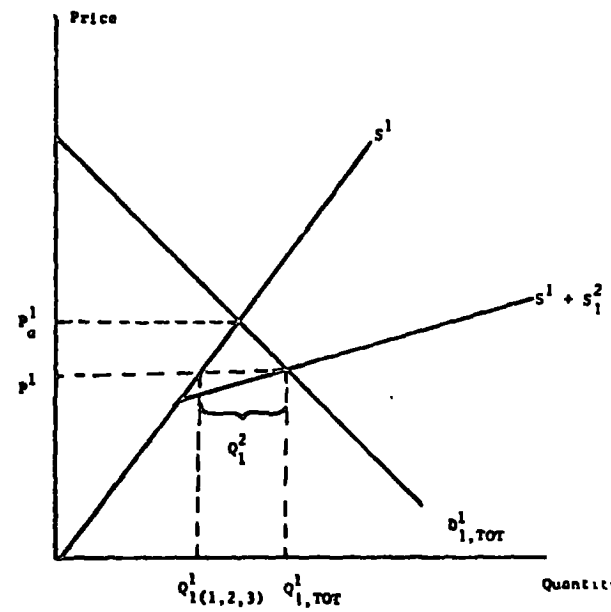


Figure 16. Efficient Allocation in HSU 1 Using Excess Supply

2. Water Transfer Costs

Economic feasibility is achieved when total economic benefits are equal to or greater than costs. Deviations from efficient allocation impose economic costs on society. Public decision-making may be judged by the criterion of the costs of deviations from economic efficiency.

Deviations from efficient allocation of water in Utah fall into two categories: Investments in transfer facilities that are made earlier than necessary; and institutional constraints which may restrict development of low-cost water supplies. The two categories will be treated separately for convenience.

Assume two available alternative sources of water yielding marginal cost curves: S^1_{sw} (surface water), and S^1_{gw} (groundwater). The total MC curve is a horizontal summation of the curves: $S^1 = S^1_{sw} + S^1_{gw}$. Further, assume a supply curve for water transferred from HSU 2 to HSU 1 (S^2_1).

The total available supply curve of water in HSU 1 is the horizontal summation of all three curves, termed $S^1 + S^2_1$. The next more expensive water provision is to be developed at a time when the present value of the benefits foregone by not developing are equal to or become greater than the costs of providing facilities. Not developing facilities results in a loss of benefits. As the demand increases over time, annual losses in benefits increase. When the present value of the losses equals investment costs, investment is indicated. Practically, optimum timing is difficult to determine, particularly if the S^2_1 curve shifts over time as a result of increased demand in the transferring area. -- By using these supply and demand curves, costs of inefficient public policies may be examined.

Institutional constraints may impose restrictions on use of locally available water. In that case, the use of alternative sources of water (such as water transfers) results in higher marginal costs to users. There is a concomitant loss in producers' and consumers' surplus for the given time period. As the total demand rises over time, the annual loss of producers' and consumers' surplus will increase. The surplus loss can be calculated as the difference between the integrals of the two supply functions, plus the difference between the integral of the demand function and that of the total supply function.

Each annual loss can be estimated and discounted to determine the present value of foregone economic benefits due to maintenance of restrictive institutions. No account has been taken of the losses suffered in the transferring HSU, whose producers must pay a higher price for in situ water as a result of increased demand. Present values of losses as discussed are, therefore, an underestimate of the total cost of institutional constraints.

PRWG 100-3, p. 9-10.

3. Mathematical Programming Model

Mathematical programming is chosen as the analytical technique because it is one of the more effective means of manipulating large numbers of variables and equations to achieve the optimization of some objective function. The solution of the mathematical programming problem yields a dual variable value which is the shadow price. This shadow price is equivalent to the Lagrangian multiplier, and represents the marginal cost or value of a given variable as it derives from the objective function.

The objective function is the net profit for the entire state. The objective function is maximized subject to the following limitations: The right-hand-side values establish limits to each constraint (maximums or minimums of the given constraints). Bounds are established on many of the variables. Coefficients in the constraint matrix fall into four categories:

1. Technical relationships of water development and distribution;
2. Productivity relationships between inputs and outputs in agriculture, including rotation requirements;
3. Input availabilities;
4. Prices of outputs and costs of inputs.

PRWG 100-3, p. 11.

4. Simplifying Assumptions

Several simplifying assumptions are made in order to construct a model that is both feasible and usable for research on Utah water allocations. Whether or not these simplifications are warranted is an arguable matter.

Demand for water is separated into M + I, wetland, and agricultural sectors.

Assumption 1. Municipal and industrial uses enter the model as fixed withdrawals projected to the year 2020. The fixity of M + I withdrawals is equivalent to a perfectly inelastic demand for water. The effect on total demand may be viewed as a shift of the vertical axis rightward to the water demanded by M + I users.

Assumption 2. Wetland consumption enters the model likewise at fixed levels as a perfectly inelastic demand.

As a result of Assumptions 1 and 2, only the VMP in the agricultural sector in each HSU determines efficient allocations within and between sectors.

The productivity of agricultural water is dependent upon relationships with other factors of production, such as land quality, cropping patterns, and frost-free growing season, for example. The model uses per acre crop production by land class by county within a given HSU as the appropriate production unit. Managerial ability, technological change, and market conditions determine profitability for any given farm. Some further simplifying assumptions are necessary to limit the model to a workable size:

Assumption 3. An average farm manager as projected for 1980 is assumed. This implies:

a. Yields on a given class of land of a given crop in a given HSU are the HSU averages for that class of land for 1980.

b. Inputs per unit of a given crop production are the average for the given HSU and land class, including labor, water, and other variables, and are utilized in fixed proportions projected for 1980. Variable and fixed input costs are identified, the former with amounts of crops grown, the latter with acreages of land in production. Both present and potential land developments are identified by class, county, and HSU.

c. Rotations of crops are the normal rotations for the HSU.

As a result, unit profitability and, therefore, the VMP is the same for each crop on a given land class in a given county within a given HSU. Agricultural productivity has shown increases in the past, and could well increase beyond 1980 as a result of technological and cultural improvements. There is some reason to believe that some productivity may fall as a result of restricted cultivation practices required by environmental quality limitations.

The model will over- or underestimate the value of transfers, depending upon these and other offsetting effects.

Assumption 4. Prices of agricultural products and costs of production inputs will change at the same relative rate, so that profitability of each crop on each land class in each county will remain constant over time, given the productivity levels.

Trends over time would indicate that agricultural product prices rise at a considerably lower rate than do costs of production. However, technological advancement in production has previously offset the relative increase in input prices. The model will under- or overestimate agricultural profitability, depending on the relative changes in input prices and technological advancement. It is implicitly assumed that average agricultural income and all other prices are constant and equal among all HSU's.

Assumption 5. Water delivery timing is irrelevant to its value. Often, in arid regions, late season water is considerably more productive and, therefore, more valuable, than early season water at the margin. However, the productivity of water in the model is an "average" marginal productivity over the growing season, so that the model overestimates the marginal value of spring water as well. Thus, the model may over- or underestimate the value of water transfers, depending on relative differences between each season's water and the model's average.

Assumption 6. Any new land developed will contain the same proportion of new classes (with the exception of class 5 land and above) as presently developed land.

The demand part of the model is essentially the application of the theory of the firm to the agricultural sector of each HSU, as if only one firm, with the specified constraints, were involved in production, and perfect competition existed in that market. Representing the sum of individual demand curves of all "approximately average" farmers, the HSU demand curve is open to all the objections to aggregating demand curves. The treatment of the production of one region as if it had no effect on market prices received or paid in other regions may be erroneous to some degree. Other demand assumptions may be made for purposes of parameterizations, population projections, alternative efficiencies, etc.

The supply part of the model is open to many of the same kinds of criticisms as the demand model. Average values, such as costs of surface water storage and water transfer facilities, are assumed for physical and economic variables in the various HSU's. Electrical power production costs have been deducted from development costs. Marginal costs used are residual costs charged to agriculture and M + I use. Included in the model are both surface and groundwater availabilities, return flows from agricultural use and effluents from M + I use, potential recharge capabilities, and concomitant costs.

The marginal cost of newly developed water is assumed to be constant for a given development practice, even though operation and maintenance costs might theoretically cause the curve to be upward or downward sloping over given intervals of quantity. When capacity of one facility or practice is reached, the marginal cost of water increases to the next more expensive alternative of supply, or becomes infinite if no further water can be developed. Thus, the model is upward sloping, but "stepped," and is an approximation of a continuing upward sloping supply function.

Assumption 7. All land surrounding present water delivery systems has been developed. Newly-developed land will incur costs commensurate with the development and delivery of new water. Presently-developed land can, however, use newly-developed water at costs net of new delivery costs.

PRWG 100-3, p. 11-13.

5. Water Allocation Optimization

Water allocation, within an HSU and between HSU's, is dependent on maximizing net agricultural returns for the entire state. Inter-basin transfers of water occur when the value of the marginal productivity in a given HSU is sufficiently high to pay the cost of water transfers, and earn an equal or higher net profit in the receiving HSU than in the providing HSU. As long as water is available for agricultural use, and a positive net profit is earned, water will be allocated to agriculture. If water availability is restricted, it will be allocated to the agricultural use(s) and HSU(s) from which the highest net profit can be earned.

An optimum solution to the programming model indicates the amount of each variable which is required to maximize state-wide profit from agriculture, given M + I and wetland requirements. A solution for any given level of M + I or wetland requirement can be achieved by making appropriate changes in coefficients, right-hand-sides, or bounds. Series of these changes can be simulated by parameterizations of appropriate variables. The model is used to generate optimal solutions for the projected changes in M + I requirements over time, and for alternative requirements for wetland consumption which represents water salvage potentials.

A simplified example of the model's construction and operation is presented below. Assume that two products, corn and alfalfa, are produced in an HSU with one county containing two land classes. Further assume two sources of water, surface and groundwater. Finally, assume a profit-maximizing objective function. The simplified equations are:

1. Profit = (Returns to alfalfa) + (Returns to corn) - (Average costs per unit of land + Variable costs per crops grown)

2. Alfalfa = (Tons of alfalfa produced from each land class)

3. Corn = (Tons of corn produced from each land class)

4. Variable costs of production = (Variable costs of harvesting, hauling, and water application per ton of product) x Tons of product

5. Average costs of production = (Costs of fertilizing, leveling, etc. per acre of crop) x Acres of crop

6. Water available to agriculture = (Total groundwater) - (Groundwater to M + I and to wetlands) + (Total surface water) - (Surface water to M + I and to wetlands)

7. Land available = (Land in class 1 used for each crop) + (Land in class 2 used for each crop).

The model determines the optimal amount of water used to maximize profit, given productivity of each crop on each land class, and given costs of production and water. If the water supply is restricted by M + I and wetland demands, so that water available for agriculture can only be obtained at high cost, then only the most productive land will be used for agriculture. As long as applications of water are profitable and water is available, production will occur with the most profitable crop being grown, restricted by any rotation constraints applied. The solution will yield the maximum

profit gained, given costs of production, availability and costs of water, and productivity of land by crops.

PRWG 100-3, p. 14.

6. Results from the Model

The model's solutions indicate that development of the Central Utah Project hinges on several alternative policies with respect to locally available water. Efficient development of the Ute Indian portion requires delay until after 2020. One portion of the water transfer system that does appear efficient at present is the Sevier Area. The transfer consists of water from HSU 8 transported to HSU 5 using slightly improved existing facilities. The transfer can be made at less cost than developing new locally available water.

The timing of the development of the Bonneville Unit depends on use of alternative locally available water sources. The following implications are drawn from the model's results:

1. Development of early stages of the Bonneville Unit is dependent on water availability in HSU 5. If salvage of water and use of the groundwater reservoir in HSU 5 is allowed up to levels at which groundwater mining occurs, the Bonneville Unit is not economically efficient until 2005 or 2010 for inflows to Great Salt Lake of less than 850,000 ac-ft/yr. For inflows of up to 1,014,000 ac-ft/yr, postponement of development to 1995 is indicated. With no salvage, low levels of importation are immediately indicated.

2. Development of the Bonneville Unit to full capacity is dependent on water availability in HSU 4. A "take-off" of demand for Bonneville Unit water is indicated when groundwater pumping including groundwater recharge reaches a maximum. Depending on inflows to Great Salt Lake

and water salvage, the "take-off" occurs near 1975, 2000, 2005, or 2015, with maximum capacity being reached about 20 years later.

3. The use of Bonneville Unit water in HSU 4 depends primarily upon the growth of urban demand. A comparison of importation timing and agricultural land indicates that Bonneville Unit water is sufficiently costly to be inefficient for new land development. Further, for every solution, only available groundwater is sufficiently cheap to provide for new agricultural water. Low cost recharge is utilized for M + I demands, and the residual groundwater storage is used for new agriculture.

4. For inflows to Great Salt Lake of 850,000 ac-ft/yr or more, and if present groundwater reservoir levels are to be maintained, full development of the Bonneville Unit is efficient by 2000 with salvage, by 1990 without.

5. Agricultural practices are limited to present land in HSU 5. It is unprofitable to develop new land with any source of water.

6. There exists a surplus of water in HSU 7 available until 2020 under all assumptions.

Costs of inefficient water allocations can be calculated either from foregone returns to investment, or from higher costs of supply.

PRWG 100-3, p. 20-22.

APPENDIX D. DEMAND FOR AGRICULTURAL WATER

Source:

The Demand for Agricultural Water in Utah, by Mark H. Anderson, John E. Keith, and Calvin G. Clyde, Utah Water Research Laboratory, College of Engineering, Utah State University, Logan Utah 84322.

PRWG 100-4 September 1973

Scope and Use

The State of Utah was divided into drainage regions, and irrigable land areas (by land classes) were estimated within each region. Water use factors, crop rotation constraints, production costs, yields, product prices, and costs of bringing new land into production were also estimated. These values were then used in a linear programming model to estimate a normalized demand (marginal value product) schedule for water to be used in agricultural production in each region. The amount of water made available to the production model for each region was varied so that the model created a shadow price (marginal value product) at each level. These were then combined to estimate the relationship between the quantity of water and its economic value (a demand schedule or function).

This model and technique appears applicable to any primarily or partly agricultural state for determining an optimum allocation of water resources to agricultural production.

APPENDIX D. DEMAND FOR AGRICULTURAL WATER

1. Theory

A demand curve for water is a schedule showing what a user is willing to pay (based on its productivity) for various quantities of water utilized over a period of time. The demand curve for water in agriculture is derived from the value of agricultural crop production which results from the application of an increment of water. Included is consideration of production functions for various crops, price of the crops, and price of other inputs such as land, labor, fertilizer, and capital, soil quality, farming methods, and level of technology used. No user will pay more than the net value of production, since to do so would yield diminished net return.

If water is a constraining input in agricultural production, then optimal allocation occurs when the returns to water itself are maximized and the marginal productivity of water is equal among every agricultural use and user. If this condition is not met, increases in total water productivity could be achieved by transferring some of the water resource from uses or areas of lower marginal productivity to those of higher productivity. Thus, knowledge of the demand function is crucial to decision-making.

Derivation of marginal productivity curves can be achieved using mathematical programming. Linear programming is adequate for this study. Solving the primal problem yields to combination of resources, given the constraints, that will lead to the greatest net benefit. Every linear programming model

has both a primal problem and a counterpart problem called the dual. If the primal problem maximizes output with a given cost outlay, the dual minimizes costs for the given product output. In the dual problem, values (shadow prices) are imputed to fixed facilities. The dual thus determines the shadow price or marginal value product of water used in irrigated farming. Total resource values from the dual solution equal maximum revenue from the primal solution.

The analysis assumes that water is the main constraining resource on production. The dual solution allocates the marginal value product to irrigation water and maximizes net returns to irrigation water, given water availability and rotation constraints. As water availability is reduced using parameterizations, fewer acres are irrigated, and the rotations changed to rotations which are less water intensive, so that the marginal value of irrigation water increases. Thus a demand schedule, or curve, is produced by the parameterizations.

PRWG 100-4, p. 1-2.

2. Assumptions

Demand curves are established for each of Utah's hydrologic study units (HSU's) using separate though compatible linear programming models. Some general assumptions apply to all these models:

1. Prices are the same for all models.
2. Input-output, output-output, and input-input relationships are assumed linear.

3. A firm water-right is assumed to exist. That is, the present use of water on presently-irrigated land must be met before water within a region can be released for new development.

4. The process of agricultural production can be divided into separate, independent activities.

5. Fractions of these production activities can be used.

6. Constant returns to scale and fixed proportions among inputs characterize each of these activities.

7. Projected requirements for water to be used in M + I activities must be met before water will be released for agricultural use.

8. No external economies or diseconomies exist.

9. The level of farm managerial ability is slightly above the present average to approximate 1980 conditions and is reflected in the yields and cost coefficients.

10. Yields for each land class are assumed constant within each county in each HSU.

PRWG 100-4, p. 2.

3. Rotation, Costs, Yields, Water Use

Basic rotation constraints are:

1. Alfalfa acreage $\geq$ Barley acreage
2. Barley acreage $\geq$ Nurse crop acreage
3. Alfalfa acreage $\geq$ 5(Nurse crop acreage)
4. Alfalfa + barley + nurse acreage $\geq$ 7(Sugar beet acreage)
5. Alfalfa + barley + nurse acreage $\geq$ 7(Corn silage acreage)

Many additional rotation limitations need to be applied.

Costs associated with production activities are divided into average and variable components, which do not correspond to typical economic definitions, but are used to clarify input information. Average costs are fixed once the decision is made to grow a certain crop. Examples are fixed overhead, seed, and plowing. Variable costs vary with amount of output, the number of cuttings, or the number of irrigations. Variable costs are assumed to be the same throughout the state, while average costs may be slightly different due to differences in production activities.

Curtailment of fertilizer and pesticide use due to environmental concern could cause actual productivity to be lower than projected. It is also possible that improved technology and farming methods could lead to higher-than-projected production.

From consumptive irrigation water requirements for every crop in each county in each HSU, should be deducted estimated soil moisture storage and effective precipitation. Consumptive use may be transformed into withdrawals through the use of irrigation system efficiency factors. Use of a single water level and yield for crops other than alfalfa is indicated. Alfalfa has two water and yield levels corresponding to different numbers of cuttings per growing season. Alfalfa, nurse crop, and corn are estimated to consume 0.4 ac-ft of water per irrigation; barley and pasture, 0.3 ac-ft; sugar beets, 0.25 ac-ft. The number of irrigations may be calculated by dividing the consumption by water use per irrigation for each crop and HSU.

PRWG 100-4, p. 5-7.

4. Results

Parameterizations yield stepped functions relating quantity of water used to shadow price (willingness to pay, or demand). Each change in activity mix and acreage affects the shadow price. The relationships are likely to appear as downward-sloping demand curves. Demand curves can be presented for each HSU for presently-irrigated land, and for new land with or without restrictions in regard to maintenance of land class proportionality.

There is evidence that shortness of the growing season in the northern areas of Utah is offset by soil productivity, while the much longer growing season in the south does not compensate for lack of good soil. Transfers of water from marginal land now in production to better land with only partial water supply could be economically efficient.

The maximum value for water on new land appears to be about $20 per ac-ft, with $6 - $8 per ac-ft being the value ascribed to any large-scale developments. It is also clear that if new land is developed on a best-land-first basis, much less water will be demanded than if selectivity is not encouraged.

PRWG 100-4, p. 7.

APPENDIX E. PROFESSIONAL PAPERS BY MEMBERS OF THE UTAH WATER RESEARCH LABORATORY

Sources

1. "Optimal Allocation of Water Resources in Utah," by Calvin G. Clyde and Alton B. King, in Journal of the Hydraulics Division, ASCE Proceedings, Vol. 99, No. HY10, October 1973, pp. 1777-1794.

2. "Jordan River Basin Water Resource Allocations: A Systems Analysis Approach," by John E. Keith and Jay C. Andersen, in The Great Salt Lake and Utah's Water Resources, Proceedings of the First Annual Conference of the Utah Section, American Water Resources Association, Held in Salt Lake City, Utah in November 1972, pp. 169-179.

3. "A Systems Analysis Approach to Water Resource Allocations in Utah," by John E. Keith and Jay E. Andersen. Unpublished paper presented at a meeting of The Institute for Management Science, Salt Lake City, Utah, January 1973.

4. "Determining Economic Costs of Economically Non-Optimal Public Policy," by John E. Keith and Jay C. Andersen. Unpublished paper presented at the Southwest Section of the AAAS, in Lubbock, Texas, March 1973.

APPENDIX E. PROFESSIONAL PAPERS BY MEMBERS OF THE UTAH WATER RESEARCH LABORATORY

1. Optimal Allocation of Water Resources in Utah

This professional paper lists the 10 hydrologic study units (HSU's) into which the state has been divided, as well as Utah's principal available water resources. One of the state's greatest sources of undeveloped water is in the Upper Colorado River Basin, which is separated from the most significant population growth areas by the Wasatch Mountains. Because of that separation, much of Utah's share of Colorado River water flows out of the state unused. The Central Utah Project of the U.S. Bureau of Reclamation is to transfer a sizable portion of Utah's share of Colorado water to the Great Basin.

Seasonal and long-term storage requirements are established for probabilities of 75% to 95% of meeting the draft with no deficiency. The potential for groundwater recharge is evaluated.

The allocation model is described as a mathematical model used to study optimal allocation of water resources within the State of Utah. The linear objective function to be minimized is an expression for the total dollar cost of meeting the demand for water in Utah. The variables are made up of alternative allocations which may combine to form the solution to the problem. Each variable has an associated cost coefficient reflecting the cost of allocating one ac-ft/yr to the given alternative. The objective function thus represents the total cost for allocation of Utah's water resources in dollars per year.

Six types of constraints and eight categories of variables are spelled out. The allocation pattern depends on (1) the intended uses (demands); (2) the amount of excess water available above demand; (3) geographic limitations; and (4) presently structured water systems.

Results from the model include three outputs: (1) the optimum solution to the linear programming problem; (2) results of post-optimal analysis; and (3) findings resulting from manipulation of coefficients, right-hand-side values, and variable bounds. The optimum value of the objective function represents the minimum annual cost of development of new facilities to meet specified demands for water under a particular set of assumptions. For the base year, the cost represents the yearly cost of developing new facilities to eliminate groundwater mining. For subsequent years to 2020, additional developments will be needed.

Shadow prices of resources are determined from the solution of the dual of the linear programming model. The economic interpretation of the dual property of linear programming lies in the concept that resource allocation and pricing are two aspects of the same problem. The optimal solution to the dual problem gives the values of the dual variables, referred to as shadow prices, which indicate the rate at which costs increase (or decrease) for a corresponding increase (or decrease) in the amount of a given resource. Parametric analysis is a procedure for generating new optimal solutions from an original optimal solution, while allowing one or more parameters (coefficients or constraints) to vary systematically over a specified range of values. The parameterized right-hand-side values are estimated as time passes from the base year to the year 2020. Other right-hand-side values, such as water demand, can also be varied, to represent alternative projections.

The effects of changing irrigation efficiency, groundwater policy, inter-basin transfer limits, changing growth projections, etc. can likewise be determined through parametric analysis. Even changes in the probability level of having sufficient storage to supply the required draft, and changes in the policy of maintaining the level of Great Salt Lake, can be calculated.

Results of the study comprise the following: A mathematical programming model, comprising 338 variables and 204 constraints, has been solved by the simplex method, yielding a least-cost allocation of water within Utah. The model is comprehensive and includes all uses, all areas, all sources, and all transfers. Water allocation has been optimized for various sets of assumptions. Sensitivity analysis pinpoints critical cost coefficients; parametric analysis tests the effects of changing constraints. Manipulations of the model show how the allocation is affected by operating rules, legal policies, and political and social limitations.

"Optimal Allocation of Water Resources in Utah," by Calvin G. Clyde and Alton B. King, in Journal of the Hydraulics Div., ASCE Proceedings, Vol. 99, No. HY10, October 1973, p. 1777-1794.

2. Jordan River Basin Water Resource Allocations: A Systems Analysis Approach

A linear programming model formulated for the State of Utah is applied to the optimum allocation of water resources in one of ten hydrologic study units, namely the Jordan River Basin (HSU 4).

Demand schedules for agricultural water are derived from the value of the productivity of incremental units of water in agricultural applications. Supply schedules are developed from the costs of providing incremental units of water to agriculture from local sources and by inter-basin transfers. Supply and demand curves are compared for given levels of inflow to the Great Salt Lake to determine economically efficient allocations. Consideration is given the proposed Bonneville Unit of the U.S. Bureau of Reclamation's Central Utah Project. Inflows to Great Salt Lake and water salvage potentials are shown to be critical to the timing of inter-basin transfers of water for agricultural purposes.

"Jordan River Basin Water Resource Allocations: A Systems Analysis Approach," by John E. Keith and Jay C. Andersen, in The Great Salt Lake and Utah's Water Resources, Proceedings of the First Annual Conference of the Utah Section of the American Water Resources Association, Held in Salt Lake City in November 1972, p. 169-179.

3. A Systems Analysis Approach to Water Resource Allocations in Utah

This paper attempts to analyze the economic aspects of social welfare accruing from alternative water allocation policies and decisions.

The schedule relating successive increments of resource provided with their marginal costs is the supply schedule. The schedule relating marginal revenue with successive increments of resource use is the demand schedule. The superimposition of the demand schedule on the supply schedule yields the economically efficient allocation of the resource. The last unit of resource applied costs precisely the amount that it produces in revenue.

If public allocation policies are such as to cause supply to be unequal to demand, then the loss of revenue, or the extra cost, is a social cost. This social cost is measurable.

The model used to optimize water allocation in Utah comprises 3700 variables in 1500 relationships identified as important to demand for water. About 800 variables in 700 relationships represent the water supply. When these are totaled, they result in 4,500 variables, 2,200 equations, and 9,900,000 coefficients. The objective function is maximum net profit subject to constraints.

While results of the statewide model have not been tabulated, analysis of the Jordan River Basin has been completed.

"A Systems Analysis Approach to Water Resource Allocations in Utah," by John E. Keith and J. C. Andersen. Unpublished paper presented at a meeting of The Institute for Management Science, Salt Lake City, Utah, January 1973.

4. Determining Economic Costs of Economically Non-Optimal Public Policy

The purpose of this professional paper is to determine the penalty cost that would be incurred by water users as a result of the implementation of various public policies at variance with economic efficiency in the allocation of Utah's water resources.

Public Policy is often made to achieve goals other than economic efficiency; that is, public policy frequently causes shifts in resources from the optimal or most efficient allocation which would result in a market. A methodology is needed for measuring the costs to society in losses of production and higher costs of inputs resulting from such policy.

Assuming a perfectly competitive market, the economically efficient allocation of a given resource to alternative uses is determined by equating the marginal cost of producing the resource (or supply) with the marginal value produced from application of that resource to alternative production possibilities (or demand). Public policy may alter such an efficient allocation in several ways, some of which might be: (1) taxes levied on, or subsidies granted to, producers, which alter the value of the marginal product (VMP); (2) taxes levied or subsidies granted against resource use, so that the supply or marginal cost curve is shifted; and (3) restrictions on amounts which can be supplied, such as allowing only a given amount of the resource to be produced.

Taxes and subsidies are approximations to economic costs of altering allocations, since the allocation will change only insofar as VMP's and marginal costs change in the market. However, restrictions of use, and

investing public monies in high-cost production facilities have costs which are not so easily calculable. Losses in producers' and consumers' surplus result from such restrictions, and are indicative of welfare changes.

The consumers' surplus is defined as the amount which consumers would have been willing to pay less the market price. This is the area which lies between the total demand curve and the price line in the supply and demand graph. Producers' surplus, or rent, is the difference between the actual cost of producing a quantity of goods and price received. This area lies between the supply (marginal cost) curve and the price line in the same graph.

Restricting the supply results in a net loss of both producers' and consumers' surplus from the unrestricted equilibrium. The area between the demand and supply curves for the restricted and unrestricted equilibrium quantities measures the welfare loss from decreased use.

A mathematical program which optimizes an objective function presents a methodology for determining the costs of restrictive policies. The steps to creating an appropriate mathematical programming problem are:

1. Develop a mathematical programming model representative of the values of marginal product for alternative levels of available resources. A demand curve can be generated by maximizing net returns to various levels of resource availability, using the dual solution's shadow prices (equivalent to the Lagrangian multiplier) as the price variable.

2. Develop a mathematical programming model representative of the costs of supplying the resource. Generate a supply function by minimizing costs for alternative resource productions, again using the dual solutions to calculate shadow prices.

3. Develop supply and/or demand functions modified to represent alternative public policies which are of interest. For a given program, parameterizations will yield solutions for each basis change, so that demand and supply curves are "lumpy" (stepped or discontinuous).

4. Link supply and demand models using the resource availability variables and a "net profit" objective function. The solution is the optimal or efficient allocation for a given set of policies or assumptions. Temporal changes in efficient allocations can be determined by altering various components of the programming problem consistent with projected future conditions. The competitive solution, using the supply and demand functions which represent no restrictions, is assumed to be the efficient, or optimal, market-determined allocation.

5. Compare solutions for alternative policies, and calculate changes in losses in producers' and consumers' surplus, using the differences between supply and/or demand curves for the market-efficient and alternative models.

A linear programming model for the allocation of water in the State of Utah is solved, showing the least-cost method of meeting demand for M + I, agricultural, and wetland water, from 1965 to 2020. The optimum solution states that, with a specified degree of salvage and an inflow of no more than 850,000 ac-ft/yr into the Great Salt Lake, water transfer from the Colorado River into the Great Basin will be needed in about the year 2020, investment being required in 2005.

If the Bonneville Unit of the Central Utah Project, which is designed to implement the transfer, is initiated now, returns on the investment will be foregone until 2005. The investment is estimated at $130 million, the amount contracted by the Central Utah Water Conservancy District. Annual returns to

that investment, after minor adjustments, are, at an interest rate of 5%, $5,375,000; of 7%, $7,975,000; of 12%, $14,475,000. Present worth of these annual foregone returns is $84,936,000 at 5%, $100,860,000 at 7%, and $117,407,000 at 12%.

If no salvage is assumed, the time of investment is moved to the year 1990. Present worths are $62,834,000 at 5%, $80,220,000 at 7%, and $104,929,000 at 12%.

If public policy restricts use of alternative supplies, loss of benefits consisting of foregone producers' and consumers' surplus can be calculated by multiplying quantity differentials by marginal cost differentials for various assumptions.

Since no imported water is efficient for development of new irrigation projects, foregone net returns to alternative water sources are used to estimate the losses in producers' and consumers' surplus accruing to new agriculture.

Annual costs of institutional constraints for alternative sources and appropriate quantities are, beginning in 1980, $1,232,000 (M + I) and $1,599,000 (Agriculture); beginning in 1990, $1,394,000 (M + I) and $1,306,500 (Agric); beginning in 2000, $1,568,000 (M + I) and $871,000 (Agric); and beginning in 2010, $1,643,000 (M + I) and $0 (Agric).

Present value of the loss in consumers' and producers' surplus to M + I uses is $16,012,000 (at 5%), $10,462,000 (at 7%), and $4,329,000 (at 12%). Present value of losses of net benefits to new irrigation is $11,959,000 (at 5%), $8,660,000 (at 7%), and $4,063,000 (at 12%).

"Determining Economic Costs of Economically Non-Optimal Public Policy," by John E. Keith and Jay C. Andersen. Unpublished paper presented at the Southwest Section of the AAAS, in Lubbock, Texas, March 1973.

THIRD PART

WATER QUALITY

VII

PLANNING FOR WATER QUALITY

VII. PLANNING FOR WATER QUALITY

Source:

Evaluation of Quality Parameters in Water Resource Planning -- A State-of-the-Art Survey of the Economics of Water Quality, by Eric D. Bovet, Consultant.

IWR Contract Report 74-13 December 1974

Scope and Use

The objective of this report was to assist Corps of Engineers planners in giving fuller recognition to water quality characteristics, quantifying them, and integrating them more meaningfully into their water supply plans and project designs. The report reviews surface water resources and their contaminants, water quality indices and standards, water user tolerances, water quality models, technology and costs of raw and waste water treatment, water quality benefits, and optimization of water supply purification, allocation, and waste water renovation.

Cost estimating data, presented on pages VII-14/17, 20/25, and 27/38, are taken from Black & Veatch: Standardized Procedure for Estimating Costs of Conventional Water Supplies, an unpublished report sponsored by the U.S. Office of Saline Water, 87 pages, 1963. Cost figures can be updated by applying a trending factor. For trending construction costs (except land and rights), the trending factor is the Engineering News Record Building Cost Index divided by 584, the index for the base period of 1963. For trending O + M costs, Table C-1, "Transportation and Public Utilities," in the "Monthly Labor Review" published by the U.S. Bureau of Labor Statistics, may be used. The trending factor is the current average hourly earnings of production workers in "Water, Steam and Sanitary Systems," divided by $2.37, the average hourly earnings in this category for January 1963. O + M costs of transmission pipelines and raw water storage need not be trended, since they are tied to trended construction costs.

A parallel series of costs, borrowed from IWR Contract Report 74-3, appears in Section III. Those costs are more recent and may be preferable for individual cost centers, but the method pursued in the Black & Veatch cost estimating procedure is probably superior through its completeness where total costs of providing a new municipal water supply are desired.

Presented in Report 74-13 are a number of economic techniques for: taking into account and quantifying water quality parameters, utilizing available water quality enhancement technology, determining the value of water quality, assessing damages attributable to the use of water of inferior quality, blending and allocating water resources of varying quality, and maintaining instream water quality levels. A broad range of water quality problems are reviewed, and examples given of the application of optimization techniques in arriving at alternative solutions.

The planner may find in the passages compiled in Section VII the gist of a number of useful approaches and methods for dealing with water resource situations in which water quality is a non-negligible or even an important factor. Where the information appears too succinct, he is invited to consult the document itself. Again, each excerpt is carefully referenced by chapter and page number.

Topics in Section VII are grouped under four subsections:

A. Water Quality Concepts
B. Water Quality Enhancement Technology and Cost
C. Water Quality Benefits
D. Water Quality Optimization

IV. PLANNING FOR WATER QUALITY

A. Water Quality Concepts

1. Water Quality Classifications

Water quality can be classified by at least three criteria: by contaminants, by uses, and by treatment processes required to remove contaminants.

(a) Water quality classification by contaminants begins with a grouping of like characteristics into types of contaminants such as: physical, biological, chemical, toxic, thermal, and radioactive; to these should be added contaminants generating color, odor, and taste. Under each of the seven types, individual parameters should be listed, and the concentration of each parameter, noted. Variations in time and space should be recognized and recorded.

It is not safe to average parameters or measure water quality by types of contaminants only. A single parameter, such as viruses, arsenic, DDT, etc. can be critical. But this makes water quality classification by contaminants unwieldy. For, contaminants exceed sixty in number and the concentration of each contaminant present must be measured and recorded. It also makes it difficult to compare the relative merits of waters of different quality.

(b) Water quality classification by uses is an attempt to eliminate irrelevant water quality parameters. A requisite is that tolerances to each parameter, in terms of concentrations, be known -- or that safe water quality standards be established for each use. By means of tolerances or

standards, one may discriminate, for each use, between critical and non-critical parameters. For example, high mineral content of water is critical in its use as domestic water supply; but in a recreational use such as swimming, even sea water is tolerated. By ignoring parameters which, in a given water use, are non-critical, the classification of water quality may be simplified while being made more meaningful. Waters of different quality can also be compared more readily when related to a given water use.

(c) Water quality classification by treatment cost constitutes another substantial simplification. Water quality is measured by the cost of bringing it to levels acceptable in given water uses. This presupposes that, in addition to tolerances or standards in various water uses, the effects of water treatment, in percent removal of critical parameters, and the cost of such treatment, be known. Most water treatment processes are capable of removing more than one parameter, although possibly not by the same percent. By cumulating appropriate treatment processes, water quality can always be raised to any desired level of purity.

Parameters that are critical in a given water use and yield to a given treatment can be combined into groups, thus simplifying considerably the classification of water quality. There exist but a limited number of water treatment processes, perhaps twenty in all. Their combinations increase the number of possible treatment plant configurations. However, if water quality is to be classified by treatment cost in a given water use, costs of all required processes (for a specified plant size, and in dollars of a specified year) are simply aggregated into a total amount. The classification of waters of every description can thus be reduced to the measurement of a single dimension per water use. Waters of widely differing quality can

readily be compared. And, for each water use, the aggregate treatment cost, upon identification with an appropriate cost bracket, can be used as a water quality index. 74-13: II, 1-3

2. Water Quality Indices

The need for judging over-all water quality in terms of individual parameters and their concentrations, particularly critical parameters, needs to be kept in mind. One type of quality index, the cost of bringing water quality to desirable levels in given uses, is described above. Other water quality indices have been proposed in the literature. Each may serve a limited purpose. None is ideal.

Syracuse University's Pollution Index (P. I.) comprises 14 widely monitored water quality factors. They are: Temperature, color, turbidity, bacteria, total solids, suspended solids, total nitrate, alkalinity, hardness, chloride, iron, manganese, sulfate, and DO.

For each factor, the measured value is divided by the recommended ceiling for each use, the resulting ratio for each factor and each use indicating the need for treatment if the value exceeds one. For each use, the mean impairment of the 14 factors is computed, but the maximum impairment is used along with the average. The pollution index is obtained by taking the square root of one-half the sum of the square of the mean impairment and the square of the maximum impairment.

The restricted number of quality parameters included in the index is its chief weakness. However, the statistical technique is equally well

applicable to any desired number of quality factors. Time variations should be included.

Other indices have been developed. Those failing to recognize tolerance levels in various water uses are deemed inadequate. 74-13: II, 3-6

3. List of Water Uses

a. By Public Water Supply Customers

Residential or domestic use
Commercial use
Industrial use (of public supply)
Institutional use
Firefighting use

b. By Industry (not using public supplies)

Boiler make-up use
Processing use
Product use
Cooling use
Sanitary use
Firefighting use

c. By Recreation Seekers

Water-contact activities (swimming, diving, skiing)
Water-based activities (boating, fishing)
Water-related activities (picnicking, camping, hunting)
Use of park facilities (swimming pool use, drinking, cooking)

d. By Plants and Animals

Phytoplankton, water plants
Zooplankton
Macroinvertebrates, fish, reptiles
Waterfowl, other aquatic animal life
Wildlife

e. By Waste Producers

f. By Seekers of Well-Being and Aesthetic Enjoyment

g. By Regional Planners 74-13: II, 7-10

4. Contaminants Added Through Utilization of Public Water Supply

	Parameter	Average Increment per Use (mg/l)
Inorganics		
Cations:	Na^+	16
	K^+	10
	NH^+	15
	Ca^{++}	18
	Mg^{++}	6
Anions:	Cl^-	74
	NO_3^-	10
	NO_2^-	1
	HCO_3^-	100
	CO_3^{--}	--
	SO_4^{--}	28
	SiO_3^{---}	15
	PO_4 (Total)	24
	PO_4^{---} (Ortho)	25
Others:	Hardness (as $CaCO_3$)	79
	Alkalinity (as $CaCO_3$)	81
	TDS	320
	pH (in H ions)	0.6
Organics	BOD	16
	COD	87
	MBAS*	6.4

* MBAS = Methylene Blue Active Substances, formerly restricted to anionic detergents, now including chemically related materials.

74-13: II, 12-13

5. Raw Water Quality Criteria

Parameter	Unit	NTAC 1968 (a) Mandatory	Desirable
Physical Parameters			
Color	PCU	75	10
Odor	TON	Narr.	V.abs.
Temperature	Degrees C	Narr.	Narr.
Turbidity	JTU	Narr.	V.abs.
Biological Parameters (b)			
Coliforms	#/100 ml	10,000	100
Focal Coliforms	#/100 ml	2,000	20
Inorganic Chemicals			
Alkalinity ($CaCO_3$)	mg/l	Narr.	Narr.
Ammonia (as N)	mg/l	0.5	0.01
Arsenic	mg/l	0.05	Abs.
Barium	mg/l	1.0	Abs.
Boron	mg/l	1.0	Abs.
Cadmium	mg/l	0.01	Abs.
Chloride	mg/l	250	25
Chromium, hexavalent	mg/l	0.05	Abs.
Copper	mg/l	1.0	V.abs.
Dissolved oxygen	% satur.	-	Near
monthly mean	mg/l	4+	-
individual sample	mg/l	3+	-
Fluoride	mg/l	Narr.	Narr.
Hardness	mg/l	Narr.	Narr.
Iron, filterable	mg/l	0.3	V.abs.
Lead	mg/l	0.05	Abs.
Manganese, filterable	mg/l	0.05	Abs.
Nitrates, nitrites	mg/l	10	V.abs.
pH range	H ions	6 - 8.5	Narr.
Phosphorus	mg/l	Narr.	Narr.
Selenium	mg/l	0.01	Abs.
Silver	mg/l	0.05	Abs.
Sulfate	mg/l	250	50
TDS (filt. residue)	mg/l	500	200
Uranyl ion	mg/l	5	Abs.
Zinc	mg/l	5	V.abs.

Note: Narr. = Narrative; Abs. = Absent; V.abs. = Virtually absent.

Parameter	Unit	NTAC 1968 (a) Mandatory	Desirable
Organic Chemicals	mg/l		
BOD		-	-
CCE (Carbon-chloroform extract)		0.15	0.04
COD		-	-
Cyanide		0.20	Abs.
Herbicides		0.1	Abs.
MBAS (Methylene blue active substances)		0.5	V.abs.
Oil & grease		V.abs.	Abs.
Pesticides			
Aldrin		0.017	Abs.
Chlordane		0.003	Abs.
DDT		0.042	Abs.
Dieldrin		0.017	Abs.
Endrin		0.001	Abs.
Heptachlor		0.018	Abs.
Heptachlor epoxide		0.018	Abs.
Lindane		0.056	Abs.
Methoxychlor		0.035	Abs.
Organic phosphates & carbamates		0.1	Abs.
Toxaphene		0.005	Abs.
Phenols		0.001	Abs.
Radioactivity (c)	pc/l		
Gross beta		1,000	100
Radium 226		3	1
Strontium 90		10	2

(a) The nature and extent of the expected raw water treatment are defined in the "Green Book." Simple processes will produce drinking water of acceptable quality.

(b) Microbiological limits are monthly arithmetic averages based on an adequate number of samples. Total coliform limit may be relaxed if fecal coliform concentration does not exceed the specified limit.

(c) The unit for radioactivity is the picocurie, or micro-microcurie, per liter, expressed by pc/l.

74-13: II, 19-21

6. Treated Water Quality Standards and Goals

Parameter	Unit	PHS 1962 Mand. (a)	PHS 1962 Desir. (b)	AWWA 1968 (c)	EPA 1973 Health	EPA 1973 Esth.
Physical Parameters						
Color	PCU	-	15	3	-	15
Non-filt. solids	mg/l	-	-	0.1	-	-
Odor	TON	-	-	None	-	3
Taste	-	-	-	Unobj.	-	-
Turbidity	JTU	-	1	0.1	1	-
Biological Parameters						
Coliforms filter	#/100 ml	1 or 4 (d)	-	None	1 or 4 (d)	-
ferment	#/100 ml	-	-	None	-	-
Fecal coliforms	#/100 ml	-	-	None	-	-
Macroscopic organisms	#	-	-	None	-	-
Inorganic Chemicals						
Alkalinity ($CaCO_3$)	mg/l	-	-	(e)	-	-
Aluminum	mg/l	-	-	0.05	-	-
Arsenic	mg/l	0.05	0.01 (f)	-	0.1	-
Barium	mg/l	1.0	-	-	1	-
Boron	mg/l	5.0(g)	1.0(g)	-	-	-
Cadmium	mg/l	0.01	-	-	0.010	-
Calcium	mg/l	-	-	-	-	-
Chloride	mg/l	-	250	-	-	250
Chromium, hexavalent	mg/l	0.05	-	-	0.05	-
Copper	mg/l	-	1.0	0.2	-	1
Corrosion	mg/sq cm	-	-	5.00 (h)	-	-
Fluoride	mg/l	(i)	(i)	-	(i)	-
Hardness	mg/l	-	-	80 (j)	-	-
Incrustation	mg/sq cm	-	-	0.05(k)	-	-
Iron, filter	mg/l	-	0.3	0.05	-	0.3
Lead	mg/l	0.05	-	-	0.05	-
Magnesium	mg/l	-	-	-	-	-

Parameter	Unit	PHS 1962 Mand. (a)	PHS 1962 Desir. (b)	AWWA 1968 (c)	EPA 1973 Health	EPA 1973 Esth.
Manganese, filterable	mg/l	-	0.05	0.01	-	0.05
Mercury	mg/l	-	-	-	0.002	-
Nitrates & nitrites (N)	mg/l	-	45	-	10	-
Selenium	mg/l	0.01	-	-	0.01	-
Silver	mg/l	0.05	-	-	0.05	-
Sulfate	mg/l	-	250	-	-	250
TDS, filt. residue	mg/l	-	500	200	-	-
Zinc	mg/l	-	5	1.0	-	5
Organic Chemicals	mg/l					
CAE		-	-	-	3.0	-
CCE		-	0.2	-	0.7	-
Cyanide		0.2	0.01	-	0.2	-
Herbicides						
2, 4-D		-	-	-	0.02	-
2, 4, 5-TP (Silvex)		-	-	-	0.03	-
MBAS		-	0.5	-	-	0.5
Mineral Oil		-	-	-	-	-
Pesticides		-	-	-		-
Aldrin					0.001	
Chlordane					0.003	
DDT					0.05	
Dieldrin					0.001	
Endrin					0.0005	
Heptachlor epoxide					0.0001	
Lindane					0.005	
Methoxychlor					0.1	
Organic phosphates and carbamates (parathion)					0.1	
Toxaphene					0.005	
Phenols		-	-	-	-	-
Radioactivity (1)	pc/l					
Gross alpha		-	-	-	-	-
Gross beta		1,000 (m)	-	100	-	-
Radium 226		-	3	-	-	-
Strontium 90		-	10	-	-	-

Unobj. = Unobjectionable

(a) If the concentration of any of these constituents is exceeded, the further use of this water for drinking and culinary purposes should be evaluated by the appropriate health authority because water of this quality represents a hazard to the health of consumers.

(b) If the concentration of any of these constituents is exceeded, a more suitable supply or treatment should be sought.

(c) For all health-related constituents not stated herein, these goals shall require complete compliance with all recommended and mandatory limits contained in current USPHS Drinking Water Standards. Unless other methods are indicated, analyses shall be made in conformance with the latest edition of Standard Methods for the Examination of Water and Wastewater.

(d) Water quality fails the standard if:
(1) arithmetic average of samples collected is greater than 1 per 100 ml;
(2) two or more samples (5% or more if more than 20 are examined) contain densities more than 4/100 ml.

(e) Alkalinity should not change by more than 1 mg/l (decrease or increase in distribution system, or after 12 hours at 130°F. in a closed plastic bottle, followed by filtration).

(f) Although the recommended arsenic concentration is 0.01 mg/l, because of interferences in some waters, the concentration of arsenic was only determined to be less than 0.03 mg/l. For the purposes of this study, these waters were considered not to exceed the recommended standard.

(g) Proposed for inclusion in the Drinking Water Standards.

(h) Loss by corrosion of galvanized iron by coupon tests.

(i) Public Health Service limits are as follows: Temperatures shown for fluoride concentrations are annual average maximum day temperatures for 5 years or more.

Temperature	Limits (mg/l)	
	Mandatory	Desirable
50.0 - 53.7° F	2.4	1.7
53.8 - 58.3° F	2.2	1.5
58.4 - 63.8° F	2.0	1.3
63.9 - 70.6° F	1.8	1.2
70.7 - 79.2° F	1.6	1.0
79.3 - 90.5° F	1.4	0.8

EPA's limits are, for temperatures of 65° F or less, 1.5 mg/l; 66 - 79° F, 1.3 mg/l; 80° F or over, 1.2 mg/l.

(j) A balance between deposition and corrosion characteristics is necessary; a level of 80 mg/l seems best, generally, considering all the quality factors; however, for some supplies, a goal of 90 or 100 mg/l may be deemed desirable.

(k) By 90-day coupon tests on stainless steel.

(l) The unit for radioactivity is the picocurie, or micro-microcurie, per liter, expressed by pc/l.

(m) Acceptable in water in the known absence of strontium 90 and alpha emitters.

74-13: II, 22-25

7. Water Quality Models

Twenty-three Water Quality Models are reviewed from the standpoint of their parameters and simulation capability. Seven of these models are evaluated in greater depth because of their inclusiveness.

74-13: III, 4-18

B. Water Quality Enhancement Technology and Cost

1. Natural Instream Reoxygenation

The reaeration coefficient has been formulated and calculated by a number of authors. One formula that seems fairly accurate as a method of prediction covers a wide range of flow scales, and is rather insensitive to variations in basic data. The equation reads:

$$k_2 = 0.000125 \left(1 + F^{\frac{1}{2}}\right) \frac{u^*}{h}, \quad \text{where:}$$

k_2 = Reaeration coefficient

F = Froude number = $\frac{\bar{u}}{\sqrt{gh}}$

u^* = shear velocity

h = depth

g = acceleration due to gravity

u = velocity in a given direction

A mechanized instream aerator has been used to determine the natural reaeration coefficient. Operated in a sinusoidal fashion with a known amplitude and frequency, it can be used as a boundary condition in conjunction with a detailed one-dimensional, unsteady-state Streeter-Phelps model of a polluted river. The use of single-station mechanical aerators in polluted rivers appears as a promising new water quality monitoring device.

The natural reaeration coefficient, for medium-sized streams under summer-flow conditions, varies by a factor of 4. 74-13: IV, 1-4

2. Artificial Instream Reaeration

Natural self-purification can be accelerated or intensified by various artificial aeration devices. Whether artificial aerators are considered as substitutes for, or merely as adjuncts to, waste treatment facilities, they have the great advantage of operating on all collective wastes, including diffuse natural outfalls -- not just individual recorded outfalls.

a. Mechanical surface aerators, sold with a guaranteed efficiency of 4 lbs. of DO added per shaft-HP per hour of operation, usually have a much lower actual performance. Efficiency increases slightly with flow velocity. Their costs are:

Costs of Electric Mechanical Surface Aerators

Units in Configuration	Horse-Power per Unit	Horse-Power per Configuration	Capital Costs $	Total Annual Costs $
3	75	225	132,000	41,500
6	75	450	263,000	72,900
9	75	675	391,000	102,000

b. Submerged air diffusers aerate by releasing fine bubbles. Subject to clogging, they have not come into general use. Oxygen transfer rates per shaft-HP per hour equal about two-thirds of those of mechanical surface aerators. Their costs follow:

Costs of Electric or Diesel Diffusion Aerators

Units in Configuration	Horse-Power per Unit	Horse-Power per Configuration	Capital Costs $	Total Annual Costs $
4	80	320	198,000	52,000
8	80	640	395,000	91,000
12	80	960	587,000	128,000

Unless a very complete BOD study is made, the spacing of aerator sites might be at one-mile intervals. When compared with the cost of treatment of individual effluents, artificial reaeration costs may equal about one-third.

c. Gravity-flow aerators consist of washboard-type river bed coverings made of transversely corrugated galvanized iron, with a sinusoidal characteristic of 1.25 inch amplitude. Oxygen transfer was found to be a function of the angle of inclination, but not of flow rate. The cascade board appears as an aeration method deserving wide-spread consideration. It would have minimum operating costs and would not impede navigation. 74-13: IV, 4-7

3. Gravity Quality Storage Reservoirs

With passage of the 1972 Amendments (PL 92-500) of the Federal Water Pollution Control Act, storage reservoirs may no longer be built or used for water quality enhancement as a substitute for waste water treatment at the source. Low flow augmentation will, nevertheless, continue to have beneficial effects on water quality.

Benefits of flow augmentation for water quality may be measured by the cost avoided of downstream collective waste treatment. That cost increases rapidly in the upper ranges of BOD removal. When quality storage construction costs can be shared with one or more additional purposes, and DO standards require high BOD removal, flow augmentation may well be competitive with other water purification methods. The quality of the mixed water is a function of the reservoir layer from which the incremental water is removed. If the additional flow is drained from lower reservoir layers, the DO may equal zero. Flow augmentation benefits may, nevertheless, be positive up to a certain proportion. Within that proportion, the flow should be augmented by an amount that minimizes costs. 74-13: IV,8

4. Costs of Impounding Reservoirs

To determine the correct design capacity of the reservoir, the experienced average day demand for the most recent year, in mgd, is multiplied by a design capacity ratio that varies with flow and projected system growth per decade.

Design Capacity Ratio for Impounding Reservoirs

Experienced Average Day Demand (mgd)	Rate of System Growth per Decade			
	Up to 10%	10-20%	20-30%	30-50%
0.1	2.0	2.8	3.6	5.0
0.2	1.9	2.7	3.4	4.7
0.5	1.8	2.5	3.1	4.3
1.0	1.7	2.3	2.9	4.0
2	1.7	2.2	2.7	3.7
5	1.6	2.0	2.5	3.3
10	1.5	1.9	2.3	3.0
20	1.4	1.8	2.1	2.7
30	1.4	1.7	2.0	2.5
40	1.3	1.6	1.9	2.4
50	1.3	1.6	1.8	2.3
60	1.3	1.6	1.8	2.2
70	1.3	1.5	1.8	2.1
80	1.3	1.5	1.7	2.1
90	1.3	1.5	1.7	2.0
100	1.2	1.5	1.7	2.0

This is the desired yield of the reservoir. The storage-yield relationship is given in the following table. The dependable annual yield required (R) in mgy is divided by the average annual stream flow (Q) in mgy, and for this ratio the table gives the ratio of the reservoir storage design capacity (C) in mg divided by the average annual stream flow (Q) in mgy. The reservoir's design capacity is obtained by multiplying the second ratio in the table by the average annual stream flow (Q).

Impounding Reservoir Storage-Yield Relationship

R/Q	C/Q	R/Q	C/Q
.04	.015	.35	.93
.06	.030	.40	1.30
.08	.05	.45	1.8
.10	.07	.50	2.5
.15	.15	.55	3.5
.20	.27	.60	4.8
.25	.43	.65	6.7
.30	.64	.70	9.7

R = Dependable annual yield required

Q = Average annual stream flow

C = Design reservoir capacity

Construction costs for impounding reservoirs, including the necessary intake, are tabulated below:

Impounding Reservoir Construction Cost

Storage Capacity (bg)	Cost ($1,000)	Storage Capacity (bg)	Cost ($1,000)
0.1	200	30	7,700
0.2	300	40	9,150
0.5	600	50	10,450
1.0	950	60	11,600
2	1,500	70	12,750
5	2,600	80	13,800
10	3,950	90	14,800
20	6,000	100	15,800

Construction costs are trended by multiplying them by the current Engineering News Record Building Cost Index and dividing by 584, the index for the January 1963 base period.

Land to be acquired for the reservoir may be estimated at 1.5 times the reservoir surface area at spillway level. The surface area of a reservoir may be derived from its storage capacity, as follows:

Impounding Reservoir Surface Area

Storage Capacity (bg)	Area (acres)	Storage Capacity (bg)	Area (acres)
0.1	35	30	4,100
0.2	75	40	5,100
0.5	180	50	6,000
1.0	320	60	6,800
2	560	70	7,600
5	1,070	80	8,400
10	1,800	90	9,200
20	3,000	100	10,000

Capital investments are subject to a 10% supplement for engineering, administrative, and financing costs. Interest during construction is added at the rate of one-half the interest rate of the project loan for one year.

Reservoir operation and maintenance costs may be estimated at $0.007 per 1,000 gallons produced. O & M costs are trended by multiplying them by the average hourly earnings of production workers for "Water, Steam, and Sanitary Systems," in Table C-1 (Transportation and Public Utilities) of the "Monthly Labor Review" published by the Bureau of Labor Statistics, U.S. Department of Labor, and dividing by $2.37, the average hourly earnings for January 1963. 74-13: IV, 9-12

5. Pumped Quality Storage Reservoirs

Contrary to gravity storage, pumped storage has a substantial operating cost. All the more reason for pumped quality storage to remain ancillary to other pumped storage purposes. However, in the case of pumped storage, one additional function may contribute to making quality storage economically viable: power storage. Water is pumped up daily during hours of low power demand, and released in peak power demand periods, much of the power being recuperated. Water storage has a seasonal rhythm, while the demand for non-storable electric power has a diurnal rhythm. Quality storage water can be pumped up during successive nights, and released every day during peak power demand hours. Because of hourly power rate differentials, most, all, or more than all of the power costs are recovered.

Pumped storage has these advantages over gravity storage: More sites are available for pumped than for gravity storage reservoirs, because pumped storage reservoirs do not depend for replenishment on tributary drainage areas. Instead of having to be located in the upper reaches of a watershed, hours or days of water travel time removed from populated demand areas, pumped storage can be sited downstream, near areas in need of water supply, drought control, and year-round water quality.

Pumped storage reservoirs assure higher quality water than gravity storage reservoirs. The daily cycling of large volumes of water displaced by power generation and back-pumping virtually eliminates stratification, ensuring cool water with consistently high DO content. In gravity storage

reservoirs, water released from upper layers is too warm; the DO in water drained from lower layers approaches exhaustion. This affects the river's self-purifying capacity. Because of this, the design capacity of a gravity type reservoir may have to be considerably larger than that of a pumped storage reservoir to maintain a prescribed water quality in the stream.

Conjunctive operation of gravity and pumped storage reservoirs expands the flexibility needed to ensure stable gravity reservoir levels for water-based recreation throughout the summer season. It may also be cheaper than gravity storage alone, as shown in the following example:

Savings from Joint Operation of Gravity and Pumped Storage

Item	Unit	Storage System		Savings
		Gravity	Joint	
Water Required	a/f	262,000	188,000	74,000
from pumped storage		-	80,000	
from gravity storage		262,000	108,000	
Power Required	KW	721,000	700,000	21,000
from hydropower		-	325,000	
from local steam power		-	375,000	
from distant steam power		721,000	-	
Capital Cost -- Water	$1,000	34,562	38,000	-3,438
pumped storage		-	25,000	
gravity storage		34,562	13,000	
Capital Cost -- Power	$1,000	80,500	64,000	16,500
hydro-power		-	26,500	
local steam power		-	37,500	
distant steam power		80,500	-	
Total Capital Cost	$1,000	115,062	102,000	13,062

Item	Unit	Storage System Gravity	Storage System Joint	Savings
Annual Cost -- Water	$1,000	2,246	2,470	-224
pumped storage		-	1,625	
gravity storage		2,246	845	
Annual Cost -- Power	$1,000	19,595	16,910	2,685
hydro-power		-	3,740	
local steam power		-	6,000	
distant steam power		13,320	-	
associated power costs		6,275	7,170	
Total Annual Cost	$1,000	21,841	19,380	2,461

74-13: IV, 13-15

6. Well Supply Costs

To determine the correct design capacity of well supplies, the maximum day demand experienced during the past 10 years, in mgd, is multiplied by a design capacity ratio varying with flow and expected system growth per decade, as shown in the following table:

Design Capacity Ratio for Well Supplies

Experienced Maximum Day Demand (mgd)	Rate of System Growth per Decade: Up to 10%	10-20%	20-30%	30-50%
0.1	2.2	2.4	2.6	2.8
0.2	2.0	2.2	2.4	2.6
0.5	1.8	1.9	2.2	2.4
1.0	1.7	1.8	2.0	2.2
2	1.6	1.7	1.9	2.1
5	1.5	1.6	1.7	2.0
10	1.4	1.5	1.6	1.9
20	1.4	1.4	1.6	1.8

Design Capacity Ratio for Well Supplies (continued)

Experienced Maximum Day Demand (mgd)	Rate of System Growth per Decade			
	Up to 10%	10-20%	20-30%	30-50%
30	1.3	1.4	1.6	1.8
40	1.3	1.4	1.5	1.8
50	1.3	1.4	1.5	1.7
60	1.3	1.4	1.5	1.7
70	1.3	1.4	1.5	1.7
80	1.3	1.4	1.5	1.7
90	1.3	1.4	1.5	1.7
100	1.3	1.4	1.5	1.7

This is the desired yield. Costs follow:

Well Construction Cost

Well Field Capacity (mgd)	Cost ($1,000)	Well Field Capacity (mgd)	Cost ($1,000)
0.1	20	30	750
0.2	21	40	1,000
0.5	26	50	1,250
1.0	34	60	1,500
2	50	70	1,750
5	125	80	2,000
10	250	90	2,250
20	500	100	2,500

Construction costs may be trended in accordance with directions given for reservoir costs. Trended costs are subject to a 10% supplement for engineering, administrative, and financing services. The land needed for the well field is estimated to cost 2% of the trended construction cost. Interest during construction is estimated at the rate of one-half the interest rate of the project for one year.

Well supply operating and maintenance costs may be estimated at $0.007 per 1000 gallons of water produced. To this must be added pumping O + M costs, as follows:

Pumping O + M Costs
(exclusive of power)

Average Quantity of Water Produced (mgd)	Cost ($/Kgal)	Average Quantity of Water Produced (mgd)	Cost ($/Kgal)
0.1	.050	30	.008
0.2	.039	40	.008
0.5	.027	50	.007
1.0	.020	60	.007
2	.015	70	.006
5	.012	80	.006
10	.011	90	.006
20	.009	100	.005

Operation and maintenance costs may be trended in accordance with directions given for reservoir O + M costs.

Power costs must be added at the rate of $0.004 per 1000 gallons produced for each 100 feet of static and friction head to be overcome. Pumping lift is estimated at 300 feet below ground, plus 100 feet for water pressure.

7. Artificial Ground Water Recharge

An examination of the economics of advanced waste treatment (AWT) systems and recycling schemes has led to the conclusion that artificial ground water recharge using treated municipal wastes presents the most feasible solution to effluent reclamation. Artificial recharge provides the quantitative flexibility needed in using aquifers, prevents mining or a lowering of the water table, as well as land subsidence. It may even avoid sea water intrusion in coastal areas.

74-13: IV, 24

8. Intake and Pumping Stations

Besides impounding reservoirs and wellfields, a water supply may use for its source a natural lake or river. Direct withdrawal requires an intake and pumping station. To determine the correct design capacity for intake and pumping station, the maximum-day demand experienced during the last 10 years is multiplied by a design capacity ratio which varies with flow and projected system growth per decade. The correct ratio may be obtained from the following table:

Design Capacity Ratio for Intake and Pumping Station, Transmission Pipeline Pumping Station, and Treated Water Pumping Station

Experienced Maximum Day Demand (mgd)	Rate of System Growth per Decade			
	Up to 10%	10-20%	20-30%	30-50%
0.1	2.3	3.0	4.0	5.5
0.2	2.2	2.9	3.8	5.3
0.5	2.1	2.8	3.6	5.0
1.0	2.0	2.6	3.5	4.8

Experienced Maximum Day Demand (mgd)	Rate of System Growth per Decade			
	Up to 10%	10-20%	20-30%	30-50%
2	1.9	2.5	3.3	4.6
5	1.8	2.4	3.1	4.3
10	1.8	2.3	3.0	4.1
20	1.7	2.2	2.8	3.9
30	1.6	2.2	2.7	3.8
40	1.6	2.1	2.7	3.7
50	1.6	2.1	2.6	3.6
60	1.6	2.1	2.6	3.6
70	1.6	2.0	2.6	3.6
80	1.5	2.0	2.6	3.5
90	1.5	2.0	2.5	3.5
100	1.5	2.0	2.5	3.5

This is the design capacity for the intake and pumping station. The following table presents construction costs:

Intake and Pumping Station Construction Cost

Design Capacity (mgd)	Cost ($1,000)	Design Capacity (mgd)	Cost ($1,000)
0.1	40	30	630
0.2	40	40	800
0.5	40	50	980
1.0	40	60	1,150
2	55	70	1,300
5	130	80	1,480
10	240	90	1,660
20	465	100	1,820

Intake and pumping station construction costs may be trended in accordance with directions given for reservoir construction costs. Trended costs are subject to a 10% supplement for engineering, administrative, and financing services. Interest during construction is estimated at the rate of one-half the interest rate of the project for one year.

As with reservoirs, O + M costs for intake and pumping station may be estimated at $0.007 per 1000 gallons of water produced. To this must be added pumping O + M costs, as tabulated under "Well Supply Costs." Intake and pumping O + M costs may be trended in accordance with directions given for reservoir O + M costs.

Power costs must be added, at the rate of $0.004 per 1000 gallons produced, for each 100 feet of static and friction head to be overcome. Pumping lift should include an extra 100 feet for water pressure. Power costs are likewise subject to trending as indicated above for O + M costs.

74-13: IV

9. Blending Proportions for Water from Several Sources

When water from two or more sources with several dissimilar critical quality parameters is to be mixed, three cases may be distinguished.

Case 1. One or the other water source meets all the standards. No blending is necessary.

Case 2. For each critical water quality parameter, one or the other source meets the corresponding quality standard; in addition, for each parameter, a + b is smaller than 2s, where:

a = Parameter concentration in source with higher content;

b = Parameter concentration in source with lower content;

s = Standard permissible concentration of parameter.

In that case, an equal amount of water from both sources (and a limited number of other ratios) meet all standards.

Case 3. For each parameter, one or the other source meets the corresponding standard; for one or more parameters, a + b is greater than 2s. A solution may be possible. Establish, for each parameter, the range of mixes that will produce water of acceptable quality. If all ranges overlap, a solution (or a limited number of solutions) is possible. If two or more ranges are incompatible, no solution can be found.

Ranges are computed as follows for each parameter. The range for the source with the higher parameter concentration is:

$$0\% \text{ to } \frac{100(s-b)}{a-b}\%$$

For the other source, the range is complementary:

$$\frac{100(a-s)}{a-b}\% \text{ to } 100\%$$

Here is an example of compatible and incompatible parameter concentrations:

Ranges of Blending Proportions for Two Sources and Several Quality Parameters

Parameter	Concentration			Blending Range	
	Source 1	Standard	Source 2	Source 1	Source 2
A	4	8	10	33-100%	0- 67%
B	60	50	24	0- 72%	28-100%
C	6	8	12	67-100%	0- 33%
Combined		Compatible		67- 72%	28- 33%
		Addition of Parameter D			
D	0.18	0.1	0.05	0- 61%	39-100%
Combined		Incompatible		No solution	

Where no solution can be found through any blending proportions, it may be desirable to compute the damages associated with the use of water of substandard quality with a view to minimizing it, or to find a combination of blending and treatment that will minimize damages and costs.

Where three or more sources are available for blending, linear programming may be applicable. 74-13: IV, 22-23

10. Transmission Pipelines

It is difficult to suggest water importation costs in the abstract. Water conveyance may involve rock excavation, tunneling, bridging, channeling, piping, and pumping. Costs vary with terrain and climate. Nevertheless, average construction and O + M costs of surface transmission pipelines and pumping facilities can be estimated as indicated below.

To determine the correct design capacity for transmission pipelines, the maximum day demand experienced during the last 10 years is multiplied by a design capacity ratio which varies with flow and projected system growth per decade. Applicable ratios may be read from the following table:

Design Capacity Ratio for Transmission Pipelines

Experienced Maximum Day Demand (mgd)	Rate of System Growth per Decade			
	Up to 10%	10-20%	20-30%	30-50%
0.1	2.5	3.2	4.0	5.0
0.2	2.4	3.1	3.8	4.7
0.5	2.3	2.9	3.5	4.4
1.0	2.2	2.7	3.3	4.1
2	2.1	2.6	3.1	3.9
5	1.9	2.4	2.8	3.6
10	1.8	2.2	2.7	3.3
20	1.7	2.1	2.5	3.1
30	1.7	2.0	2.3	2.9
40	1.6	1.9	2.3	2.8
50	1.6	1.9	2.2	2.7
60	1.6	1.8	2.1	2.7
70	1.5	1.8	2.1	2.6
80	1.5	1.8	2.1	2.6
90	1.5	1.8	2.0	2.5
100	1.5	1.7	2.0	2.5

This is the design capacity for transmission pipelines. Construction costs are tabulated below:

Construction Cost of Transmission Pipeline

Flow Required (mgd)	Pipe Diameter (inches)	Cost per Mile ($1,000)
0.1	4	24
0.2	6	29
0.5	8	35
1.0	10	41
2	12	48
3	16	65
5	20	83
7.5	24	103
10	24	103
12.5	26	114
15	28	125
17.5	30	136
20	32	146
25	34	156
30	36	167
35	38	178
40	40	190
45	42	201
50	44	212
55	46	225
60	48	237
70	50	250
80	52	261
90	56	288.
100	58	301

If the source of raw water supply is not at an elevation adequate for gravity flow to the point of discharge, it is necessary to pump the water to overcome static lift as well as friction losses in pipelines. To determine the number of pumping stations required, three inputs are needed:

Es = Ground elevation at source;
Ed = Elevation at point of discharge;
SD = Length from source to discharge in 1000 feet.

Three cases are possible:

1. $\frac{Es-Ed}{SD}$ = 4 or over: no pumping is needed;

2. $\frac{Es-Ed}{SD}$ = 0 to 4: pumping is required. Number of pumping stations = $\frac{(4xSD) - (Es-Ed)}{400}$

3. $\frac{Ed-Es}{SD}$ = positive: pumping is required. Number of pumping stations = $\frac{(4xSD) + (Ed-Es)}{400}$

Once the number of pumping stations is known, the design capacity of pipeline pumping stations may be determined by referring to the table of ratios presented in Section IV, B, 8, "Intake and Pumping Stations." Construction costs for pipeline pumping stations are shown in the following table:

Construction Cost of Pipeline Pumping Stations

Design Capacity (mgd)	Construction Cost ($1000)	Design Capacity (mgd)	Construction Cost ($1000)
0.1	37	30	605
0.2	40	40	770
0.5	50	50	920
1.0	67	60	1,080
2	94	70	1,235
5	160	80	1,390
10	256	90	1,550
20	438	100	1,700

Construction costs of pipeline and pumping stations may be trended in accordance with directions given for reservoir costs. Add to this a 10% supplement to cover engineering, administrative, and financing services. The prevailing regional right-of-way cost per mile should be included; if not available, add $2,500 per mile. Interest during construction may be computed at one-half the interest rate for the project for one year.

Annual operation and maintenance costs for transmission pipelines are 0.25% of the trended construction cost. Those for pumping combine two items: an O + M cost per 1000 gallons for each pumping station, plus a pumping power cost. The O + M cost for pumping is tabulated below:

Pumping O + M Cost Exclusive of Power

Average Quantity of Water Produced (mgd)	O + M Cost (¢/Kgal)	Average Quantity of Water Produced (mgd)	O + M Cost (¢/Kgal)
0.1	5.0	30	0.8
0.2	3.9	40	0.8
0.5	2.7	50	0.7
1.0	2.0	60	0.7
2	1.5	70	0.6
5	1.2	80	0.6
10	1.1	90	0.6
20	0.9	100	0.5

The pumping power cost is estimated at $0.004 per 1000 gallons for each 100 feet of static and friction head to be overcome. Total pumping station and power O + M costs must be trended as explained under "Reservoirs."

74-13: IV, 24-29

11. Raw Water Storage

Where a single transmission pipeline conveys water from the source to the treatment plant, there is a risk of interruption of supply. To assure a reliable water supply, raw water storage may be provided at the discharge end of the pipeline. For maximum reliability, raw water storage in the amount of 10 days of average day treatment plant design capability should be provided for all pipelines over 5 miles in length. For pipelines less than 5 miles long, the cost of raw water storage, as tabulated below, may be reduced 20% for each mile less than 5 in length.

Raw Water Storage Construction Costs

Storage Capacity (mg)	Cost ($1,000)	Storage Capacity (mg)	Cost ($1,000)
1	20	50	290
2	30	100	510
5	50	200	960
10	80	500	2,240
20	137	1,000	4,400

These costs cover a lined, fenced, open raw water basin, with emergency pumping facilities. Costs may be trended by reference to directions given for trending reservoir costs. Add a 10% supplement for engineering, administrative, and financing services. Interest during construction is estimated at one-half the project interest rate for one year.

O + M costs relating to raw water storage facilities may be estimated at an annual cost of 0.5 percent of the trended construction cost of the storage. No further trending is needed. 74-13: IV

12. Raw Water Treatment Plant and Storage

It is assumed that all supplies must be treated or at least disinfected. Treatment includes softening where the supply has over 200 ppm hardness, and disinfection. The design capacity of the treatment plant and storage is a function of flow and rate of system growth per decade. It may be determined by multiplying the maximum day demand experienced during the last 10 years by the appropriate ratio in the following table:

Design Capacity Ratio for Treatment Plant

Experienced Maximum Day Demand (mgd)	Rate of System Growth per Decade			
	Up to 10%	10-20%	20-30%	30-50%
0.1	2.2	2.6	3.0	4.0
0.2	2.0	2.4	2.7	3.6
0.5	1.8	2.1	2.5	3.2
1.0	1.6	2.0	2.3	3.0
2	1.6	1.9	2.2	2.8
5	1.5	1.7	2.1	2.6
10	1.4	1.6	2.0	2.5
20	1.3	1.6	1.9	2.4
30	1.3	1.6	1.9	2.3
40	1.3	1.5	1.8	2.3
50	1.3	1.5	1.8	2.2
60	1.3	1.5	1.8	2.2
70	1.3	1.5	1.8	2.2
80	1.3	1.5	1.8	2.2
90	1.3	1.5	1.8	2.2
100	1.2	1.5	1.8	2.1

The following costs cover the treatment plant, treated water storage provided to the extent of 25% of the plant design maximum day capacity) and, for treatment plants of less than 1 mgd capacity, treated water pumping facilities:

Treatment Plant and Storage Construction Cost

Design Capacity (mgd)	Cost ($1,000)	Design Capacity (mgd)	Cost ($1,000)
0.1	60	30	2,700
0.2	90	40	3,400
0.5	140	50	4,000
1.0	220	60	4,600
2	380	70	5,100
5	700	80	5,600
10	1,150	90	6,100
20	2,000	100	6,550

These costs may be trended as shown for reservoir capital costs. Add a 10% supplement for engineering, administrative, and financing services, plus a 2% supplement for land acquisition. Interest during construction is estimated at the rate of one-half the interest rate for the project for one year.

Operation and maintenance costs for treatment plants and treated water storage are tabulated below, in cents per 1000 gallons, exclusive of chemicals and power:

Water Treatment and Storage O + M Costs
Exclusive of Chemicals and Power

Average Quantity Treated (mgd)	O + M Cost (¢/Kgal)	Average Quantity Treated (mgd)	O + M Cost (¢/Kgal)
0.1	12.0	30	2.2
0.2	10.2	40	2.1
0.5	7.8	50	2.0
1.0	6.2	60	1.9
2	4.8	70	1.8
5	3.4	80	1.8
10	2.8	90	1.7
20	2.4	100	1.7

Water treatment O + M costs should be trended in accordance with directions given under "Reservoirs."

The cost of chemicals must be added to treatment O + M costs as follows:

Treatment Process	Cost of Chemicals (¢/Kgal)
With softening	3.0
Without softening	
Supply from flowing river or stream	1.8
Supply from other source	0.9

Where no treatment other than disinfection is required, the treatment plant and treated water storage facility can be dispensed with. The cost of disinfection is estimated at 0.4 cents per 1000 gallons. This is an all-inclusive cost, but remains subject to trending in accordance with directions given for trending reservoir O + M costs. 74-13: IV, 30-33

13. Treated Water Pumping Stations

Treated water pumping sufficient to overcome a 250-foot head is required for distribution pressure. To determine the correct design capacity for treated water pumping stations, the maximum-day demand experienced during the last 10 years is multiplied by a design capacity ratio which varies with flow and projected system growth per decade. A table of such ratios is provided under "Intake and Pumping Stations." Once the design capacity has been established, the following construction costs may be applied:

Treated Water Pumping Station Construction Cost

Design Capacity (mgd)	Pumping Station Integral with Treatment Plant ($1000)	Pumping Station Separate from Treatment Plant ($1000)
0.1	*	30
0.2	*	32
0.5	*	40
1.0	40	52
2	60	75
5	110	135
10	185	220
20	325	375
30	460	530
40	600	675
50	740	810
60	870	950
70	1,000	1.090
80	1,140	1,230
90	1,270	1,360
100	1,400	1,500

* The cost of treated water pumping stations of less than 1 mgd capacity is included in the cost of the treatment plant.

Construction costs for treated water pumping stations are subject to trending as directed under "Reservoirs." Add 10% for engineering, administrative, and financing services. Interest during construction is estimated at the rate of one-half the interest rate of the project for one year.

Treated water pumping station O + M costs, exclusive of power, are as shown in the table under "Well Supply" pumping costs. O + M costs should be trended in accordance with directions given under "Reservoir" O + M costs.

Power costs may be estimated at $0.004 per 1000 gallons produced for each 100 feet of static and friction head to be overcome. For treated water ready for distribution, the head is estimated at 250 feet for pressure purposes. 74-13: IV, 33

14. Water Allocation to Meet Quantity and Quality Needs

A water utility operator may be faced with a water allocation problem involving several sources. His objective is to meet quantitative and qualitative water supply requirements. If he is concerned only with the over-all concentration of total dissolved solids, a simple method for solving the problem is provided by the type of schedules shown below. In a fictitious example, there are, in addition to the existing water supply, one or more fresh water sources, renovated wastewater, and an unlimited amount of saline water such as is available on the sea coast. Through the application of distillation to the saline water resource, any desired quantity and quality of water can always be achieved regardless of the quality of any source.

Definitions and Symbols

Description	Quantity	Quality
Total water requirements	R	r
Present water supply	S	s
Additional fresh water source I	F1	f1
Additional fresh water source II	F2	f2
Renovated wastewater	W	w
Distilled saline water	D	d

Fictitious Problem

Quantity (mgd)		Quality (ppm)	
R =	60; 70; 80	r =	800; 500; 200
S =	40	s =	500
F1 =	24	f1 =	1,200
F2 =	12	f2 =	1,000
W =	30	w =	850
D =	infinity	d =	50

Rules are that water is to be used in the order of increasing mineral content; however, distilled water remains last because of cost, and wastewater next to last because of resistance to its use. At this point, costs are otherwise not considered. Nine solutions are grouped below under three schedules:

Schedule I: 60 mgd

	800 ppm		500 ppm		200 ppm	
	mgd ppm	gpd-TDS	mgd ppm	gpd-TDS	mgd ppm	gpd-TDS
R	60x 800	=48,000	60x 500	=30,000	60x 200	=12,000
S	40x 500	=20,000	40x 500	=20,000	20x 500	=10,000
Bal	20	28,000	20	10,000	40	2,000
F2	12x1,000	=12,000	9x1,000	= 9,000	0	0
Bal	8	16,000	11	1,000	40	2,000
F1	8x1,200	= 9,600	0	0	0	0
Bal	0	6,400	11	1,000	40	2,000
W	0	0	0	0	0	0
Bal	0	6,400	11	1,000	40	2,000
D	0	0	11x 50	= 550	40x 50	= 2,000
Bal	0	6,400	0	450	0	0
R	60x 693	=41,600	60x 492	=29,550	60x 200	=12,000

Schedule II: 70 mgd

	800 ppm		500 ppm		200 ppm	
	mgd ppm	gpd-TDS	mgd ppm	gpd-TDS	mgd ppm	gpd-TDS
R	70x 800	=56,000	70x 500	=35,000	70x 200	=14,000
S	40x 500	=20,000	40x 500	=20,000	23x 500	=11,500
Bal	30	36,000	30	15,000	47	2,500
F2	12x1,000	=12,000	12x1,000	=12,000	0	0
Bal	18	24,000	18	3,000	47	2,500
F1	18x1,200	=21,600	0	0	0	0
Bal	0	2,400	18	3,000	47	2,500
W	0	0	0	0	0	0
Bal	0	2,400	18	3,000	47	2,500
D	0	0	18x 50	= 900	47x 50	= 2,350
Bal	0	2,400	0	2,100	0	150
R	70x 765	=53,600	70x 470	=32,900	70x 198	=13,850

Schedule III: 80 mgd

	800 ppm			500 ppm			200 ppm		
	mgd	ppm	gpd-TDS	mgd	ppm	gpd-TDS	mgd	ppm	gpd-TDS
R	80x	800=	64,000	80x	500=	40,000	80x	200=	16,000
S	40x	500=	20,000	40x	500=	20,000	26x	500=	13,000
Bal	40		44,000	40		20,000	54		3,000
F2	12x	1,000=	12,000	12x	1,000=	12,000	0		0
Bal	28		32,000	28		8,000	54		3,000
F1	22x	1,200=	26,400	0		0	0		0
Bal	6		5,600	28		8,000	54		3,000
W	6x	850=	5,100	8x	850=	6,800	0		0
Bal	0		500	20		1,200	54		3,000
D	0		0	20x	50=	1,000	54x	50=	2,700
Bal	0		500	0		200	0		300
R	80x	794=	63,500	80x	497=	39,800	80x	196=	15,700

Note: For the sake of simplicity, whole numbers of millions of gallons per day were used in all three schedules, leaving residues whereby the quality of the blended water is slightly upgraded. In a real situation, this would not be necessary.

The nine alternative requirements have been met, as shown in the following summary:

The Nine Solutions

	800 ppm			500 ppm			200 ppm		
S	40	40	40	40	40	40	20	23	26
F1	8	18	22	--	--	--	--	--	--
F2	12	12	12	9	12	12	--	--	--
W	--	--	6	--	--	8	--	--	--
D	--	--	--	11	18	20	40	47	54
R	60	70	80	60	70	80	60	70	80

The above procedure can be used also if brackish water is available and a single-phase process such as electrodialysis or reverse osmosis is selected. However, there will be instances when no solution is feasible,

because of the limitations of these processes. In those cases, distillation will be necessary. -- In addition to the multiple-source problem presented here, there is always the possibility of applying some form of desalination to any of the sources directly. 74-13: IV, 36-40

15. Selection of a Suitable Desalting Process

Selection of the most suitable desalting process for a given purpose depends on many circumstances. Raw water salinity, hardness, temperature are but a few. If it is hoped to blend desalted with more raw water for cost reduction, then a process based on a change-of-phase is needed. If desalination is intended for intermittent use, an electric process such as electrodialysis or reverse osmosis permits more flexible operation. If a combined power and desalting plant can be justified, some form of distillation is the answer. If a relatively small plant with low operating costs is desired to reduce the salt content in brackish water, a membrane process is indicated. A very large plant would probably be designed as a combination of two highly economical distillation processes: multi-stage flash and vertical tube evaporation. The freezing process has applications of its own, and so has ion exchange. Several other processes are theoretically feasible and may some day become practical. 74-13: IV, 40-41

16. Desalination Costs

The following cost table is based on actual experience:

Realized Desalination Costs

Plant Location	Year	Size (mgd)	Process	Fixed Charge	Water Cost (¢/Kgal)
Buckeye, Arizona	1962	0.65	ED	6.7%	69
Key West, Florida	1966	2.6	MSF	6%	94
St. Thomas, Virgin Isl.	1967	2.5	MSF-dual	6%	90
Rosarito Beach, Mexico	1969	7.5	MSF-dual	6%	85

Estimated capital, annual, and water costs for six desalting processes are tabulated below:

Estimated Desalination Costs

Item	Unit	Desalting Process*					
		MSF	VTE-MSF	ED	RO	VF-VC	IX
Assumptions							
Days/year	%	90	75	85	90	90	90
Design Capacity	mgd	24.4	58.4	3.2	9.1	1.5	1.5
Interest Rate	%	7	7	7	7	7	7
Plant Life	yrs	30	30	30	30	30	30
Annual Charge**	%	8.56	9.06	8.06	8.06	8.46	8.06
Capital Cost	$MM	39.7	54.8	3.34	15.7	3.63	5.7
Annual Costs							
Capital	$MM	3.7	5.0	0.27	1.25	0.31	0.46
O + M	$MM	4.0	5.4	0.36	1.18	0.27	0.25
Total	$MM	7.7	10.4	0.63	2.43	0.58	0.71
Water Cost	¢/Kgal	97	65	63	81	116	141

* Processes: Multi-stage flash (MSF); Vertical tube evaporation/multi-stage flash (VTE-MSF); Electrodialysis (ED); Reverse osmosis (RO); Vacuum-freeze vapor compression (VF-VC); Ion exchange (IX).

** Annual Charge: This is the percent rate of the capital sufficient to cover the annual interest plus the average annual amortization of the principal. In the case of some processes, the annual charge also includes taxes and insurance: MSF (0.5%), VTE-MSF (1%), and VF-VC (0.4%).

74-13: IV, 40-44

17. Wastewater Outfall Management

The enactment of PL 92-500 should put an end to the discharge of waste water into streams and lakes, and even to the oceans. The remarks that follow are, nevertheless, of at least theoretical interest. Moreover, even treatment of all man-made effluents will not ensure 100% pure water in our waterways. Natural pollution through diffuse discharges is much more difficult to control, and may require artificial instream reaeration or other treatment methods.

Individual waste treatment may be completely or largely avoided if outfalls are captured for conveyance to the ocean or staggered in space or time. Pipelines have been laid on river beds for waste disposal for many years. A stream moves by gravity, and a pipe following its course can inexpensively convey wastes of all descriptions, including sludge, to selected ocean outfalls.

A river's waste-assimilative capacity can be stretched by staggering outfalls in space. Longitudinal spreading of outfalls that are bunched together in industrial areas will avoid releasing waste loads in excess of the stream's self-purification rate. The reoxygenation capacity governs the tons of waste per mile which the river can digest. By capturing outfalls

into by-pass pipelines for release at predetermined distances downstream, it should be possible to protect the quality of the receiving water while reducing the need for additional waste treatment.

In addition, wastewater outfalls can be managed through diurnal staggering. Industrial discharges are concentrated between 8 a.m. and 6 p.m.; sewage is produced in two daily peaks. If these loads could be released at staggered time intervals throughout the 24 hours of the day, the river could digest them more readily. The construction of 24-hour retention ponds would permit successive rather than simultaneous releases from a number of polluter stations. An appropriate time schedule could minimize by-pass piping and costs.

Conjunctive space and time staggering of discharges can stretch the waste-assimilative capacity of a stream to its maximum. By adjusting trade-offs between piping costs and storage costs, the most economical waste disposal system can be designed. Calculations can take into account BOD-type pollution, but also other water quality parameters such as suspended solids, turbidity, phosphorus, nitrogen, pH, and salinity. 74-13: V, 1-3

18. Sewage Treatment for Disposal

Treatment processes are selected to purify to predetermined levels the sewage resulting from injection of a large variety of domestic and other wastes into a potable public water supply of highly variable quality. Such processes are linked together in groups to perform in succession the mechanical (primary treatment), biological (secondary treatment), and chemical (tertiary treatment) removal of specific types of contaminants. A table of processes, not all of which are necessarily included in every sewage treatment operation, together with their efficiencies in removing pollutants, is presented below:

Wastewater Treatment Processes and Their Capabilities

Processes	Pollutant Removal in Percent					Residue
	SS	BOD	COD	TKN	P	TDS (ppm)
Preliminary Treatment						
Screening						
Grinding						
Grit, Grease, and Scum Removal						
Primary Treatment (Mechanical Processes)	63	32.5	35.7	16.7	15.4	--
Sedimentation in Settling Basins						
Mechanical Aeration						
Final Sedimentation						
Chlorination or Other Disinfection						
Secondary Treatment (Biological Processes)						
a. Trickling Filter Process		85				
b. Activated Sludge Process	91.3	90	85.7	33.3	23.1	--
Aeration by Air Diffusers						
Sludge Thickening						
Sludge Elutriation						
Vacuum Filtration						
c. Digestion Process		92				
Aerobic Digestion in Stabilization Pond						
Completely Mixed Anaerobic Digestion						
Tertiary Treatment (Chemical Processes)						
a. Microscreening	97.3	96	90	43.3	30.8	--
Rapid Sand Filtration						
b. Lime Clarification Sequence						
Coagulation						
Flocculation (alum or lime)						
Sedimentation	93.5	98	93.4	50	46.2	--
Ammonia Air Stripping	93.5	98.5	93.4	86.7	46.2	--
Multi-Media Filtration	98.7	98.5	93.7	86.7	84.6	--
Granular Carbon Adsorption	99.1	99.5	98.6	90	84.6	--
c. Nitrification and Denitrification	97.8	98	94.9	93.3	96.2	--
Multi-Media Filtration	99.6	98	95.1	95	98.5	--
Desalination						
Distillation or Evaporation						10
Freezing						50
Ion Exchange						500
Electrodialysis						500
Reverse Osmosis						200

Notes:

1. Pollutants: SS = Suspended Solids
   BOD = 5-day biochemical oxygen demand
   COD = Chemical oxygen demand
   TKN = Total Kjeldahl nitrogen, including ammonia and organic nitrogen
   P = Phosphorus
   TDS = Total dissolved solids

2. Treatment Sequences: The three secondary treatment processes are alternate -- not cumulative technologies. The three tertiary sequences again are separate options that are rarely cumulated.

3. Desalting: The effect of desalination is indicated as residue in ppm of TDS.

Costs and effectiveness of various sewage treatment processes are presented in the following tables:

Wastewater Disposal Treatment Costs

Treatment Process	Treatment Cost (¢/Kgal)		
	1 mgd	10 mgd	100 mgd
1. Primary Sedimentation + Sludge Disposal	13.7	7.7	4.4
2. Primary, Activated Sludge, + Sludge Disposal	23.3	13.5	8.2
3. Microscreening	1.4	1.1	0.9
4. Single-Stage Lime Clarification	17.1	6.9	3.7
5. Two-Stage Lime Clarification	20.9	8.4	5.0
6. Ammonia Stripping and Recarbonation	7.0	4.0	3.0
7. Multi-Media Filtration	6.8	3.0	1.4
8. Granular Carbon Adsorption (40-minute contact)	32.3	10.8	7.2
9. Chlorination (8 mg/l)	2.1	0.8	0.4

Inasmuch as the above costs contain some duplication (1 & 2; 4 & 5), they are not all additive. Chains of processes which lead to specified degrees of contaminant removal follow:

Treatment Effectiveness and Cost

Estimated Contaminant Removal, in Percent				Processes Included	Treatment Cost (¢/Kgal)		
BOD	COD	Phos	Nitr		1 mgd	10 mgd	100 mgd
35	--	10	0	1	13.7	7.7	4.4
88	--	25	0	2,9	25.4	14.3	8.6
95	--	35	0	2,3,9	26.8	15.4	9.5
97	--	92	0	2,4,9	42.5	21.2	12.3
97	--	92	0	2,5,9	46.3	22.7	13.6
97	--	92	85	2,4,6,9	49.5	25.2	15.3
--	98	95	85	2,4,6,7,8,9	88.6	39.0	23.9
--	98	98	85	2,5,6,7,8,9	92.4	40.5	25.2

Note: Costs in the above two tables are as of January 1970.

74-13: V, 5-11

19. Sewage Treatment for Reuse

If renovated wastewater is to be reused for public water supply, a very complete succession of sewage treatment processes is required. Desalination is advisable, not that the last vestige of mineral content needs removing, but in order to minimize the risk to health from virus, pathogens, toxic, or radioactive substances.

Treated wastewater can be utilized in many ways short of domestic water supply. An ingenious method of reuse whereby much of the treatment is made unnecessary has been named the cascade method of water reuse. Water whose quality no longer meets the requirements of one use can be of value in another use with less stringent quality specifications; this process can be repeated several times before a single treatment for disposal is performed.

Treatment costs are tabulated below for wastewater reuse as agricultural, industrial, recreational, and even potable water:

Wastewater Reuse Treatment Costs

Treatment Process	Treatment Cost (cents/Kgal) 1 mgd	10 mgd	100 mgd
1. Conventional Treatment	36.0	11.5	6.3
2. Separate Nitrification	11.5	4.0	2.0
3. Lime Clarification	11.2	8.4	5.0
4. Filtration	6.8	3.0	1.4
5. Carbon Adsorption	32.3	10.8	7.2
6. Ion Exchange	22.9	16.1	10.9
7. Eletrodialysis	26.7	17.0	11.2
8. Reverse Osmosis	37.8	30.4	27.6
9. Chlorination	2.3	0.9	0.3
10. Brine Disposal (Evaporation Ponds)	7.8	7.0	6.2

The following chains of processes achieve water renovation levels suitable for various types of water reuse:

Reuse Treatment Effectiveness and Costs

Effluent Concentration (mg/l)				Processes Included	Suitable Reuse	Treatment Cost (cents/Kgal)		
COD	Phos	Nitr	TDS			1 mgd	10 mgd	100 mgd
50	10	20	1000	1	---	0	0	0
30	8	19	1000	1,4,9	Agricultural	9.1	3.9	1.7
22	0.2	18	1000	1,3,4,9	Industrial	20.3	12.3	6.7
22	0.2	15	1000	1,2,3,4,9	Recreational	31.8	16.3	8.7
5	0.2	8	500	1,2,3,4,5,6,9	Potable	87.0	43.2	26.8
5	0.5	5	200	1,4,8,9	Potable	46.9	34.3	29.3
10	8	18	1000	1,5,9	Potable	35.4	11.7	7.5

Note: Not included in the table above is the cost of conventional treatment or that of brine disposal. Costs in both tables are as of January 1970.

The most notable recycling plant in the U.S. is at Lake Tahoe, California. In operation 24 hours a day since 1968, with a capacity of 7.5 mgd, it produces water exceeding all drinking water quality standards. Removal efficiency for selected contaminants is: Suspended solids, color, odor, coliform bacteria, and viruses (100%); turbidity (99.9%); BOD (99.4%); phosphorus (99.1%); MBAS (97.9%); and COD (96.4%). Costs are tabulated below:

Lake Tahoe's Wastewater Treatment Costs
(Plant Capacity: 7.5 mgd)

Process	Capital Cost (¢/Kgal)	O + M Cost (¢/Kgal)	Total Cost (¢/Kgal)
Conventional Treatment	6.75	10.45	17.20
Advanced Waste Treatment			
Lime Coagulation	0.97	3.13	4.10
Lime-Mud Dewatering	0.29	0.65	0.94
Lime-Mud Recalcining	1.06	3.21	4.27
Ammonia Stripping (intermittent)	0.80	0.71	1.51
Recarbonation	0.40	0.44	0.84
Filtration	1.78	2.33	4.11
Carbon Adsorption	1.63	1.12	2.75
Carbon Regeneration	0.52	2.17	2.69
Total Advanced Waste Treatment	7.45	13.76	21.21
Miscellaneous	0	1.16	1.16
Grand Total	14.20	25.37	39.57

74-13: V, 11-15

20. Trade-Offs Between Wastewater Disposal and Reuse

Wastewater disposal by land spraying is cheaper than any reuse. Disposal is also cheaper than reuse when renovation requires distillation or deionization from 2500 ppm of TDS.

Wastewater reuse is cheaper than disposal when:

a. Wastewater contains valuable products.
b. Incremental water supply costs over $5/Kgal.
c. Effluents are strictly regulated.
d. Effluent quality must be higher than raw water quality.
e. Disposal involves injection into mined cavities.
f. Renovation requires only standard secondary treatment and disposal is done by injection or 5-mile transport.
g. Renovation requires distillation or deionization from 2500 ppm, and disposal is done by injection or by 50-mile transport of wastes weaker than 1500 ppm.

74-13: V, 16

21. Storm Water Treatment

Storm water may carry higher pollutant loads than sewage. If it enters the sewer system, it may overtax the treatment plant's capacity, necessitating the by-pass of sewage along with storm water. Combined sewers are not believed effective in controlling pollution. A more satisfactory design would consist of separate sewage and storm water collection systems, with the option of routing storm water through the treatment plant when the latter's capacity is adequate, and an automatic by-pass provision for storm water only, whenever that capacity comes close to being exceeded.

Settling ponds may have the advantage of cutting down the excessive turbidity if not the contaminants. If such ponds are located upstream from the sewage treatment plant, they might act as temporary buffers permitting subsequent purification of the storm water in the treatment plant.

Storm water needs attention, whether it is intended for disposal or for reuse. Not only is it sudden, hard to contain, sometimes persistent and even disastrous, but may likely be toxic, containing lead and oil in solution, and may require expensive desalting treatment. One analysis showed as much as 5,500 ppm of lead, 2,150 ppm of chloride, 34 ppm of COD, 27 ppm of hexavalent chromium, 5 ppm of bromine, and a relatively high content of nitrate.

The cost of separate storm water collection and treatment in the U.S. has been estimated at $49 billion -- over ten times the cost of industrial waste treatment. Partial separation of storm water from sewage would cost $30 billion. The use of holding ponds and underground reservoirs, if practicable everywhere, would bring the cost down to $12 billion.

74-13: V, 16-17

22. Industrial Wastewater Treatment

All of American industry must henceforth comply with the stipulations of PL 92-500. It must clean its wastes before discharge, or modify its processing methods to permit internal recycling of water. Recycling involves treatment, and wherever treatment for disposal restores water to a quality fit for reuse, why discharge it? It will become increasingly difficult to draw a line between treatment for disposal and treatment for reuse.

To reduce total wastewater discharge volume, it is necessary either to recycle increasing amounts of spent water within each function, or to reuse spent water from one function as makeup water for another. Three methods of industrial water use are illustrated below in Figure 17. Each method provides 4 units of process water, 10 units of cooling water, and 1 unit of boiler makeup water. In the once-through method, 15 units of water are withdrawn and released; in the multiple-use system, the same functions are performed by 10 units; the reuse-recycle method, incorporating several treatment processes, provides the same services with one unit.

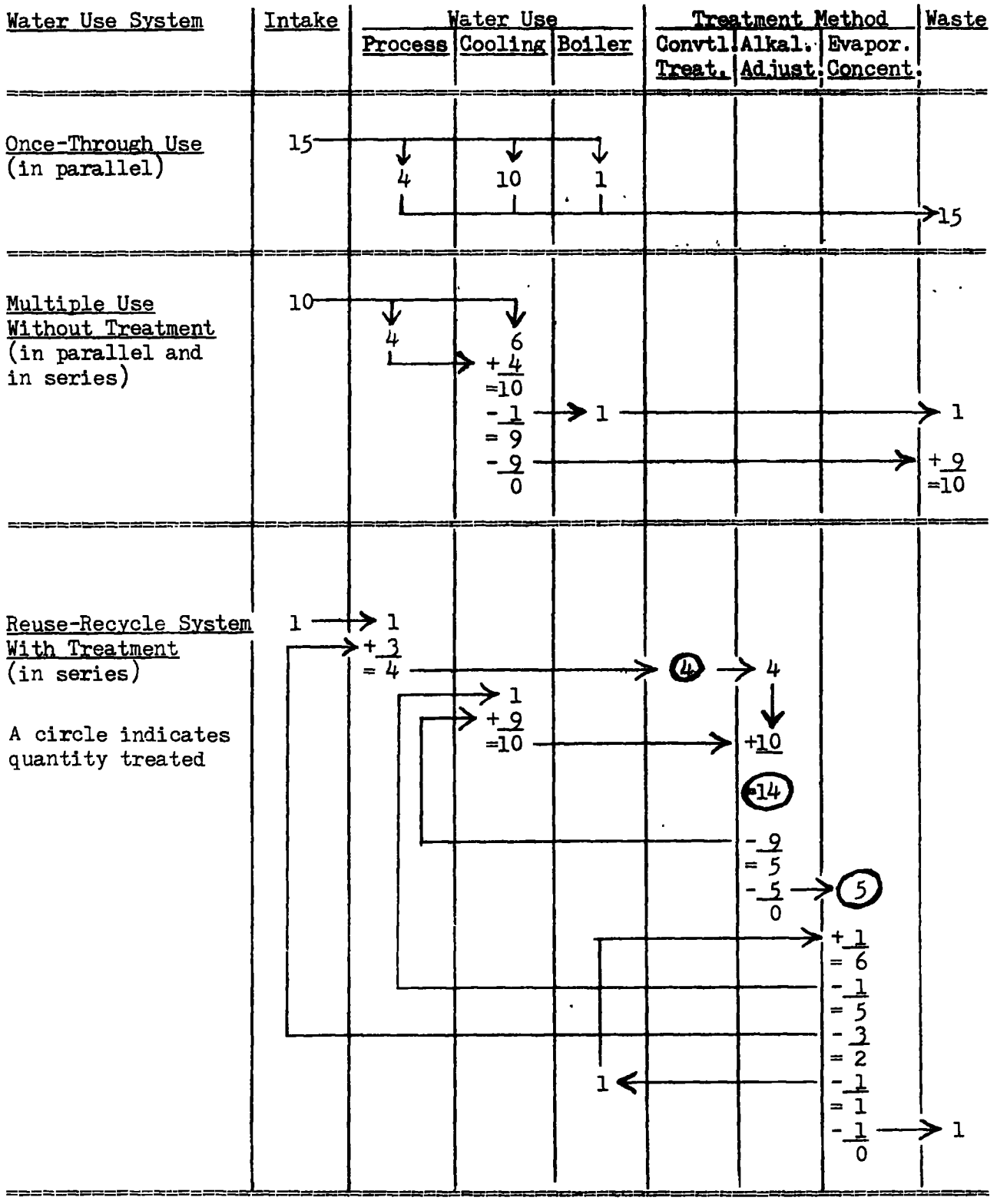


Figure 17. Industrial Water Use Systems
(Numbers represent water units)

Tabulated below are industrial wastewater treatment costs applicable to the removal of main pollutant types:

Industrial Wastewater Treatment Costs

Pollutant	Treatment Process	Removal (ppm)	Cost per Kgal (¢)	Cost per lb Removed (¢) Pollutant	Sludge	Total
Suspended Solids	Primary: Sedimentation	200	2.5	2.0	0.5	2.5
Organic Matter	Secondary: Biological Oxidation	400	5.0	2.0	1.0	3.0
TDS, incl. Hardness	Multi-effect Distillation	3500	100.0	3.5	-	3.5
	+ Evap. Pond (1 mgd)	250	-	-	-	-
Alkalinity (as $CaCO_3$)	Addition of Acid	-	-	2.0	-	2.0
	Total:		107.5	9.5	1.5	11.0

Note: Costs are as of 1971. The cost of acid to adjust alkalinity is estimated at twice the chemical cost of 1¢/lb for sulfuric acid.

74-13: V, 18-21

23. By-Product Recovery from Industrial Effluents

Industrial Wastewater reclamation may pay a bonus: the recovery of formerly discharged materials, chemicals, or other by-products. One study listed six industries which can profitably treat their effluents through desalination, the commercial value of recovered by-products more than paying the costs. By desalting residual whey traditionally discharged to streams, the cheese industry can recover a food product rich in protein and lactose worth $300 million per year, at an annual desalting cost of $120 million.

Through the use of ion exchange or electrodialysis, the plating and metal finishing industry can turn highly toxic wastes into valuable chemicals such as: chromic acid, nickel sulfate, and cyanides of copper, zinc, brass, cadmium, and silver. The pulp and paper, iron and steel, nuclear power, and coal mining industries can likewise benefit from by-product recovery through desalination.

74-13: V,22

24. Collective Wastewater Treatment

Collective treatment involves the interception of multiple effluents before their discharge to receiving waters, and their conveyance by pipeline to a conveniently located central plant site for aggregate treatment and disposal. Municipal wastewater, which may be excessively loaded with phosphorus and nitrogen, and industrial effluents, which may be deficient in such nutrients, may compensate one another in collective treatment facilities.

Cost indicates in most instances what effluents should participate in collective treatment. Conveyance costs must be weighed against economies of scale. Certain pretreatment may be required or desired. Factors which significantly influence the cost of collective wastewater treatment include: the flow rate, BOD concentration, suspended solids, and the biological reaction rate.

Tabulated below are sizing factors and costs of collective waste water treatment facilities. For further details, consult reference (20), in 74-13: V, 44.

Capacity Sizing Factors for Individual Processes in Collective Wastewater Treatment Facilities

Range of Flow Rate (Q) = 1 to 100 mgd

Process	Sizing Factor
Preliminary	1.0
Primary Clarification	2.0 - 0.008Q
Activated Sludge	1.3 - 0.002Q
Aeration	1.8 - 0.004Q
Sludge Return	2.0 - 0.005Q
Final Clarifier	2.0 - 0.007Q
Chlorinator	1.0
Thickener	1.5 - 0.004Q
Aerobic Digester	1.5 - 0.003Q
Anaerobid Digester	2.0 - 0.005Q
Centrifuge	2.0 - 0.005Q
Vacuum Filter	2.0 - 0.005Q
Sludge Drying Beds	1.0

Capital and Operating Costs of Collective Wastewater Treatment Facilities

(Costs as of September 1969)

Process	Sizing	Cost Base	Capital Cost ($M)	Operating Cost ($/mgd)
Pretreatment	Q	Q	$19 \times Q^{0.63}$	$500 + \frac{2,150}{Q^{0.63}}$
Primary Clarification or Sedimentation	Overflow Rate (800 gal/sq ft/day)	SA	$17.3\ SA + 6.7\ (SA)^{0.1}$	$909 + \frac{2,370}{Q^{0.5}}$
Activated Sludge	F/M ratio nitrification rate, or reaction rate	Basin volume (mg)	(226 x volume) + 67	$2,700 + (\frac{2,500}{volume})^{0.67*}$

Process	Sizing	Cost Base	Capital Cost ($M)	Operating Cost ($/mgd)
Oxygen Requirements	BOD Reduction and Respiration	Aerator HP	--	--
Blower House	--	--	$13.6 + \frac{7.6 \text{ cu ft/min}}{1,000}$	--
Sludge Pumps	Q	Q	$4.7 + 1.45Q$	--
Final Clarifier	Overflow Rate (750 gal/sq ft/day)	SA	$16.2\ SA + \frac{6.9}{SA^{0.13}}$	--
Aerobic Digester	15-Day Retention Time	Basin volume (mg)	--	--
Anaerobic Digester	20-Day Sludge Retention	Volume (1000 cu ft)	$134V + \frac{13.8V}{V^{0.87}}$	$\frac{1200V}{Q}\left(0.048 + \frac{0.54}{V^{0.5}}\right)$**
Thickener	Mass Loading (10 lb/sq ft/day)	SA	$SA\left(24.2 + \left(\frac{11.7\ SA}{13.3\ \exp}\right)\right)$	
Centrifuge	Flow Rate (gm/HP)	HP	--	--
Vacuum Filter	4-7 lb Sludge/sq ft	Area (sq ft)	$16.5 + 48\ \frac{\text{Area}}{100}$	$0.18\frac{S}{Q}\left(\frac{700}{F} + 0.38\ (2-0.1Q) + 0.027\frac{C}{F}\right)$
Sludge Drying Beds	0.0165 lb Sludge/Day/sq ft	Area (sq ft)	--	$1.2\frac{S}{Q}\left(0.21 + \frac{29.7}{S^{0.5}}\right)$
Sludge Incinerator	Solids/Day (lbs)	Solids/Day (lbs)	$\frac{S}{24,000}(170) + 7.15S^{0.61}$	$1500 + \frac{6,450}{Q^{0.63}}$
Chlorinator	Q	Q	$11.6 \times Q^{0.47}$	--
Tertiary Treatment	Q	Q	--	--
Control House	--	--	$51.6 \times Q^{0.7}$	--

Legend

C	=	Capital cost
F	=	Vacuum filter area (100 sq ft)
Q	=	Flow rate (mgd)
S	=	Total sludge production (lb/day)
SA	=	Surface area (1000 sq ft)
V	=	Volume (mgd or 1000 cu ft; see Cost Base)
*	=	Includes final clarified, sludge return blower; excludes power cost.
**	=	Includes thickener and sludge handling.

The break-even distance between two communities beyond which joint treatment is no longer economical may be calculated with the help of the following formula:

$$L = \frac{T_c + T_r - T_{cr}}{Q_c \cdot C_s(Q_c)}, \text{ where:}$$

L = Break-even pipeline length

T_c = Total cost of treatment at contributor community

T_r = Total cost of treatment at receiving community

T_{cr} = Total cost of treatment in combined plant

Q_c = Volume flow from contributing community (mgd)

C_s = Total cost of gravity sewers (¢/Kgal/mile)

$C_s(Q_c)$ = Total cost of gravity sewer at the average flow Q_c.

The corresponding equation for force mains follows:

$$L = \frac{T_c + T_r - T_{cr} - Q_c\, C_{ps}(Q_c)}{Q_c\, C_{fm}(Q_c)}, \text{ where:}$$

C_{ps} = Total cost of pump stations for force mains (¢/Kgal)

$C_{ps}(Q_c)$ = Total cost of pump stations for force mains at the average flow Q_c

C_{fm} = Total cost of force mains (¢/Kgal/mile)

74-13: V, 22-28

25. Land Disposal of Wastewater

This technique is familiar to the Corps of Engineers as an inexpensive and advantageous means of disposing of sewage while irrigating and improving soil fertility. It has been used for generations in Europe and has proved its value. The same method can be used for sludge disposal; however, sludge can be made more easily assimilable to the soil by mixing with sewage. Large acreages of land could be irrigated and fertilized, while a high concentration of nutrients would be diverted from streams and lakes.

Municipal wastewaters are used for land spraying in a few Western communities with scarce water supplies. Elsewhere, farmers may resist land spraying because of the uncertainties involved. Technical and economic problems include: soil characteristics, build-up of salts and heavy metals, odor problems, land availability and cost, initial spraying system cost, amortization, upkeep, and ultimate effects on surface and ground water quality.

While land application systems are viable alternatives to water-based disposal, IWR has sponsored no economic studies in that field. Research has been performed by the Corps of Engineers under its "Waste Water Management" and "Urban Studies" programs. 74-13: V, 28-30

26. River Bed and Lake Purification

Dredging of excess sediment and anaerobic sludge from the beds of flowing water bodies may be desirable. Sediment accumulations can cause trouble to navigation channels, water intakes, and other installations along waterways. Sludge and benthic biomasses rob the water of DO. Periodic dredging may check these ills.

Lakes and impoundments have additional problems due to their stagnant characteristics. They are subject to cumulative pollution. A number of measures can be taken to arrest and hopefully reverse that trend. Lake water aeration may be the best method of combating diffuse discharges, most of which are natural and not subject to treatment at the source. 74-13: V, 32-33

27. Excess Vegetation Control

Excessive algal and other vegetable growths may be caused by nutrients such as carbohydrates, humus, BOD, and other organic matter as well as by phosphorus and nitrate. Comprehensive measures for their control include the removal of all such matter from effluents. The activated sludge and biological sludge process of phosphorus removal, which has been found capable of removing 90% of total phosphorus in raw municipal waste, promises to be significantly cheaper than other treatment processes. The three most feasible nitrogen removal processes are: air stripping of ammonia, ion exchange, and biological nitrification and denitrification.

Once the inflows are under control, it may be advisable to proceed to a one-time massive harvesting of algal blooms and other excessive growths. After that, a 2 ppm concentration of copper sulfate should be effective in avoiding further infestations. 74-13: V, 33-35

28. Thermal Pollution Control

The most effective means for reducing thermal pollution are cooling ponds or lagoons, and cooling towers. Both lend themselves to partial or total recycling of cooling water. Both expand the number of favorable power plant sites. Too many cooling ponds or cooling towers, by warming up the atmosphere could alter the climate.

There are sixteen types of cooling towers. Large wet natural draft cooling towers have been used in Europe for over 50 years. The following tables show costs of cooling devices:

Capital Costs of Water Cooling Devices per KW Produced

Cooling Device	Capital Cost ($/KW) Source 1	Source 2	Source 3
Run of river cooling	5	1.0	-
Bay or lake cooling	6	3.5	-
Cooling pond	10	-	-
Cooling towers			
Wet induced draft	-	3.2	7
Dry induced draft	-	-	27
Wet natural draft	7½-11	7.2	11
Dry natural draft	22	-	25

Note: Plant sizes are 1,800 MW (Source 1), and 150 MW (Source 2); no size was given by Source 3.

Estimated Number of Cooling Towers in the U.S. and Aggregate Investment to Year 2000

Cooling Tower	Estimated Number of Towers	Aggregate Investment ($ billion)
Wet induced draft	380	11
Dry induced draft	0	--
Wet natural draft	540	16
Dry natural draft	1,880	60
Total	2,800	87

Capital and operating costs for individual units of the above types of cooling towers are as follows:

Capital and Operating Costs per Cooling Tower Unit

Cooling Tower	Capital Cost ($1000)	Annual Cost ($1000)
Wet induced draft	28,715	6,453
Dry induced draft	32,305	7,283
Wet natural draft	29,580	6,476
Dry natural draft	31,905	7,217

Note: The above costs are for cooling towers serving a 200 MWe power plant.

The cost of recirculation cooling has been estimated at 1% of the total cost of power generation and distribution. 74-13: V, 35-38

29. Radioactivity Control

One of the main dangers of radiation is that emitted by radioactive substances discharged to streams and lakes. Through the food chain, they may penetrate the human body. Human health can be safeguarded against this insidious threat by: (1) tighter controls in nuclear reactors to eliminate the possibility of leaks; (2) water treatment for the removal of radioactive substances.

A more definitive answer to these problems is the development of so-called "clean reactors." This will hopefully come to pass before the end of the century. 74-13: V, 38-39

30. Municipal Treatment Charges to Industry

The formula used by New York City for determining treatment charges to industrial customers follows:

$$D = CFV((SS - 350) + (BOD - 300)),$$

where: D = Waste disposal surcharge (in $)

C = Cost, per lb, of treating wastes (recalculated every year; was $0.025 in 1970)

F = Conversion factor to transform mg/l to lbs/million cu ft

V = Wastewater volume (in cu ft), determined by the water meter minus a retention factor for water consumption

SS = Suspended solids (in mg/l)

BOD = Biochemical oxygen demand (in mg/l)

Note: SS and BOD are borrowed from SIC averages.

74-13: V, 42.

31. Waste Treatment Cost Allocation

How should public wastewater treatment costs be apportioned among beneficiaries? Nine cost allocation formulas, shown in the following table, have different effects on water users, property owners, and the general public:

Wastewater Treatment Cost Allocation
Under Nine Formulas

Allocation Formula	Cost Allocation in Percent		
	Water User	Property Owner	General Public
1. Public Utility	100	--	--
2. Relative Use	74	26	--
3. Added Expense	62*	38*	--
4. Alternative Revenue	60	27	13
5. Historical	51	25	24
6. Joint Committee	45	55	--
7. Capital and Operating Costs	40	60	--
8. Differential Benefit	30	50	20
9. Diffused Benefits	--	--	100

* Plus or minus 5%

Participation in collective treatment facilities also poses problems of cost allocation. Five cost allocation formulas are: the Measure of Pollution Method, the Rebate Proportional to Pollution Method (indicated for small numbers of pollutants), the Alternative Cost Method (likewise for small numbers), the Free Market Bargaining Method, and the Bargaining-with-the-Regional-Authority Method (indicated for larger numbers of pollutants).

74-13: V, 40-42

C. Water Quality Benefits

1. The With-Without Criterion

A basic principle for evaluating the potential benefits of a public project is the "with-without" criterion. The state of the nation in the event the project is built and operated should be contrasted with the state of the nation in the absence of the project. This is not the same as looking at the state of the nation before and after the project. 74-13: VI, 2

2. Willingness to Pay

To measure benefits of improved water quality by the method of willingness to pay, a representative sample of water customers should be interviewed to determine the aggregate amount the population would be willing to pay for the higher quality. If willingness to pay is determined for different water qualities, a curve relating quality to willingness to pay can be drawn. The marginal benefit is the slope of that curve; it equals the amount of benefit generated by a unit increase in quality from a given quality. Marginal benefit is compared with marginal cost to determine up to what quality level it is economically advantageous to pursue water purification.

The marginal benefit, when multiplied by the total increase in water quality, may not be representative of the entire willingness to pay. The difference is known as the consumer surplus. The computation of the consumer surplus requires knowledge of the aggregate willingness to pay. When figuring benefits by the method of willingness to pay rather than by marginal

benefits and quality provided, consumer surplus is automatically included.

Many difficulties arise when willingness to pay is used as a measure of benefits. Changes in water quality are not readily discernible by the user. The user's preference varies according to who, in his view, is to pay for the improvement. Attempts have been made to establish potential users' willingness to pay by inference, for example by travel cost and time expended in reaching a recreation facility. That approach has its own limitations. A population's willingness to accept an offer for the construction of a water quality improvement construction project would be a more dependable measure of benefits.

74-13: VI, 3-6d.

3. Alternative Cost

As a measure of gross benefits, alternative cost supersedes willingness to pay as soon as an alternative project is authorized which would be constructed in the event the public project is not implemented. Only in the absence of an alternative project should willingness to pay be resorted to as a substitute.

When a government project provides goods or services that would have been provided by private enterprise, the entire reduction in private cost is attributable to the government project as a gross benefit. If there is a difference in the amount of goods or services provided, differential benefits (evaluated by willingness to pay) are either added to or subtracted from the private cost, depending on whether the government project provides a greater or lesser amount of goods or services, respectively. Net benefits are obtained by subtracting gross government costs. 74-13: VI, 6e-9

4. Damages Avoided

Water quality is primarily a matter of avoiding costs, either damage costs (incurred when water of inferior quality damages clothing, plumbing, or even health), or treatment costs (incurred to improve water quality). Improvements in water quality obtained by assuming treatment costs lead to reductions in damages caused by the use of water of inferior quality. To the extent that such damages can be avoided, a benefit accrues from the treatment operation. The benefit can be measured by the reduction in damage costs. With respect to benefit generation, damages avoided are in the same category as alternative costs avoided. Damages avoided are measured in terms of the costs that would have been incurred had the treatment operation not been undertaken. Thus, the with-without criterion is met, and damages avoided are alternative costs.

74-13: VI, 7 & 9

5. Land Values

Increases in land values have been used as partial measures of benefits associated with increases in water quality. In general it is difficult to separate the portion of the land value attributable to water quality. If such a value can be determined, then it represents a minimal estimate of the value of clean water, since consumer surplus is not included in the selling price of land but is properly included as a benefit. 74-13: VI, 9-10

6. Stochastic Nature of Water Quality

At any point in a lake or stream, water quality varies constantly. This makes it doubly difficult to evaluate from static water quality data the true benefits of water quality improvement. Yet, with the present lack of sophistication in techniques for measuring benefits, the variability in water quality can scarcely be taken into account, and its real effect on benefits remains unknown.

When benefits are used in calculations, expected values are appropriate. The probability distribution of critical water quality parameters must be known to determine the expected value of benefits. The sum of the products of the probability of occurrence of successive brackets of water quality and the benefits expected within each respective bracket measures total expected benefits.

Other problems of uncertainty exist and can be handled. To deal with variance in streamflow, a normal distribution of the variance is assumed. If standards are to be met a large fraction of the time, the size of that fraction and the streamflow variance will determine a critical value, f_o, for flow. If treatment is designed so that standards are satisfied at this critical flow, then they will be met the required fraction of the time. Variance in streamflow increases the required treatment of wastes. The standard deviation of streamflow in the U.S. has been estimated at 25% of the mean. 74-13: VI, 10-11

7. Time Discounting of Benefits

Time differentials may complicate the use of alternative cost to determine benefits. Consider the case in which a Corps proposal is to construct a reservoir some 10 years prior to the time the identical reservoir will otherwise be constructed by another agency, public or private. Certainly the results of the two alternatives are the same, but only after a ten-year period. This problem is accounted for as follows: If the Corps builds now, the alternative cost is the cost of the same project at the time it would have been built, discounted to present value. Because the projects in this case are identical, if C is the cost of the Corps project, then $(\frac{1}{1+i})^{10} \cdot C$ = the alternative cost, where i is the appropriate discount rate.

However, the benefit streams accruing to the alternatives are also different. After 10 years, the benefits will be assumed identical, but during the first ten years benefits will accrue to the Corps project and not the private project. The willingness to pay for these additional benefits must be added in the gross benefits of the Corps project. If B is the present value of the first 10 years' benefits, then the gross benefit of the Corps project equals:

$$((\frac{1}{1+i})^{10} \cdot C) + B$$

74-13: VI, 12-13

8. Benefits of Preserving Irreplaceable Resources

Problems of irreversible commitments of natural resources can complicate the computation of costs and benefits. Benefits of preserving unique resources should be compared not with the value of the proposed project which destroys

them, but with the cost penalty incurred for placing the project elsewhere or accomplishing its results in some other way. In the case of the proposed dam across Hell's Canyon, it was pointed out that while the Canyon provides some $900,000 of recreational benefits per year, benefits which would be lost if the Canyon were to be dammed, the additional cost of generating the electricity not provided by the dam, using a steam-electric plant, would be only $80,000 per year. 74-13: VI, 13

9. Benefits Accruing to M + I Water Customers

The health hazard incident to unsanitary drinking water has been greatly reduced in the U.S. by technological advances in the design of water works. As a result, the effects of water quality in promoting human health may play but a small role in determining benefits of water quality projects. Among other benefits likely to accrue to water supply users is the cost avoided from a reduction in repair and replacement of water appliances and plumbing facilities when total dissolved solids or hardness are reduced.

74-13: VI, 14-16

10. Benefits Accruing to Patrons of Water-Based Recreation Facilities

The highest damages to recreation are wrought by pollution resulting in prohibited use of existing resources and facilities. The usual measure of benefits derived from water quality improvement is willingness to pay, or actual expenditures by the public for utilizing recreational opportunities.

74-13: VI, 16

11. Benefits Accruing to Users of the Waste-Assimilative Capacity of Water Bodies

With strict limitations imposed on wastewater dischargers, the most useful property of rivers and lakes -- the conveyance and disposal of wastes -- may be lost forever. Should water quality standards replace effluent limitations, any water quality improvement to a level above the standard would permit limited use of a water body for waste disposal. Benefits of water quality improvement could then be measured by the cost avoided of having to treat effluents. Users would be willing to pay any amount below that cost for the advantage of not having to treat their wastes. 74-13: p. VI, 17.

12. Marginal Costs and Marginal Benefits

Economic theory states that if net benefits are to be maximized, marginal benefits should equal marginal costs. With respect to water quality (Q), the derivative of benefits (B) minus the derivative of costs (C), equals the derivative of net benefits (N), which must equal zero.

$$\frac{dB}{dQ} - \frac{dC}{dQ} = \frac{dN}{dQ} = 0$$

From this equality can be inferred:

$$\frac{dB}{dQ} = \frac{dC}{dQ}.$$

Provided the rates of change in benefits and in costs are continuous, maximum net benefits occur when these rates of change are just equal.

Where cost schedules expressed in terms of water quality are available, marginal costs can readily be computed: Divide the cost differential between consecutive increments of water quality enhancement by the units of enhance-

ment. The marginal cost is the slope of the line joining cost points on a graph.

When cost schedules are not available, but one cost for a specified size of facility is given, a vicarious method for computing the marginal cost can be used -- provided some relation between facility size and quality level is known. This is the 0.6 power rule, which states that the ratio of the costs of two facilities is equal to the ratio of their sizes raised to the 0.6 power. If X_1 is the size of facility 1, X_2 the size of facility 2, C_1 the cost of facility 1, and C_2 the cost of facility 2,

$$\frac{C_1}{C_2} = \frac{X_1^{0.6}}{X_2^{0.6}}$$

This equation can be solved for C_2:

$$C_2 = \frac{C_1 X_2^{0.6}}{X_1^{0.6}}$$

The latter equation can be used to generate a schedule of costs from which marginal costs can be computed. However, by taking the derivative with respect to X_2 of this equation, the marginal cost of a facility of size X_2, given the size of a particular facility X_1 and its cost C_1, can be obtained directly:

$$\frac{dC_2}{dX_2} = \frac{0.6\ C_1 X_2^{-0.4}}{X_1^{0.6}}$$

When benefits of enhancing water quality can be computed from alternative costs or damages avoided, marginal benefits can be determined in the same manner as marginal costs. When this is not possible, benefit schedules can sometimes be derived from users' willingness to pay.

Marginal costs may be equal to marginal benefits and yet net benefits may be negative: in that case, the project should not be built. Marginal costs may equal marginal benefits at more than one point: then, the degree of quality enhancement should be that at which (1) marginal costs equal marginal benefits, and (2) net benefits are greatest, or costs plus damages are least.

74-13: VII, 1-5

D. Water Quality Optimization

1. Optimal Water Supply Augmentation Involving Desalination

Problem: A community has a water supply of 10 mgd with 500 ppm of TDS. An incremental 5 mgd is needed but the only available water source has 3000 ppm of TDS. The community plans to build a distillation plant to demineralize a portion of its total water supply, blending the product water with more brackish water and/or the existing water supply. Costs plus damages should be minimized.

Solution: A table is first prepared to derive the water supply quality, in ppm of TDS, for incremental quantities of 1 mgd distilled. With 0 mgd distilled, the resulting quality will be 1333 ppm. With 5 mgd distilled, the quality is 333 ppm; with 10 mgd, 167 ppm. A distillation cost schedule can be developed from a three-point curve representing desalination costs in ¢/Kgal for three orders of magnitude of plant capacity: a 1-mgd plant can distill water for 80 ¢/Kgal, a 10-mgd plant for 50 ¢/Kgal, and a 100-mgd plant for 40 ¢/Kgal.

For incremental quantities of 1 mgd distilled, and corresponding resulting water qualities, costs in $/day are read off the three-point curve; cost increments between consecutive quantities distilled are determined and divided by salinity differentials to obtain marginal costs in $/ppm/day.

A benefit schedule and marginal benefits are developed from a study indicating a $720 damage cost per mgd for a differential in salinity from 250 to 1750 ppm of TDS. From curves in that study, damages are read for 500, 750, 1000, 1250, and 1500 ppm. After benefits and marginal benefits are established for 250-ppm increments from 250 to 1750 ppm of TDS, this information is converted to the basis of water quantity desalted, in increments of 1 mgd.

When marginal benefits are compared with marginal costs, their equality occurs at 7.5 mgd of water desalted (provided the brackish water is desalted first), and at a corresponding water supply salinity of 250 ppm. When damage is added to cost, the lowest sum occurs likewise at 250 ppm.

74-13: p. VII, 5-11.

2. Optimal Water Supply Protection Through Reaeration

Problem: Eight industrial polluters discharging a total of 10 mgd of liquid waste into a river are being sued for damages by a municipality located downstream which draws its 5 mgd water supply from the river. The municipality's damage schedule is based on its extra treatment costs to remove BOD. With 0% BOD removed, the damage is $1313/day. If the polluters remove 97% of the BOD, no damages will be charged. What is the optimum level of BOD removal that minimizes costs plus damages? The polluters who are planning to install artificial instream aerators into the river need to know how many units to order.

Solution: When municipal treatment costs, damages charged, annual and daily costs of reaerating (based on data presented in VII, B, 2 and 18), marginal benefits ($/% removal/day), and marginal costs ($/% removal/day) are developed

for successive brackets of BOD removal, the results indicate that the use of 22 reaeration units, with a daily cost of $593 for 97% BOD removal and a daily gross benefit of $1313, yields a daily net benefit of $720. The benefit-cost ratio is 2.21. Marginal benefits and marginal costs are equated at two levels: 88% BOD removal (with 12 aerators) and 97% (with 22 aerators). What will help the polluters decide between these two solutions is the measurement of the lowest cash outlay per day. They must pay damages and/or reaeration costs. The minimum sum of the two occurs at 97% BOD removal, when damages are zero and costs, $593/day. 74-13: VI, 11-14a

3. Optimal Quality Storage Reservoir Releases

By virtue of PL 92-500, Section 102, quality storage can no longer be substituted for waste treatment at the source. On the other hand, treatment at the source is insufficient to clean rivers and lakes. Reservoirs can improve instream water quality during low flow periods. Reservoirs can also degrade water quality when the dams reduce sedimentation, when evaporation increases salinity, or when stratification raises water temperature and lowers DO content. Several studies of the Potomac River Basin and Estuary have produced specific release schedules capable of maintaining a given DO level. One author compared the cost of several methods for meeting the 4 ppm of DO objective for the Potomac Estuary:

Low flow augmentation	$115 million
Effluent staggering	85 million
Reoxygenation	29 million

For details, see 74-13: IV, 6; and IV, 45, references (9) and (10).

If the reoxygenation cost was based on mechanical surface aerators, then the same result can probably be achieved at still lower cost through appropriate configurations of gravity-flow, washboard-type aerators.

Other studies have developed a linear programming approach for determining the minimum size reservoir which will simultaneously meet water requirements for water supply, flood control, and recreation. Because of the stochastic flow regimen, the solution is presented in terms of reservoir design and operating policy that will meet requirements a high percentage of the time.

74-13: IV, 6; VII, 15-17

4. Optimal Water Supply Allocation from Multiple Sources with a Single Water Quality Parameter

Problem: What is the optimal water allocation from multiple sources to meet requirements (R) of 60, 65, 70, 75, or 80 mgd of water with a quality (r) of 800, 700, 600, 500, 400, 300, or 200 ppm of TDS? Data are:

Source	Quantity	Quality
Existing water supply	S = 40	s = 500
Fresh water source 1	F-1 = 24	f-1 = 1,200
Fresh water source 2	F-2 = 12	f-2 = 1,000
Renovated wastewater	W = 30	w = 850
Distilled sea water	D = infinity	d = 50

Sea water must be distilled, but fresh water sources 1 and 2, and renovated wastewater can all be desalted by a membrane process to 500 ppm of TDS.

Solution: Thirty-five tables are first constructed on the pattern of three schedules presented in Section VII, B, 14. The need for desalination is tabulated below in summary form:

Quantities of Water in Need of Desalination (mgd)

Quality (ppm)	Quantitative Water Supply Requirement 60	65	70	75	80
Membrane Process					
800	--	--	--	--	--
700	--	--	--	--	--
600	8	12	16	20	24
500	20	25	30	35	40
400	6	10	14	18	22
300	--	--	--	--	--
200	--	--	--	--	--
Distillation Process					
800	--	--	--	--	--
700	--	--	--	--	--
600	--	--	--	--	--
500	--	--	--	--	--
400	14	15	16	17	18
300	27	29	32	35	38
200	40	44	47	50	54

Desalting costs can be converted into cost schedules from the following 3-point curves:

Capacity (mgd)	Membrane (¢/Kgal)	Distillation (¢/Kgal)
1	50	80
10	30	50
100	20	40

Costs are computed from the curves for quantities from 1 to 10, and from 10 to 100 mgd:

Desalination Cost Schedules

Quantities (mgd)	Membrane Costs ($/day)	Distillation Costs ($/day)
1	500	800
2	850	1,320
3	1,180	1,830
4	1,490	2,320
5	1,780	2,800
6	2,060	3,270
7	2,320	3,720
8	2,560	4,160
9	2,790	4,590
10	3,000	5,000
20	5,100	9,100
30	7,200	13,200
40	9,200	17,200
50	11,100	21,100
60	13,000	25,000
70	14,800	28,800
80	16,600	32,600
90	18,300	36,300
100	20,000	40,000

From the above table, daily dollar costs can be computed for the 35 alternatives:

Desalination Costs for Each Alternative
($/day)

Quality (ppm)	Quantitative Water Supply Requirement 60	65	70	75	80
	Membrane Process				
600	2,560	3,420	4,260	5,100	5,940
500	5,100	6,150	7,200	8,200	9,200
400	2,060	3,000	3,840	4,680	5,520
	Distillation Process				
400	6,640	7,050	7,460	7,870	8,280
300	11,970	12,790	14,000	14,800	16,400
200	17,200	18,760	19,930	21,100	22,660
	Total Desalting Cost				
600	2,560	3,420	4,260	5,100	5,940
500	5,100	6,150	7,200	8,200	9,200
400	8,700	10,050	11,300	12,550	13,800
300	11,970	12,790	14,000	14,800	16,400
200	17,200	18,760	19,930	21,100	22,660

From the cost curves used for Section VII, D, 1, a schedule of damages and benefits is developed:

Damages and Benefits

Salinity (ppm of TDS)	Damage ($/mgd)	Benefit ($/mgd)
800	420	0
700	375	45
600	320	100
500	250	170
400	165	255
300	65	355
200	0	420

From the above schedule can now be developed a gross benefit schedule. In order to avoid uncertain and future benefits, it is assumed that any water not needed at project completion can be sold to industry at a uniform rate of 20¢/Kgal, or $200 per million gallons. The income is reflected in gross benefits:

Gross Benefit Schedule ($/day)

Quality (ppm)	Quantity (mgd) 60	65	70	75	80
800	0	1,000	2,000	3,000	4,000
700	2,700	3,925	5,150	6,375	7,600
600	6,000	7,500	9,000	10,500	12,000
500	10,200	12,050	13,900	15,750	17,600
400	15,300	17,575	19,850	22,125	24,400
300	21,300	24,075	26,850	29,625	32,400
200	25,200	28,300	31,400	34,500	37,600

Net benefits and benefit-cost ratios can be derived from cost and benefit schedules:

Net Benefits ($/day)

Quality (ppm)	Quantity (mgd) 60	65	70	75	80
800	0	1,000	2,000	3,000	4,000
700	2,700	3,925	5,150	6,375	7,600
600	3,440	4,080	4,740	5,400	6,060
500	5,100	5,900	6,700	7,550	8,400
400	6,600	7,525	8,550	9,575	10,600
300	9,330	11,285	12,850	14,825	16,000
200	8,000	9,540	11,470	13,400	14,940

Net benefits, as may be noted in the above table, tend to increase with quantity and purity of the water. The 300 ppm quality level shows the highest net benefits for all quantities. Among these, an 80-mgd supply yields the highest net benefit of all, $16,000 per day. Because of the credit earned from the sale of excess water, net benefits always increase with quantity. Benefit-cost ratios appear below:

Benefit-Cost Ratios

Quality (ppm)	Quantity (mgd) 60	65	70	75	80
800	--	inf.	inf.	inf.	inf.
700	inf.	inf.	inf.	inf.	inf.
600	2.34	2.19	2.11	2.06	2.02
500	2.00	1.96	1.93	1.92	1.91
400	1.76	1.75	1.76	1.76	1.77
300	1.78	1.88	1.92	2.00	1.98
200	1.47	1.51	1.58	1.64	1.66

Ratios are favorable throughout the range of quantities and qualities. Ruling out infinity (when benefits accrue at no cost), benefit-cost ratios range from 1.47 to 2.34, both extremes occurring at 60 mgd. Ratios decrease as quality increases, with a temporary recovery at the 300 ppm level.

The next two tables, derived from desalting cost and benefit schedules, show marginal costs and marginal benefits:

Marginal Cost Matrix

Costs horizontally opposite quality levels are in $/day; marginal costs are in $/ppm/day

Quality (ppm)	Quantity (mgd) 60	65	70	75	80
800	0	0	0	0	0
Marginal cost	0	0	0	0	0
700	0	0	0	0	0
Marginal cost	25.60	34.20	42.60	51.00	59.40
600	2,560	3,420	4,260	5,100	5,940
Marginal cost	25.40	27.30	29.40	31.00	32.60
500	5,100	6,150	7,200	8,200	9,200
Marginal cost	36.00	39.00	41.00	43.50	46.00
400	8,700	10,050	11,300	12,550	13,800
Marginal cost	32.70	27.40	27.00	22.50	26.00
300	11,970	12,790	14,000	14,800	16,400
Marginal cost	52.30	59.70	59.30	63.00	62.60
200	17,200	18,760	19,930	21,100	22,660

Marginal Benefit Matrix

Benefits horizontally opposite quality levels are in $/day; marginal benefits are in $/ppm/day

Quality (ppm)	Quantity (mgd) 60	65	70	75	80
800	0	1,000	2,000	3,000	4,000
Marginal benefit	27.00	29.25	31.50	33.75	36.00
700	2,700	3,925	5,150	6,375	7,600
Marginal benefit	33.00	35.75	38.50	41.25	44.00
600	6,000	7,500	9,000	10,500	12,000
Marginal benfit	42.00	45.50	49.00	52.50	56.00
500	10,200	12,050	13,900	15,750	17,600
Marginal benefit	51.00	55.25	59.50	63.75	68.00
400	15,300	17,575	19,850	22,125	24,400
Marginal benefit	60.00	65.00	70.00	75.00	80.00
300	21,300	24,075	26,850	29,625	32,400
Marginal benefit	39.00	42.25	45.50	48.75	52.00
200	25,200	28,300	31,400	34,500	37,600

As may be noted from comparing the last two tables, marginal benefits generally exceed marginal costs. Exceptions are: Between 700 and 600 ppm, for 70 mgd and more; and between 300 and 200 ppm, for all quantities. Marginal costs and marginal benefits are equated, for 60 and 65 mgd, at 300 ppm only; for 70, 75, and 80 mgd, at 600, 500, and 300 ppm of TDS. For the latter quantities, quality enhancement should not be brought below 600 ppm unless it is intended to proceed further to below 500 ppm. The result generally confirms the signal given by net benefits.

For the summation of damages and costs, a schedule of damages is derived from the schedule of damages and benefits, where damages are shown in dollars per mgd.

Damage Schedule ($/day)

Quality (ppm)	Quantity (mgd) 60	65	70	75	80
800	25,200	27,300	29,400	31,500	33,600
700	22,500	24,375	26,250	28,125	30,000
600	19,200	20,800	22,400	24,000	25,600
500	15,000	16,250	17,500	18,750	20,000
400	9,900	10,725	11,550	12,375	13,200
300	3,900	4,225	4,550	4,875	5,200
200	0	0	0	0	0

Sum of Damages and Costs ($/day)

Quality (ppm)	Quantity (mgd) 60	65	70	75	80
800	25,200	26,300	27,400	28,500	29,600
700	22,500	23,375	24,250	25,125	26,000
600	21,760	23,220	24,660	26,100	27,540
500	20,100	21,400	22,700	23,950	25,200
400	18,600	19,775	20,850	21,925	23,000
300	15,870	16,015	16,550	16,675	17,600
200	17,200	17,760	17,930	18,100	18,660

For all quantities, totals are lowest at the 300-ppm quality level. Totals invariably increase with quantity, despite the special credit given for selling excess water. It should be kept in mind, however, that damages are not instant, out-of-pocket expenditures by the utility, but rather diffuse costs to water customers over a period of time. Thus, net costs may be more significant than damages plus costs.

To summarize, net benefits favor a supply of 80 mgd with 300 ppm of TDS; benefit-cost ratios are generally favorable; marginal benefits equal marginal costs at 300 ppm for all quantities and, in addition, for 70-80 mgd, at 500 and 600 ppm; damages plus costs are lowest at 300 ppm for all quantities; net outlay, after deduction of income from the sale of excess water, is essentially proportional to quality improvement, regardless of quantity. Trade-offs between quantity and quality are possible. Final decisions, made by city fathers and government officials, are generally the result of compromise. 74-13: VII, 17-33

5. Optimal Water Supply Allocation from Multiple Sources with Multiple Quality Parameters

Where more than one quality parameter is to be considered, more sophisticated techniques are necessary. However, important sacrifices must be consented. Two techniques are described below.

In the first technique, no provision is made for upgrading the quality of the water sources. Neither is an attempt made to determine net benefits, or the sum of damages and costs. Although quantity and quality levels can be relaxed from set constraints after cost minimization has been achieved, it is

necessary to decide in advance what the total water quantity and the concentration of each quality parameter in the blended supply shall be. Cost limitations are not taken into account at first. Thus the technique is round-about, and may require a good deal of manipulation and iteration of computer runs, if all aspects of the problem are to be weighed against one another for an acceptable compromise.

The problem is stated as follows: Given N alternative sources of raw water, each with a known maximum supply, each containing a given concentration of up to M types of impurities, and available at a known cost per gallon -- minimize the cost of providing a given total amount of water with a maximum concentration of each of the M impurities.

Define:

Q_i = the maximum yield of source i (gpd)

c_{ij} = the concentration of impurity j in water from source i

T = total amount of water required (gpd)

K_i = the cost of water from source i ($/gpd)

C_j = the maximum allowable concentration of impurity j in the final blended water

q_i = the amount of water to be taken from source i (to be determined) (gpd)

The entire problem can be stated mathematically as:

Minimize $\sum_{i=1}^{N} q_i K_i$, subect to:

$$\sum_{i=1}^{N} q_i \geq T$$

$$q_i \leq Q_i \qquad \text{(all i, i = 1 to N)}$$

$$\sum_{j=1}^{M} c_{ij}q_i/T \leq C_j \qquad \text{(all } j,\ j = 1 \text{ to } M)$$

$$q_i \geq 0 \qquad \text{(all } i,\ i = 1 \text{ to } N)$$

The last constraint insures that no water is returned to any of the sources.

This is a relatively simple linear programming problem which can easily be solved on any computer. Canned programs which will solve the problem are available.

Two difficulties are inherent in this approach to the blending problem: It is assumed that the impurities in the water are non-reactive, and that costs of drawing water from any source are proportional to quantities used (economies of scale, substantial in water resource development, are ignored).

The second technique is more comprehensive in that it includes provisions for upgrading water quality. Maximum application is to be made of cascading water reuse (obviating the need for treatment between consecutive uses), but treatment is provided where needed. The difficulty of multiple quality parameters is circumvented by substituting costs of upgrading water quality. All uses within a given area or region are catalogued, with water quality standards and effluent quality determined for each use. The problem is stated as follows:

Given that demands for water exceed primary supply, and that water may be imported or desalted, and knowing also (a) the amount of water required and wastewater generated by every class of water user; (b) the cost of upgrading one user's waste for the use of another, or for reuse; and (c) the cost

of primary, imported, and desalted water available -- determine the least cost method of satisfying all water demands.

The data necessary for the model are presented in tabular and matrix form. Expressed in linear programming language, the problem involves the following:

Define:

C_{ij} = the cost of upgrading the quality of water from source i for use by j

x_{ij} = the amount of water from source i used by user j (to be determined)

M = the number of sources

N = the number of users

A_i = the quantity of water available at source i

B_j = the amount of water required by user j.

The linear programming formulation is:

$$\text{Minimize} \quad \sum_{i=1}^{M} \sum_{j=1}^{N} C_{ij} x_{ij}, \text{ subject to:}$$

$$\left.\begin{array}{ll} (1) & \sum_{j=1}^{N} x_{ij} \leq A_i \\ (2) & \sum_{i=1}^{M} x_{ij} \geq B_j \\ (3) & x_{ij} \geq 0 \end{array}\right\} \quad \left(\begin{array}{l} i = 1 \text{ to } M \\ j = 1 \text{ to } N \end{array}\right.$$

The first set of constraints ensures that no more water is taken from a source than is available at that source, while the second set ensures that all water requirements are met. The third set of constraints forbids negative flow. The objective is simply to minimize total cost.

6. Optimal Effluent Charges

A monetary charge is to be collected by a local authority as an incentive for waste reduction. What will be the effect of a given charge? And what charge will cause polluters to reduce their discharges just enough to meet set quality standards?

The effluent charge, expressed as $/waste unit, is at a maximum when waste is not reduced below an initial level. Savings are a function of waste reduction. If the charge increases linearly with increasing waste discharge, the formula reads as follows:

Define:

TCh = total effluent charge ($/day)

g = constant unit charge ($/waste unit)

FD = initial waste discharge (lb/day of waste)

f = waste reduction below initial level (lb/day of waste)

TS = total savings

The total effluent charge function TCh equals g(FD - f). The maximum charge is given by: $TCh_{max} = g(FD)$. And total savings equal:

$$TS = TCh_{max} - TCh = gf$$

Standard economic theory indicates that a discharger will reduce his discharge to the point where marginal savings equal marginal costs. If the marginal cost curves of all dischargers are known, the uniform effluent charge which will cause polluters to reduce their discharges just enough to meet a given water quality standard can be determined in a manner analogous to

finding the minimum level of uniform treatment which just meets a standard. Try a charge and compute the results. If results are too high or too low, try a lower or higher charge. If the marginal cost curves are not known, determining the lowest charge is considerably more difficult.

74-13: VIII, 2-4

7. Instream Water Quality Management

If the maintenance of a preset level of DO in a water body is the objective, the mathematical description of oxygen depletion (through bacterial breathing) and oxygen generation (through diffusion and algal growth) involves the following steps:

Define:

C_s = temperature-dependent saturation concentration of oxygen

C = actual time-varying concentration of oxygen in the stream

C_o = concentration of oxygen in the stream at time zero

D = oxygen deficit

K_1 = constant called the bio-oxidation rate

K_2 = constant called the reaeration rate

L = concentration of BOD in the stream

L_o = incremental BOD load in the stream

t = time

The rate of oxygen uptake by bacteria equals the bio-oxidation rate multiplied by the BOD load:

$$\frac{dC'}{dt} = K_1 L$$

The reduction in the BOD load in the stream is equivalent to the amount of oxygen uptake by bacteria:

$$\frac{dL}{dt} = K_1 L = \frac{dC'}{dt}$$

As the bacteria remove oxygen, DO also enters the stream by diffusion, mixing, and algal growth, at a rate equaling the reaeration rate multiplied by the oxygen deficit:

$$\frac{dC''}{dt} = K_2 D$$

The overall rate of change in oxygen concentration is the difference between the two preceding terms:

$$\frac{dC}{dt} = K_2 D - K_1 L$$

When the above equations are solved for D, the classical Streeter-Phelps equation results:

$$D = \frac{K_1 L_o}{K_2 - K_1} \left(e^{-K_1 t} - e^{-K_2 t}\right) + (C_s - C_o)\, e^{-K_2 t}.$$

Being a linear function of L_o, D is suitable for use in linear programming models for minimizing cost. 74-13: VIII, 5-7

8. Optimal Waste Treatment

Linear programs have been designed to optimize effluent treatment. It is known that treatment costs, beyond primary treatment, increase at an accelerating rate with the percent of BOD removal.

From the Streeter-Phelps equation (see Section VII, D, 7) can be derived the water quality improvement in a reach of a stream resulting from a unit waste reduction by a discharger. On that basis, a linear programming model can be written:

Define:

N = number of dischargers

j = identification of the discharger

i = identification of the reach of the stream

k = identification of a particular value of reduction

f_j = amount of reduction by discharger j (unknown)

$C_j(f_j)$ = total cost of f_j

K_j = total number of specific values of reduction by discharger j

A_{ij} = water quality improvement in reach i resulting from a unit waste reduction by discharger j

B_i = required water quality improvement in reach i

M = number of reaches with water quality improvement goals

U_{jk} = a particular value of f_j, the reduction by discharger j (known)

FT_j = total waste production by discharger j

Z_{jk} = a weight associated with a particular reduction U_{jk} (unknown)

The function to be minimized is total cost:

Minimize: $\sum_{j=1}^{N} \sum_{k=1}^{K_j} C_j(U_{jk})Z_{jk}$, subject to:

$$\left.\begin{array}{ll}
(1) & \sum_{j=1}^{N} A_{ij} \cdot f_j \geq B_i \\
(2) & \sum_{k=1}^{K_j} Z_{jk} \leq 1 \\
(3) & (1/FT_j) \cdot f_j \geq 0.35 \\
(4) & \sum_{k=1}^{K_j} Z_{jk} U_{jk} = f_j
\end{array}\right\} \quad \left\{\begin{array}{l} i = 1 \text{ to } M \\ j = 1 \text{ to } N \end{array}\right.$$

The first constraint ensures that all water quality improvement goals are met. The second, that the solution follows the cost curve. The third constraint makes certain that each discharger will provide at least 35% removal of waste (primary treatment). The last defines f_j in terms of the weight variable Z_{jk}.

One of the major difficulties with the model just presented is that it often seems quite inequitable to those who must bear the cost of waste treatment. In the least cost solution, it is not uncommon to find that one discharger will have to remove a high proportion of his wastes while a neighbor will have to institute only a moderate reduction at much lower cost. It can be very difficult to implement least cost solutions which are perceived as inequitable.

Equity considerations can be satisfied, at least overall cost, by having those dischargers with only modest treatment requirements subsidize those of whom higher levels are required. Thus, a uniform basis for contributing to the aggregate cost would be substituted for the variations in treatment levels.

Failing an accord of that type, a solution can be found at higher overall cost. A river is divided into zones, with each discharger within a given zone required to provide the same level of treatment. The larger the number of zones, the lower the cost of the optimal solution. For a given maximum number of zones, it is possible to implement a zoned uniform treatment program at least cost through an integer programming formulation. The problem is how best to divide a river into N zones so as to minimize cost.

74-13: VIII, 7-11

9. Optimal Instream Water Quality Maintenance

The 1972 Amendments (PL 92-500) to the Federal Water Pollution Control Act, declaring the national goal to be "that the discharge of pollutants into the navigable waters be eliminated by 1985," preclude the maintenance of instream water quality through alternatives less expensive than full treatment at the source. It is nevertheless of interest to note that such alternatives exist. One linear programming model solved a pollution problem without the need for any treatment, simply through longitudinal staggering of outfalls. While the cost of zoned uniform treatment was cut in half by optimal non-uniform waste treatment, it was cut in four by source and collective treatment combined with staggered outfalls. Diurnal staggering of outfalls would provide additional possibilities of reducing the need for treatment at the source, and artificial instream reaeration, particularly of the corrugated washboard type, may prove the cheapest approach of all.

74-13: p. VIII-11/12.

10. Typical Planning Problems Involving Water Quality Considerations

A fictitious setting might encompass a river basin providing surface water to 3 towns and 5 industrial plants, all of which return their wastes to the river. The present problem may include the following requirements:

1. Two towns need more water;
2. One of these and the third need purer water;
3. Three industries need more water;
4. One of these plus one other need somewhat purer water;
5. The food industries, one of which also needs more water, require much purer water;
6. Much of the river basin's populated area is polluted;
7. One of the river's confluents has a high concentration of dissolved solids during the summer months;
8. There is an urgent need for recreational lake shore facilities.

The situation calls for an analysis of alternative sources from which incremental water supplies and recreational water can be obtained, and of alternative water purification methods to meet the water quality needs in the river basin. Legislation calls for pollution abatement and approved disposal of residuals. Water sources include intrabasin conveyance, ground water development, brackish water desalination, ocean water distillation, cascaded wastewater reuse, low-flow augmentation through gravity and/or pumped storage reservoirs. Purification methods include, in addition to individual waste treatment and residual disposal facilities, quality storage (gravity or pumped), artificial aeration, piping of wastes to the ocean, longitudinal and/or diurnal staggering of outfalls, by-pass piping to collective treatment plants, desalination of raw and/or wastewater, cooling ponds and cooling towers.

Under the existing legislation, what is prohibited is the release of untreated wastewater to a stream, but not the reuse of untreated wastewater so long as it is conveyed in pipelines. This permits cascading reuse as well as recycling.

Significant water quality aspects, within the framework of this typical planning situation, are, for each projected water use, those water quality parameters which occur or are apt to occur in any water source with a concentration exceeding, equaling, or approaching established tolerance levels. To the removal or reduction of each critical parameter is attached a price tag. Desalination costs are much higher than conventional treatment methods.

The solution to the problem involves four steps:

(1) For water quantity and quality, a dependable yield and quality level prevailing, for example, 95% of the time, may be considered acceptable risks.

(2) The water supply allocation problem, quantitative as well as qualitative, may be solved by techniques such as those described in Section VII, D, 4 and 5.

(3) The pollution problem may be solved in accordance with the approach delineated in Section VII, D, 6, 8, or 9.

(4) Recreational needs can be solved after it is known whether a new reservoir will be needed to expand the water supply, and whether instream water quality, available e. g. 95% of the time, will meet recreational standards.

The solution may involve, for example, conjunctive use of some ground water, installation of one small electrodialysis unit to remove excess salinity from a brackish water source, but no new reservoir for M + I water supplies. For instream water quality, the best solution may comprise lagooning for storm water treatment, by-pass piping for collective wastewater treatment, a change in agricultural irrigation techniques to reduce salt concentration in return flows, and instream artificial aeration upstream from a recreational park facility to be located on the river. This solution would be the least expensive of a number of alternatives for meeting the various objectives of the region, and would, therefore, represent a desirable solution.

(From an unpublished report)

VIII

WASTE WATER RENOVATION FOR REUSE

VIII. WASTE WATER RENOVATION FOR REUSE

Source:

An Evaluation of Water Reuse for Municipal Supply, by Daniel Dworkin, Holcomb Research Institute, Butler University, Indianapolis, Ind., and Duanne D. Baumann, Department of Geography, Southern Illinois University, Carbondale, Illinois

IWR Contract Report 74-11 December 1974

Scope and Use

This report investigates waste water reuse as an alternative source of municipal water supply. Renovation and reuse of municipal water, say the authors, is neither a new concept, nor is it an inherently efficient method which should be employed to supply water. Yet, as the years pass and water demand outstrips present resources, it is inevitable that water reuse will be resorted to for certain purposes. Determining under what conditions and limitations this is economically efficient is the object of the research. By means of a mathematical model and simulation techniques, water renovation for reuse is assessed.

In an application to the City of Colorado Springs, Colorado, the simulation shows that reuse can be an efficient and low cost method of supplementing water supply. The simulation can also determine whether present reuse practices are efficient. And alternatives can be formulated which may be more efficient than present methods of reuse.

The possibility of reuse, provided the necessary facilities are in place, has a far-reaching effect on the need for additional water system yield to meet future demand: First, water yield no longer needs to be

understated for safety. Second, reserve capacity in the system can be utilized. Third, as a substitute for high levels of assurance of adequate yield, water reuse availability can actually expand the existing yield. Fourth, water systems with reuse capacity can shorten the planning horizon, meaning that a requirement for increased capacity could be delayed or even canceled.

These are useful considerations for water supply planners. As water demands become more urgent and reuse becomes more widely accepted, a capability for waste water renovation for reuse relaxes the need for immediate and costly investments in additional development of surface or ground water resources.

VIII. WASTE WATER RENOVATION FOR REUSE

1. Rationale for Wastewater Reuse as Potable Supply

Water reuse can be inadvertent and unplanned, as the withdrawal and use of water from a river with an upstream discharger. Planned reuse involves the collection of used water for allocation with or without treatment to irrigation, recreational, industrial, or even domestic purposes. Cascading reuse involves a single treatment procedure just prior to disposal. Other reuse may require intermediate treatment. Reuse as potable water supply requires the highest level of treatment sophistication.

Reuse of waste water as potable supply has been practiced during emergencies. It is not recommended as a 100% satisfactory way of expanding water supply. It is, however, a potentially attractive alternative which should be investigated to determine if it is safe, efficient, and acceptable. To make a valid judgment of the efficiency of reuse, a plan is required for examining the supply, demand, and treatment of water, with options for expanding the system capacity through conventional additions versus reuse.

Reuse provides a source of water which could obviate or delay the need for conventional additions to supply. Furthermore, the presence of a standby source can increase system utilization and provide planning flexibility. Benefits would result from the application of reuse in three areas:

(1) as a substitute for high levels of assurance or reliability of supply;

(2) as a method of mobilizing over-supply resulting from understatement of system yield;

(3) as a method of shortening the planning cycle which would allow day-to-day evaluations of change in demand to replace present long-term projections.

74-11: 6-9

2. Planning for Dependable Water Supply

The yield of a water supply system involving storage of flows is usually expressed as a quantity of water available for a stated percentage of the time or expected probability of occurrence. The availability is of some high order, typically 95% of the time. This concept of safe yield is simple, and while it is often regarded as a deterministic quantity, it is not. The streamflow records which would be required for a statistically satisfactory calculation are not available. As a result, the short record available is used with the assumption that it represents the entire population of flows. Alternatively, a synthetic trace of streamflows is generated which also may not be representative of the actual population of flows encountered during the life of the system.

The engineer who uses the synthetic trace of flows must select the severity of events which will then determine the yield of the system. Engineering handbooks urge a conservative calculation. Social Scientists claim yields

are often understated. Renovation and reuse can provide a standby source which allows the use of present facilities till day-to-day evaluations of the response of the physical system to demands can replace long-range engineering estimates. Standby capacity allows operation of the existing system until excess yield is used.

A given stream and reservoir combination will produce higher yields as the required assurance is relaxed. The normal level of assurance is 95% or higher. To achieve this with conventional means, facilities must be scheduled to be in place when water use equals the assured yield. A period of overinvestment is unavoidable. During that time, the chance of system failure is less than 5%.

Capacity is often added before need catches up with yield. This is done even though shortages could be managed through rationing. Because shortages occurring only 5% of the time are not perceived as a viable alternative, investment is made in facilities that will be utilized only 5% of the time.

An alternative to the expansion of water supply systems is to relax the requirements for high levels of assurance, and thereby raise system capacity. The water required to make up for the lower assurance of yield could be furnished by reused wastewater. Sewage could be processed to potable quality, or some present use of potable water could be displaced and supplied by treated effluent. In the latter instance, the present quality of the treatment plant effluent might be adequate. The distribution of non-potable water to the user would represent an added cost.

3. Projecting Water Demand

The long time required for development of new water sources requires long-term estimation of future water need. In the past, the growth in demand for water has rapidly utilized excess capacity resulting from overestimation of municipal water requirements. However, the rapid growth of water systems nationwide has slowed. A longer period will be needed to use up excess capacity. This will increase costs, since investment in idle capacity will be prolonged.

Water renovation and reuse could shorten the planning cycle. The need for water could be judged by noting increased use over recent periods rather than projecting long-term trends. The needed assurance would be provided by standby reuse facilities.

Water reuse should be investigated to determine whether it is a potentially safe, economically efficient, and socially acceptable alternative water supply technology.

74-11: 15-16

4. Safety of Wastewater Reuse as Potable Supply

There are at present no cities in the United States processing effluent for direct potable reuse. Windhoek, South West Africa, has provided the only long-term example of direct introduction of effluent into the municipal supply. The water produced meets all the standards set by the World Health Organization.

There is a concern that sewage treatment plant operating personnel and equipment may not be able to maintain the degree of reliability required for potable use of the effluent. Even in conventional raw water treatment plants, there continue to be incidents of acute or incipient health hazards because of the presence of potentially dangerous substances or organisms. A study covering over one-third of the population served by surface water supplies in the U.S. estimated municipal effluent in the water supply to vary from 0% to 18%, with a median of 3.5%. The decision not to approve planned reuse for domestic water supply does not preclude unplanned reuse from taking place.

74-11: 16-19

5. Economic Efficiency of Waste Water Reuse

The capacity to recycle water is not without costs. There are many systems in use now where the costs of expanding the water supply through conventional methods would be less than the costs of present techniques of renovating effluent. To judge potential benefits of reuse vs. conventional systems, the availability and cost of conventional additions to supply must be measured as an alternative to reuse.

Some generalizations are possible. Reuse systems are less capital-intensive, but cost more to operate than diversion and storage of flows. Reuse systems are best for use as standby capacity, while stored surface water tends to be less costly if used at or near capacity. This would suggest reuse as a peak-load facility, with storage to provide base load.

Reuse capacity should be provided as a standby system used intermittently, only when reservoir levels are low. This type of analysis has not been per-

formed. The emphasis has been on computing costs of producing potable water from sewage -- even though (1) untreated sewage is not the input, (2) potable water not the only output, and (3) continuous use not the most efficient method of employing reuse.

An assessment of costs and benefits of effluent reuse requires analyzing costs of reuse as well as of alternatives over the planning cycle. Benefits are assumed equal if equal water is supplied.

Once the costs and effectiveness of the processes required to treat sewage are known, the calculations become simple. Comparison with the costs of conventional water supply indicates the relative efficiency of reuse. However, the important consideration is the cost of incremental treatment required above that needed for control of pollution.

What is required is an integrated analysis of a municipal water system in which the costs of expanding and operating the system without reusing water are compared with the same costs in a system which uses treated effluent as a supplemental source of water. The analysis should be extended to include various methods of integrating renovated water into the system.

74-11: 19-24

6. Acceptability of Wastewater Reuse

No program utilizing renovated wastewater can be implemented without public acceptance. In a survey of 300 municipal water managers, the 50%

who opposed wastewater reuse anticipated rejection by the public. In fact, water managers know very little of consumer responses concerning renovated wastewater. There is evidence that the public will accept renovated wastewater for potable use provided they are aware of the technological characteristics of water treatment.

The central question, then, is why managers and engineers perceive the public as unwilling to accept renovated wastewater when available evidence indicates the reverse? Could it be that engineers and public health officials are reluctant to innovate or change the established procedures, making the public a scapegoat? It is upon consulting engineers and public health officials that city fathers and water managers rely for advice on their water supply.

74-11: 24-26

7. The Future of Renovated Wastewater

The shortage of water will make reuse of effluent a certainty for many municipal systems. Water supply costs will rise, either as a result of distance from available sources, or as opportunity costs increase for water transferred from lower order uses to municipal use.

Meanwhile, the cost of providing water for reuse is declining. This is due mainly to the mandated upgrading of effluent discharged by municipal systems. Under the 1970 pollution control program objectives, the federal government set standards that require secondary treatment of all municipal

sewage, and additional treatment for phosphorus and nitrogen control. A product water will result which will require less than 10 cents per 1000 gallons for residual treatment. The 1972 Amendments (PL 92-500), which require the elimination of all pollution discharges by 1985, will provide water for reuse at even lower costs of residual treatment.

74-11: 26-27

8. A Model for Wastewater Reuse

A simulation model has been developed to investigate the reuse of municipal sewage as a supplement to existing sources of water supply. The model can be used for comparing alternative methods of expanding a water supply, municipal reuse included, over a 50-year span. The objective is to provide water for municipal supply at minimum cost for the duration of the projected period. All costs are incremental. By means of the simulation, the following questions can be answered:

(1) Can reuse be an economically efficient method of supplementing water supply?

(2) Are present reuse practices efficient?

(3) Can alternatives be formulated which are more efficient than present methods of reuse?

The first digital simulation models date back to 1953. None of the early programs made any attempt to optimize the theoretical ratio of benefits to costs. It remained for the Harvard Water Program to achieve this goal. In 1962, a hypothetical river basin with four multi-purpose reservoirs was simulated with the objective of optimizing their design and operation.

Harvard's simulation program was also the first of the digital models to use simulated streamflows rather than a historical trace of actual flows. The use of a series of synthetic traces of streamflows rather than of historical records allows simulating a number of alternative sequences, thus providing an opportunity for assessing the outcome of a plan over a wide range of possible flows.

Our model is capable of simulating: the supply and storage of water from streamflows and wells, the demands for water by five water-using sectors, and sewage treatment by secondary and AWT processes for discharge or reuse.

A simple equation for simulating stochastic streamflows includes a term which is the product of the standard deviation of the monthly flow and the standardized normal random deviate with a mean of zero and a standard deviation of one.

There are three problems with this simple model:

(1) Flows tend to persist. Low water tables caused by low rainfall must be raised before normal rainfall produces normal runoff.

(2) Flows are not usually normally distributed, but follow a log-normal or some other distribution variant.

(3) Flows of one stream are interrelated with flows of others.

Model HEC-4 designed by the Corps Hydrological Engineering Center considers these three problems and contain routines reproducing these effects.

The demand for water in a municipal system can be projected through a single model of total municipal use. Models of separate water-using sectors are more sensitive to changes in individual uses. The present model provides

for five different demand sectors, each with a capacity of ten variables. In the domestic water sector, water used within the house is handled separately from that used for garden sprinkling. For the sprinkling water, variables include price, an index for sprinkling requirements, the value of the dwelling, and the number of persons living in a dwelling unit.

The equations for sector demand are formulated by using the historical monthly water use figures as input to a regression program. The residual value from the regressions, the error term, is used to introduce a stochastic value into the monthly sector demand for water. This is done by calculating the mean and standard deviation of the residuals. The mean, which should be close to zero, is added to the equation for demand, while the standard deviation is multiplied by a random normal deviate and added monthly to the value of the derived equation.

The waste treatment system is represented in the model by the following processes: activated sludge, coagulation and sedimentation, filtration, and ion exchange. The processes can be rearranged or replaced by other treatment methods. Peak flow through each process determines the required capacity, while total flow governs operational costs. Capacity and operating costs associated with each process are based on data developed by the Taft Center.

74-11: 47-55

9. Results of Wastewater Reuse Simulation

The mathematical model described under Item 8 above has been used to simulate proposals of the Colorado Springs, Colorado, water department for

future expansion of the city's water supply (the Plan), as well as proposals involving wastewater reuse (the Alternative). The two simulations make cost comparisons possible. The planning period is 50 years beginning in 1974. The existing yield of the water system is 56,200 acre-feet per year; this is to be expanded to 100,000 acre-feet annually by 2024. A high, a medium, and a low population projection are used in the simulations. Five water demand sectors are included: domestic, commercial, industrial, military, and municipal demand.

The Plan and the Alternative are compared by means of six series of simulated streamflow sequences and population projections. The object is to provide water supply to the city at minimum cost for a fifty-year period. Present values are used.

When expenditures are discounted to the present, Plan costs in the first series are $21.5 million, Alternative costs, $7.6 million. In the second series, discounted costs are about the same. In the third series, the simulation of the Plan must be ended after the 36th year when the reservoir runs dry (2010); the Alternative oontinues to furnish water for the full 50 years; costs are $21.1 million (Plan) vs. $10.8 million (Alternative). The Plan fails again, in the third decade, of the fourth and fifth series, while the Alternative continues to furnish water for the entire period; the cost comparison shows about $21 million for the Plan, versus $10.0 million (fourth series) and $12.0 million (fifth series) for the Alternative. In the sixth series, the Plan has a present value of $21.5 million, the Alternative, less than $6 million.

Thus, for every assumed streamflow condition and population projection, the Alternative is a less costly method of supply than the Plan. While the Plan, insensitive to variations in supply and demand, requires a succession

of investments with a nearly constant present value, the Alternative adapts itself to supply and demand changes, with consequent variations in discounted costs.

It is to be noted that the Plan also contemplates employing reuse facilities. However, the timing is not optimal: effluent is treated when reservoirs are oversupplied and spilling. Likewise, investment in reuse facilities (as well as in conventional works) is premature. Under the Alternative, wastewater is processed only when water supply is low, and additions to supply capacity are delayed until the level of water in storage indicates a probable future need.

74-11: 108-144

10. General Principles Governing Wastewater Reuse

Wastewater reuse is advocated as a method of increasing yields provided it is judiciously applied. It can allow demand for water to rise for a period of time without the need for immediate expansion of conventional capacity: with standby reuse capacity, reserve capacity in the system can safely be used. Reuse can also provide a substitute for high levels of assurance of safe yield. This expands the usable yield, since system yield is inversely related to levels of assurance.

In addition, water systems with reuse capacity can shorten their planning horizon. Instead of having to anticipate increased requirements by at least ten years, they can be guided by current trends until a point in time much closer to increased need. This can delay or cancel requirements for increased capacity in case changing conditions are reflected in decreased use projections.

The most obvious generalization that can be made concerning the provision of wastewater reuse capacity is that a program of water renovation should not be undertaken when potable water is spilling from oversupplied reservoirs. At some point when reservoirs stop spilling and levels decrease, renovated water should be used to supply users who are indifferent to receiving stored or recycled water. As requirements for water increase, users may require encouragement to displace potable supply with renovated effluent. Encouragement could take the form of restrictions against the use of virgin flows for specific purposes. The level at which the change-over should take place is a function of future expectations of use and supply.

There is a suspicion that most water systems have yields in excess of engineering estimates. If so, conventional water projects could be delayed until excess capacity is used. Reuse could supply part of the demand if the delay in adding to system capacity should cause a shortage.

The reuse application that seems most productive is the substitution of reuse capacity (which may seldom be used) for high levels of assurance either stated as a 95-98% safe yield or implied in the designation of firm yield. As levels of assurance are relaxed, yield from the same stream and reservoir combinations rises. For every 1% decrease in desired assurance, the reuse plant could be used an additional 1% of the time.

In planning water supply for a community, the system manager has at all times three supply alternatives: (1) withdraw water from storage, (2) reuse wastewater, (3) add to capacity. If these options are exercised effectively, the municipality will be supplied water at a lower cost than if reuse were not one of the planning alternatives.

IX

CONTROL OF ACID MINE DRAINAGE

IX. CONTROL OF ACID MINE DRAINAGE

Source:

Costs and Effects of a Water Quality Program for a Small Strip Mining Company, by G. Richard Dreese, Department of Economics, West Virginia University, Morgantown, W.Va., and Harold L. Bryant, Department of Economics and Finance, Xavier University, Cincinnati, Ohio.

IWR Contract Report 71-7 August 1971

Scope and Use

A small strip mining company operating in Southeast Ohio was confronted by new water quality standards affecting its release of significant amounts of acid pollution. Determining how the problem could be dealt with was the purpose of this IWR-sponsored research. Analysis indicated the nature of the market, the firm's cost schedule as affected by water quality criteria, and the rationale of the firm's response. The response could be to reduce output or shut down, to increase output and capture available scale economies, or to maintain production essentially at current levels.

The investigation showed that the firm could adopt at least one feasible level of acid mine water treatment. That level of treatment excluded some of the more complex processes which could not be justified by the scale of operation of the mine. One problem was that the incremental treatment cost could be passed forward to the consumer only by a firm possessing sufficient market power to practice price leadership. And then only on condition of price-inelastic demand. Under the circumstances, smaller firms could pass forward some or all additional costs only by following the lead of larger firms.

This report demonstrates a technique which may be applicable by Corps planners under special circumstances. If these circumstances arise, the technique may be quite useful.

IX. CONTROL OF ACID MINE DRAINAGE

Can a small strip mine in southeastern Ohio afford to control water pollution and restore disturbed land to meet today's environmental standards?

1. Methods and Techniques

Four practical methods are available to the strip mining operator for controlling acid mine water drainage produced by the coal extraction process:

Method 1 involves an acid water neutralizing facility consisting of a hopper for screw-feeding limestone slurry into a flume of polluted water. For an operation recovering annually 32,000 tons of coal from 10 acres of land yielding 5,430,000 gallons of runoff, the annual cost of the acid-neutralizing operation is $2,333, or $0.43 per Kgal and $0.07 per ton of coal.

Method 2 utilizes gravity flow and diversion around the strip pit. Diversion ditches are cut along the contour above the highwall until the natural channel is intercepted. Ten acres of disturbed land result in 1,760 feet of highwall. The annual cost of highwall construction, scraping, lining, and piping is $7060, or $1.30 per Kgal and $0.22 per ton of coal.

Method 3 is an extension of Method 2. It provides, in addition to the diversion ditches, a ponding basin to handle sediment, debris, and sludge. The additional cost is about $1,000 per year, or $0.18 per Kgal and $0.03 per ton of coal mined. The total cost of Method 3 is thus $1.48 per Kgal and $0.25 per ton of coal.

Method 4 makes use of a mobile acid-neutralization facility treating polluted water with hydrated lime, and disposing of the sludge. It must be capable of handling 200,000 gpd of water containing 1,000 ppm of acidity. The annual cost is $2,880, or $0.53 per Kgal and $0.09 per ton of coal.

Reclamation costs for restoring disturbed land areas depend on the technique used. Three techniques are applicable to a strip mining operation.

Technique 1 consists of grading, backfilling, burial of toxic materials, and reforestation. The annual cost is $1,500, or $0.28 per Kgal and $0.05 per ton of coal.

Technique 2 is very similar, but performs a more complete operation. Annual costs are $2,000, or $0.37 per Kgal and $0.06 per ton of coal.

Technique 3 provides for blasting the highwall to a 45° angle, grading to the original contour, and adequately seeding for minimum erosion. Its annual cost is $5,000, or $0.92 per Kgal and $0.16 per ton of coal.

71-7, p. 29-60.

2. The Demand Function

The demand for coal from the area surrounding the strip mine equals the industry demand curve. Electric power utilities are by far the largest clients. Their preferred procurement method is by means of a 20 to 30-year contract for the total production of a large mining firm. Such contracts are supplemented by purchases from small operators at identical prices. Small operators, who usually sell their product through coal brokers, also supply public institutions.

The degree of elasticity of the demand for coal in the area served by the small strip mine is difficult to ascertain. Coal users cannot readily switch to other energy sources, and other forms of energy can only be procured at substantially higher prices per BTU. Thus, it is not unreasonable to assume an inelastic demand. Contractual arrangements with large operators seem to confirm this, as contracts are renegotiated once a year to permit coal producers to pass forward any incremental costs occurred since the previous price was established. Geographical limitations lend strength to the assumption of inelastic demand. Shipping costs sometimes equal coal costs.

In consequence, coal buyers are willing to pay higher prices for coal with very little substitution taking place in the short run. Coal suppliers can pass forward to coal users almost all incremental production costs regardless of their nature.

The elasticity of the demand for coal from the small strip mining firm is conditioned by its status within the industry. Since it has been supplying its entire output to electric utilities, the demand for its product is a function of the residual demand of electric utilities not provided by long-term suppliers. Under these conditions, the small strip mining firm has a perfectly elastic demand function: it must accept the price offered by the broker, and could likely do no better by selling on its own. The broker's price is not negotiable. Dominant producers set the price, and small producers must accept the same price as that negotiated by large suppliers.

Brokers enter agreements on a month-to-month basis. With some fixed costs such as debt service, depreciation, and administration, a small firm may be better off taking the broker's offer than discontinuing operations in the short run. 71-7, p. 61-91.

3. The Supply Function

The small strip mining firm's supply function is an average cost function. Monthly production and cost figures are the best source of data for an average cost function -- provided periods are selected in which the firm has operated in the same relative range of overburden, rock content, and coal seam width, and the technology has been reasonably constant. If monthly data can be obtained for major cost items, the balance of costs can be averaged from annual figures and added as a constant. When monthly cost figures are divided by monthly production figures, an average cost per ton of coal is obtained for each month.

For each year included, an equation is established representing a least-squares fit to the monthly average cost data. In each equation, AC is the average cost curve for a given year; X_1 is the monthly coal production in tons; X_2 is the square of X_1; u is the residual. The equations read:

$$AC = a - bX_1 + cX_2 + u.$$

The equations give theoretically appealing downward-sloping cost functions. The introduction of X_2 into the equations produces a curvilinear fit to the data. This is done to see if the average cost curve slope becomes positive at some level of output rather than having a continuous negative slope, i.e., a declining marginal cost. Standard errors and t ratios are computed; some significance level such as 0.05% is applied to each equation.

The output level at which the slope of the average cost becomes positive, and the output level maximizing profit can both be calculated for each year, and compared with the actual average monthly output.

4. Economic Feasibility of Mine Drainage Control and Land Reclamation in Strip Mining

The small strip mining firm has an eleven-year record of increasing revenues and increasing costs, with losses outweighing profits in both the number of years and dollars. Average revenues per ton of coal have ranged from $3.66 in 1965 to $6.19 in 1970, and average costs per ton from $3.86 in 1961 to $5.69 in 1970. In five of the last seven years, the firm incurred losses, with the profit in 1967 amounting to only 1¢ per ton.

Under such circumstances, the control of its water pollution and land disturbance would add expenses the firm can scarcely afford: 7¢ per ton of coal (Method 1), 22¢ (Method 2), 25¢ (Method 3), and 9¢ (Method 4); 5¢ per ton (Technique 1), 6¢ (Technique 2), and 16¢ (Technique 3).

To avoid larger losses, the firm could expand output to a level closer to the minimum cost level or the maximum profit level. However, it is suspected that productivity begins to decline even before these theoretical levels are reached. Should the firm switch to a six-day work week, paying time-and-a-half for all hours worked over 40 per week, equipment maintenance costs may get out of hand. Should it hire additional employees, the firm would be forced into unionization at an extra cost of 40¢ per ton of coal. The acquisition of mineral leases in competition with larger firms that have already leased much of the better mining land would involve considerable expense.

In the year 1970, the firm had its highest expenses, but also by far its highest revenue, resulting in a substantial profit which may have been

sustained and increased in more recent years of energy shortage. Under those conditions, pollution control may have become possible without expansion, and even expansion may have become feasible.

When to the cost of acid water control is added the cost of land reclamation, another responsibility of strip mine operators, the cheapest of the four control methods and three reclamation techniques combined would add 11¢ to the cost of processing a ton of coal. The most expensive combination would add 41¢ per ton. Based on past experience, neither cost could be borne by the small strip mine without increasing losses already sustained. But with steeply rising revenues, both water pollution control and land reclamation may become distinct possibilities.

71-7, p. 108-127.

SUBJECT INDEX

SUBJECT INDEX

Acid Mine Drainage	IX
Allocation, Water	VI; APP.A, B, C, D, E; VII-38/42, 78, 90
Alternative Cost	VII-67
Average-Day Demand	I-2
Benefits	VII-66/74
Blending	VII-25/27
By-Product Recovery	VII-55/56
Collective Treatment	VII-56/59
Consumer Surplus	IV-7, 9; V-16
Contaminants, Incremental	VII-5
Cost	III; V-2; VII-11b/65
Criteria, Raw Water Quality	VII-6/7
Damages Avoided	VII-68
Demand, Water	I-1/4; APP.D; VIII-4
Desalination	VII-42/44
Draft Storage	VI-4
Effluent Charges	VII-64/65, 91/92
Elasticity	V-7/8; VI-5, 9
Equal Likelihood Criterion	I-4
Gross Water Use	V-1
Ground Water	II
Ground Water Recharge	II-1; VI-4; VII-23
Hypertrophication	VII-61
Industrial Water	IV-6; V; VII-52/56
Intake, Water	III-2; V-3; VII-23/25
Integrated Water Use	II-1, 9
Interregional Planning	VI
Irreplaceable Resources	VII-70/71
Lagrange Multiplier	VI-5; APP.B-2
Land Disposal	VII-51,60
Loss	IV-2, 4
Marginal Benefits	APP.B-5; VII-72/74, 85
Marginal Costs	APP.B-5; VII-72/74, 84
Maximum-Day Demand	I-2
Optimization	VI-6, 11; APP.A-6; B-3, 5, 14/22; C-12/14; E-1/3; VII-75/99
Outfall Management	VII-44
Per Capita Water Use	I-1
Population	I-2/3
Power, Electrical	III-4; VII-32
Present Worth	IV-10/11; VII-70
Preventive Pumping	II-3, 6, 9
Pumped Storage	VII-18/20
Pumping Station	III-3; VII-30/32, 37

Quality Classification	VII-1/4
Quality, Water	II-1; VII
Radioactivity	VII-63
Raw Water Storage	VII-32/33
Reaeration	VII-11b
Reservoir	I-7, 9; III-1; VII-14/17
Restrictions, Water Supply	IV-3, 10
Reuse, Waste Water	V-5; VI-6; VII-51; VIII
Risk	IV
River Bed and Lake Bottom	VII-60/61
Sensitivity Analysis	II-11; APP.B-22/23
Shadow Price	VI-2, 7, 12; APP.A-2/3; B-22; E-2
Shortage	I-5, 9; IV
Sizing	I-5/10; VII-14/16, 20/21, 23/24, 28, 34, 56/57
Standards, Treated Water Quality	VII-8/11
Stochastic Flow	IV
Stochastic Quality	VII-69
Storm Water	VII-51
Supply and Demand	VI-6/8, 12; APP.A-4/6; IX-2/4
Surface Water	I
Synthetic Flow	I-6/8
Thermal Pollution	VII-61/63
Transfer, Water	APP.A-4/6; C
Transmission Pipeline	III-2; VII-27/29
Treated Water Storage	III-7/8; VII-33/36
Treatment, Raw Water	III-4/7, 10/12; VII-33/36
Treatment, Waste Water	VII-45/50
Ultimate Capacity	I-9
Uses, Water	VII-4
Well Field	III-8/10; VII-20/22
Willingness to Pay	VII-66/67
With-Without Criterion	VII-66
Yield	I-6, 9; II-3, 7