DEVELOPMENT AND APPLICATION OF MODELS FOR PLANNING OPTIMAL WATER REUSE

by

Larry W. Mays
Principal Investigator

and

Gerardo Ocanas
Mary Schwartz
Research Assistants

CRWR 202

CENTER FOR RESEARCH IN WATER RESOURCES
Bureau of Engineering Research
The University of Texas at Austin
10100 Burnet Road
Austin, Texas 78758

July 1983

THE UNIVERSITY OF TEXAS AT AUSTIN
COLLEGE OF ENGINEERING
BUREAU OF ENGINEERING RESEARCH

Center for Research in Water Resources
10100 Burnet Road, Austin, Texas 78758
(512) 835-3112

TO: Central Duplicating

FROM: Carol Sellers Carol Sellers

SUBJECT: INSTRUCTIONS FOR REPRODUCING CRWR TECHNICAL REPORT NO. 202

DATE: May 1983

Title of Report: DEVELOPMENT AND APPLICATION OF MODELS FOR PLANNING OPTIMAL WATER REUSE

Author(s): Mays, Ocanas, Schwartz

Please reproduce the below number of copies of the report's text, assemble the pages and notify me. I'll then check the reproduced material and ask you to forward the copies to Printing Division for covers and binding.

Do not send the original material to Printing Division; return originals to me. Please have this job ready by ASAP.

The pages should be reproduced in the way circled below:

xxxxxxxxxx FRONT AND BACK SINGLE-SIDE ONLY

No. pages to be reproduced: 192 No. copies to be made of report: 250*.

CHARGE TO ACCT. NO. 26-1404-3050.

Thank you.

*100 copies should have title page 1
150 copies shoud have title page 2

} please deliver to Printing Div. separately as they will have different covers

SIGNATURE OF AUTHORIZATION:

Ernest T. Smerdon

Ernest T. Smerdon
Director

Grant Number: OWRT 14-34-0001-9438 July 1983

FINAL TECHNICAL REPORT

DEVELOPMENT AND APPLICATION OF MODELS FOR PLANNING OPTIMAL WATER REUSE

Submitted to:

U. S. Department of the Interior
Bureau of Reclamation
Washington, D.C. 20240

Authors:

Larry W. Mays, Principal Investigator
Gerardo Ocanas
Mary Schwartz

Bureau of Engineering Research
Center for Research in Water Resources

and

Department of Civil Engineering
University of Texas at Austin
Austin, Texas 78712

OWRT/RU-82/14

ACKNOWLEDGEMENTS

The work upon which this report is based was supported in whole by funds provided by the United States Department of the Interior, Office of Water Research and Technology, as authorized under the Water Research and Development Act of 1978. The Office of Water Research and Technology has since been terminated effective August 25, 1982, by Secretarial Order 3084 and its programs transferred to other bureaus and offices in the Department of the Interior. The Water Reuse Research and Development Program was among those transferred to the Bureau of Reclamation.

ABSTRACT

Water reuse planning models have been developed for determining the optimum allocation of water and reuse of wastewater on a regional basis for single and multi-period planning that minimizes the overall cost of water supply. Both water quantity and water quality parameters of various sources of flows can be considered. Wastewater for all use sectors, along with freshwater, can be considered as candidate sources or origins of water. The models consider the capacity expansion of treatment facilities and allow for economies of scale in treatment and transportation through the use of nonlinear objective functions.

Two basic types of optimization models were developed in this research. The first type are single and multi-period models considered as "macro-level" models which consider regional planning aspects for relatively large planning periods. These mathematical models are large-scale nonlinear programming (LSNLP) models. The second basic type of models are dynamic programming (DP) models for the detailed optimal selection of treatment alternatives (processes). In comparison to the LSNLP modes, these DP models (allocation DP and treatment DP) models consider only a portion of the regional system over a planning horizon with several smaller small time increments.

Development of the LSNLP models was the major portion of this research effort. A single period model was first developed to determine the optimal water allocation with the region, and to assess the impact of various legal and environmental restrictions upon the system. This model was then expanded to incorporate the growing nature of the system. The resulting multi-period model considers the capacity expansion of the treatment facilities while assessing the impact of different planning scenarios. The mathematical optimization models consist of both linear and nonlinear constraints and a nonlinear objective function. The linear constraints include: (1) user's water demands, (2) sources water availability, (3) treatment plants capacity, (4) mass balance equations for users and treatment plants, and for the multi-period model only, (5) capacity expansion. The nonlinear constraints include: (1) the user's quality requirements, (2) the maximum mass discharge of pollutant to sources, and (3) the water quality changes produced by users and treatment plants. One or more pollutants can be considered. The number of water quality constraints increases with the number of pollutants, making the problem larger and more difficult to solve from a computational viewpoint. The objective function can incorporate the operation and maintenance costs for piping, pumping and treatment facilities, the freshwater costs for each source, and the construction costs for piping and treatment facilities. The solution of the LSNLP models requires methods capable of handling large problems with nonlinear objective and constraints.

The large scale generalized reduced gradient (LSGRG) and the successive linear programming with rejection (SLPR) methods incorporate the desired features. Due to the non-convexity of the constraint set and concavity of the objective function which incorporates economies of scale, only local optimality can be guaranteed.

The DP models were developed to consider the planning aspects on a subregional basis to determine optimal allocation and treatment alternatives on a smaller time scale. Their models include: (1) an allocation DP model, which is used for the optimal (minimize costs) allocation of water to secondary users and (2) a treatment DP model, which performs the optimal selection of treatment schemes (processes) over time. The allocation DP model determines how the available water is to be optimally (minimum cost) allocated considering water reuse. In this model the stages are the users, the decision variable is the amount of water allocated to a user from each possible treatment alternative considered, and the state variable is the amount of water left to allocate from the treatment alternative. The allocation DP model is solved for each possible treatment alternative and for each time period in the planning horizon. The allocation DP model essentially is used to define the state space in the treatment DP model. Once the allocation DP model has been solved for each possible treatment alternative and each time period in the planning horizon, the treatment DP model is solved. The stages in treatment DP model are the time periods in the planning horizon, the state variable is the various combinations of treatment alternatives defined by the allocation DP model, and the decisions variable is the choice of treatment alternatives. The overall DP model then provides a minimum cost (present dollars) water allocation and wastewater treatment scheme, considering water reuse, over a planning horizon.

Both the LSNLP models and the DP models were applied to a case study for a region including San Antonio, Texas. For the LSNLP models, the single-period model was applied to planning scenarios for the years 1980, 2000, and 2030. The multi-period model was applied using a planning horizon consisting of three periods, 1980-2000, 2000-2030, and 2030-2060. The DP models were applied to a portion of the San Antonio region using a planning period of 20 years with discrete (annual) time intervals.

This research has resulted in extensive computer software. The computer codes written to solve the LSNLP models are not discussed in any detail in this report. However, an earlier report by Ocanas and Mays (1980) provides a very extensive user's manual for the LSNLP models. The DP models are briefly described in Appendix A of this report.

DISCLAIMER

This Report has been reviewed by the Office of Water Research and Technology of the Bureau of Reclamation, U. S. Department of the Interior, and approved for public dissemination. Approval does not signify that the contents necessarily reflect the views and policies of the Department of the Interior, nor does mention of the trade names or commercial products constitute endorsement or recommendation for use.

FOREWORD

The planning of water resources allocation with water reuse alternatives has received little attention from researchers in the past. The main purpose of this research has been to provide elements for planning and implementing water reuse practices. Although current water technology has shown the existence of efficient treatment alternatives, water reuse has been looked at as a thing for the future, without realizing that the future is now.

An important achievement of this research is that, by considering the water and wastewater systems simultaneously, the best alternative selected by the models incorporates the economic impacts of the constantly increasing hard to find freshwater sources and the benefits of water reuse practices. Once the economic justification of water reuse has been proven, the social and political barriers against it would, hopefully, be less difficult to clear.

This report is the completion report for the Office of Water Research and Technology, U. S. Department of the Interior under Grant No. 14-34-0001-9438 entitled "Development and Application of a Model for Planning Optimal Water Reuse." This work was also partially supported by the Texas Department of Water Resources under Interagency Contracts 14-90029 and 14-00024 entitled "Development of a Model for Planning Optimal Water Reuse." The support of the Office of Water Research and Technology and the Texas Department of Water Resources for this research is gratefully acknowledged.

Other publications which have resulted from work associated with this research project are the following:

1. Ocanas, G. and Mays, L. W., "Models for Water Reuse Planning," Report CRWR-173, Center for Research in Water Resources, University of Texas, Austin, Texas, p. 542, August 1980.

2. Ocanas, G. and Mays, L. W., "A Model for Water Reuse Planning," Water Resources Research, American Geophysical Union, Vol. 17, No. 1, pp. 25-32, February 1981.

3. Ocanas, G. and Mays, L. W., "Water Reuse Planning Models: Extensions and Applications," Water Resources Research, American Geophysical Union, Vol. 17, No. 5, pp. 1311-1327, October 1981.

4. Ocanas, G. and Mays, L. W., "Water Reuse Planning: New Models and Their Application," Proceedings, Water Reuse Symposium II Meeting, Washington, D. C., August 1981.

Meeting, Washington, D. C., August 1981.

5. Schwartz, M. and Mays, L. W., "Optimal Allocation of Water and Treatment Schemes for Water Reuse," Journal of the Environmental Engineering Division, ASCE, to be published in 1983.

Oral presentations have been given at the following meetings that relate to this work:

1. Annual Fall Meeting, American Geophysical Union, San Francisco, California, Dec. 1979.

2. Water Reuse Symposium II Meeting, AWWA, Washington, D. C., August 1981.

3. Annual Fall Meeting, American Geophysical Union, San Francisco, California, December 1981.

This research effort has resulted in one Ph.D. dissertation and one M. S. thesis and support for another Ph.D. student. Completed are:

1. Ocanas, G. "Optimal Water Reuse Planning," Ph.D. dissertation, University of Texas at Austin, Austin, Texas, August 1980.

2. Schwartz, M. "Dynamic Programming Models for Wastewater Treatment Planning and Reuse Allocation," M. S. thesis, University of Texas at Austin, Austin, Texas, December 1981.

Special thanks and appreciation are extended to Professor Leon Lasdon of the University of Texas for the use of his computer code, LSGRG, and the valuable advice given to the authors. Also, special thanks and appreciation are extended to Dr. Quentin W. Martin of the Texas Department of Water Resources for his many valuable suggestions and for the information provided by his staff. Special thanks are due to Amy Phillips and Nickla Tayarani for their patient and painstaking typing of this report.

The excellent research facilities at the University of Texas, in particular the Center for Research in Water Resources, the Department of Civil Engineering, and the Computation Center also made this research effort possible.

CONTENTS

	Page
Acknowledgements	iii
Abstract	v
Disclaimer	vii
Foreword	ix
Contents	xi
Figures	xv
Tables	xvii
Section 1. Introduction	1
1.1 Preliminaries	1
1.2 Need for Water Reuse Planning	3
1.3 Objectives and Overview of Research	4
Objectives of the Research	4
Overview of the Models Developed	6
Large Scale Nonlinear Programming Models	6
Dynamic Programming Models	8
1.4 Review of Literature: Previous Models	9
Regional Wastewater Systems	10
Location and Capacity Expansion Models	12
Water Reuse Systems	15
Section 2. Large Scale Nonlinear Programming Models	22
2.1 Single Period Model	22
System Description - Physical Characteristics	22
Problem Statement	23
Nomenclature	28
Model Constraints	29
Objective Function	33
2.2 Multi-Period Model	35
System Description - Physical Characteristics	35
Problem Statement	36
Nomenclature	37
Model Constraints	38
Objective Function	44
2.3 Solution Techniques	50
Reduced Gradient Methods	50
Reduced Gradient Methods for Large Scale Problems	52
Large Scale Generalized Reduced Gradient (LSGRG) Method	54
Successive Linear Programming (SLP) Method	57
Successive Linear Programming Algorithms	57
SLPR Computer Implementation Features	58

CONTENTS (Continued)

	Page
Finding Initial Solutions	59
The Out of Kilter-Linear Programming (OKA-LP) Procedure	60
Program SETOKA	61
Program SETLP	66
2.4 Software Development	68
Computer Programs for Single Period Model	70
Computer Programs for Multi-Period Model	72
Computer Programs for Finding Initial Solutions	74
Program SETOKA	74
Program SETLP	75
Section 3 Application of the LSNLP Models: The San Antonio Case .	79
3.1 Description of the Application Region	79
3.2 Model Network Structure	83
3.3 Single-Period Model Application	87
San Antonio Region in 1980 - Case 1	87
San Antonio Region in 2000 - Case 2	91
San Antonio Region in 2030 - Case 3	91
3.4 Multi-Period Model Application	96
3.5 Summary, Limitations, and Capabilities of LSNLP Models . . .	100
Section 4 Dynamic Programming Models: Model Structure	104
4.1 General Structure	104
Physical Characteristics of the System	104
Assumptions	109
Conceptual Structure of the Model	112
4.2 Decisions, Data and Problem Statement	116
User Decisions & Data	116
Overall Problem Statement	116
Nomenclature	117
4.3 Allocation Model Description	118
Problem Statement	118
Model Constraints	119
Objective Function	121
4.4 Treatment Model Description	122
Problem Statement	122
Model Constraints	122
Objective Function	123
4.5 Solution Technique	124
Dynamic Programming	124
Algorithms for Allocation Dynamic Program	125
Algorithm for Treatment Dynamic Program	128

CONTENTS (Continued)

	Page
Section 5 Application of the DP Models: The San Antonio Case . .	131
5.1 Introduction	131
5.2 Region Description	131
5.3 System Elements Description	134
Water Users	134
Existing Wastewater Treatment Plants	136
Proposed Wastewater Treatment Plants	137
5.4 Treatment Alternatives Considered	138
5.5 Cost Functions for Case Study	141
Cost Functions	141
Cost Functions for Alternatives	143
5.6 Model Structure and Application	147
Model Structure	147
Process	147
The Case Study	150
Application of Allocation DP	150
Application of Treatment DP	155
5.7 Analysis of Results	159
5.8 Summary, Limitation and Capabilities of DP Models	160
Section 6 Summary and Conclusions	163
6.1 Summary of Models	163
6.2 Integrated Usage of the Models in a Planning Perspective . .	164
6.3 Conclusions	165
Conclusion for the LSNLP Models and Their Applications . .	165
Conclusion for the DP Models and Their Applications	167
6.4 Suggestions for Further Study of the LSNLP Models	168
References	170

FIGURES

Number		Page
1.1	Schematic Representation of Alternate Systems	2
1.2	Rios, Sherman and Malina (1975) Schematic Model	17
1.3	Conjunctive Urban Water Supply and Wastewater System	19
2.1	Schematic Representation of Water Source and Sink Interaction with Other System Components	24
2.2	Schematic Representation of a Water Treatment Plant with Other System Components	25
2.3	Schematic Representation of a User Interaction with Other System Components	26
2.4	Schematic Representation of a Wastewater Treatment Plant Interaction in the System	27
2.5	Operational Costs Present Cost Equivalent	47
2.6	Construction Costs Percent Cost Equivalent	49
2.7	Variable Classification Used in LSGRG Algorithm	53
2.8	An Out of Kilter Algorithm Network	62
2.9	Linear Approximation for the Exponential Cost Function	63
2.10	Modified Network for the OKA Run	65
2.11	OKA-LP Procedure for Finding Initial Solutions	69
2.12	Computer Program Components Interaction for Single Period Planning	71
2.13	Computer Program Components Interaction for Multi-Period Planning	73
3.1	Schematic Representation of the System Elements Components	80
3.2	Feasible Arcs for the San Antonio Region and the Flow Variables Numbering as Used in Cases 1, 2, and 3	84
3.3	Optimal Flow Distribution for the San Antonio Region, Case 1	90
3.4	Optimal Flow Distribution for the San Antonio Region, Case 2	94
3.5	Optimal Flow Distribution for the San Antonio Region, Case 3	95
3.6a	Optimal Flow Distribution for the San Antonio Region, Case 4, Period 1	97
3.6b	Optimal Flow Distribution for the San Antonio Region, Case 4, Period 2	98
3.6c	Optimal Flow Distribution for the San Antonio Region, Case 4, Period 3	99

FIGURES (Continued)

Number		Page
3.7	Capacity Expansion of Treatment Plants (Case 4)	101
4.1	Water Source and Sink Interactions with Other Components in the System	106
4.2	Water Treatment Plant Interaction with Other Components in the System	107
4.3	User Interactions with Other Components in the System	108
4.4	Wastewater Treatment Plant Interaction with Other Components in the System	110
4.5	Overall Interaction Between Model Components	111
4.6	Removal Efficiency versus Water Quantity Information .	113
4.7	Conceptual Algorithm for the Overall Model	114
4.8	State Space for Allocation DP Model	126
4.9	State Space for Treatment DP Model	129
5.1	San Antonio 201 Planning Area	132
5.2	Treatment Alternatives for the San Antonio Region . . .	140
5.3	The Case Study as Network Problems	149
5.4	Allocation Dynamic Program	151
5.5	Treatment Scheme DP	157
6.1	Integrated Implementation of LSNLP and DP Models to Generate Planning Alternatives	166

TABLES

Number		Page
3.1	Cost Function	81
3.2	System Elements (Existing and Proposed)	82
3.3	Node Numbering and Cost Data for Pipeline Construction and Operation and Maintenance (O and M) for Pipelines and Pumping	85
3.4a	System Element Characteristics for 1980: Sources . . .	88
3.4b	System Element Characteristics for 1980: Users	88
3.4c	System Element Characteristics for 1980: Treatment Plants	88
3.5	Optimal Concentrations Leaving Each Node: Cases 1, 2, and 3	89
3.6	User Characteristics for 2000 and 2030	92
3.7	Optimal Concentrations for Case 4	93
3.8	Computational Results	93
5.1	Quantity and Quality Demands	135
5.2	Summary of Capacity and Treatment Level Changes Considered by the Model	139
5.3	Costs Computed for Alternative 2	144
5.4	Capital Costs for Alternative 3	144
5.5	Costs for Alternative 4	146
5.6	Costs for Alternative 5	148
5.7	Secondary User Numbering for the San Antonio Region . .	148
5.8	Treatment Capabiliites for Various Types of Wastewater Treatment Units	153
5.9	Minimum Cost Allocation Schemes	154
5.10	Weighted Returns for Treatment Alternatives: Results from Allocation DP	156
5.11	Actual O and M Costs for Treatment Alternatives: Costs Before Discount	156
5.12	Solution to Treatment DP: Minimum Present Value Costs for 20 Years (Alternative 3)	158

SECTION 1

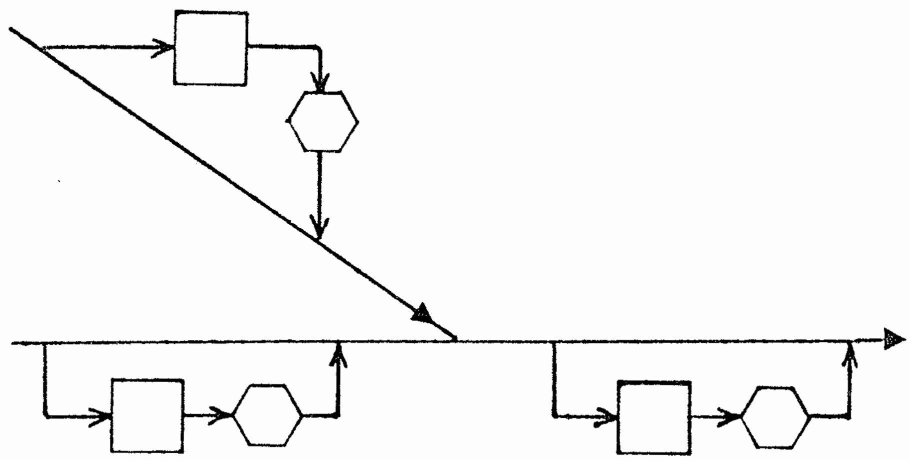
INTRODUCTION

1.1 PRELIMINARIES

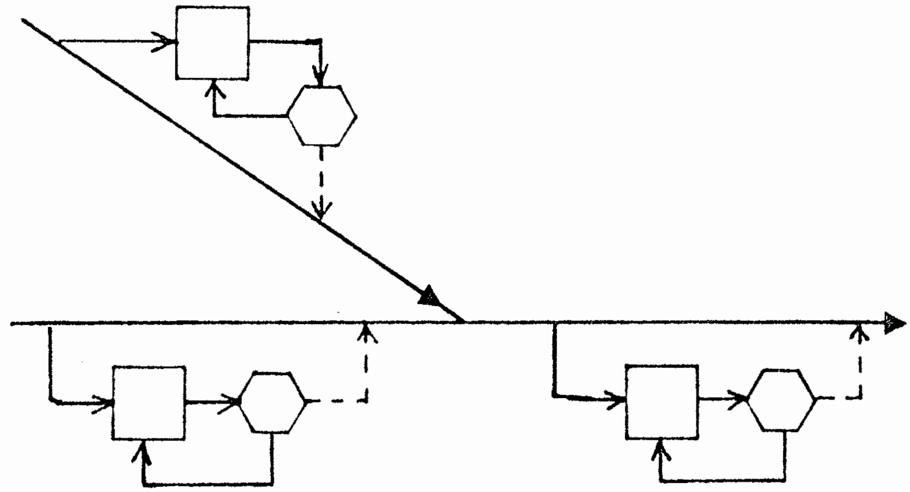
The economic growth of a region is intimately related to reliable sources of water to support the various use sectors and to satisfy the quality and quantity requirements. The concepts of both water conservation and reuse are important to obtain the maximum utility of water resources. While water conservation implies primarily an educational task, water reuse must deal with economic factors as well as health and environmental aspects. Implementing water reuse policies on a private basis has been achieved with satisfactory results. Implementing such policies on a regional basis would require very careful planning in order to optimize the overall efficiency of implementation. The idea of water reuse is becoming an important consideration, especially in states such as Texas, in extending the utility of water supplies. The basic problem is to determine optimal allocations of water from various origins so that quality and quantity requirements of each use sector can be satisfied at minimum cost, considering technological, legal and many other constraints.

In the past, consideration of water quantity had been the only criteria until increasing pollution forced the additional attention to quality considerations. As pollution reaches the sources of water supply, the need for higher degrees of treatment to achieve water quality standards becomes evident. The classical approach to water supply and wastewater disposal has been to consider the problems in the framework of an open system, (see Fig. 1.1a), since the wastewater discharged into a stream, treated or untreated, becomes the supply for demand purposes downstream. However, the limited availability of water sources for a given region justifies the idea of a closed system (see Fig. 1.1b). The wastewater produced by the region can be recycled between their elements (industries, municipalities, etc.) easing the stress placed on the availability of freshwater.

Implementing water reuse policies will present some disadvantages, such as negative public reaction to consuming recycled water, higher degrees of treatment required to convert wastewater into usable water, and in some cases the construction of new distribution networks to transport recycled water separately from freshwater. However, water quality objectives set forth in PL 92-500 include that (1) by 1983 the best available technology economically achievable should be applied to industrial effluents and the best practical technology to municipal discharges; and (2) by 1985 the elimination of discharge of pollutant is a goal to be achieved. These requirements for high quality effluents, coupled with the increasing scarcity of inexpensive freshwater sources, will make water reuse more attractive. Research on water



a) Water Management Open System



b) Water Management Closed System

LEGEND

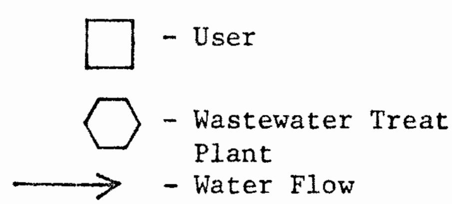


FIGURE 1.1 Schematic Representation of Alternate Systems

reuse has focused mainly in the area of treatment research oriented towards reaching specific reuse application of water quality goals. However, since water reuse will become a necessity for many regions throughout the world, finding the best methodology for its implementation becomes essential. At present, there are not many models for adequately determining and planning implementation of water reuse policies. This research develops two water reuse planning models, each consisting of a nonlinear objective function and linear and nonlinear constraints. One model is for static systems and is usable for single period planning. The other is for dynamic systems, with capacity expansion considerations and is best suited for multi-period planning.

1.2 NEED FOR WATER REUSE PLANNING

The reuse of treated wastewater is also an objective of the 1972 Federal Water Pollution Control Act Amendments (PL 92-500). Wastewater reclamation and reuse has received very serious consideration in many regions in the United States and in other countries.

The Texas Department of Water Resources (1979) estimated that the population of Texas will have increased three times, to a total of 32 million people, by the year 2030. Also, it is predicted that Texas will need 17.5 million acre-feet ($21.6 \times 10^9 m^3$) of water annually for municipal and industrial uses. Annual irrigation requirements will be more than 22 million acre-feet ($27.1 \times 10^9 m^3$). These requirements compare with a use in 1979 of 3.8 million acre-feet ($4.7 \times 10^9 m^3$) for municipal and industrial purposes and 13.5 million acre-feet ($16.6 \times 10^9 m^3$) for irrigation. Annual safe yield of groundwater has been computed to be approximately 5.3 million acre-feet ($6.5 \times 10^9 m^3$); available surface water is about 16 million acre-feet ($19.7 \times 10^9 m^3$). This is a total of 21 million acre-feet ($24.7 \times 10^9 m^3$) compared to a projected need of some 39.6 million acre-feet ($48.8 \times 10^9 m^3$) by 2030. Therefore, the potential for water and energy conservation through water reuse has to be fully exploited.

The Los Angeles County Sanitation Districts have participated actively since 1949 in water reuse planning. As a result, water reclamation plants have been constructed. The Districts serve a population of 4 million people residing in 72 cities in the southeastern sections of Los Angeles County. Garrison and Miele (1977) reported a sewage flow in the Districts of 445 MGD, with approximately 25 MGD of chlorinated secondary effluent being conveyed through flood control channels to percolation basins operated by the Los Angeles County Flood Control Districts. In these basins the reclaimed water is mixed with imported freshwater and percolated into the groundwater aquifer for replenishment purposes. Other secondary treatment effluents are used for agricultural and landscape irrigation, and approximately 0.3 MGD from a tertiary treatment plant effluent is being used for replenishment of three recreational lakes; yet the full extent of water reuse is not achieved.

These two examples show the necessity and importance of water reuse. Current technology makes it possible to return wastewater to a quality level which, with a few exceptions, equals or exceeds that of high quality natural sources. While the cost of treatment is relatively high, there is every expectation that it will decrease as the technical processes are further refined. However, the most significant sources of support for the use of recycled water are the growing problems encountered in the development of new freshwater supplies and the disposal of municipal and industrial wastewater.

One area of water reuse which has not yet received full attention is that of water reuse planning. In March 1979, a water reuse symposium was held which had the theme: "Water Reuse - From Research to Application." This symposium was co-sponsored by the American Water Works Association Research Foundation, the Office of Water Research and Technology, the U.S. Army Medical Research and Development Command, the U.S. Environmental Protection Agency, the National Science Foundation, and the Water Pollution Control Federation. Most of the papers dealt with wastewater treatment technology, specific water reuse applications, health effects associated with water reuse applications, and some cost estimation for specific uses of reclaimed water. All these papers provide an optimistic view of water reuse potentials and present some applications where the feasibility of water reuse is demonstrated. This serves as a background for the next step, water reuse implementation at a regional level. Having demonstrated the existence of the technology and the potential for reuse applications, it is of extreme importance to develop planning tools or mechanisms which include all these aspects of water reuse. Section 1.3 includes a review of some of the planning efforts existing in the literature, which are the base for the planning models developed during this research.

1.3 OBJECTIVES AND OVERVIEW OF RESEARCH

Objectives of the Research

The primary objective of the research was the development of mathematical models which optimize water allocation within a region including water reuse, for both static and dynamic systems. The models are best suited for planning purposes where various planning scenarios can be readily evaluated allowing the planner a broad spectrum of solutions to choose from.

From the literature survey many needs have been determined in order to make the planning, management, and technical implementation of a water reuse system easier and more realistic. Any model or set of models developed to evaluate systematically a water reuse system in a region and aid in the decision making process must:

1. Define the conceptual framework or context in which the results will be reviewed. This framework can be chosen from management concepts, a single or multi-component analysis or an integrative approach.

2. Consider appropriate treatment technologies for the pollutant parameters and reusers in question. Advantages and disadvantages of each technology must be considered as well as different removal efficiencies for each type of treatment.

3. Make clear the tradeoff between the size of the network, the quality of the solution and the applicability to the real problem.

The models developed in this research meet several needs not met by other planning models published in the literature. Quality considerations are not given a large amount of attention in previous systems models reviewed. There is a need to consider not only varying influent qualities (due to seasonal variation, population growth runoff quality variance, etc.) but also to consider the impact of different treatment processes or facilities on the influent and effluent produced.

The majority of the models for water reuse are static models, some do not allow for capacity expansion and some use static demands and maximum availabilities. There are models that do consider time as a factor and other models consider it on a large scale or long term basis. Long range planning models are useful in obtaining an overview of the feasible water reuse policies available to a region, but are not very realistic and applicable when trying to implement a particular water reuse strategy. Water quantity, quality and reliability information on a discrete time interval basis is needed so that the design engineer, the economic advisor (e.g., a bond counselor), the legal advisor (e.g., lawyer), the bureaucrat in the agency responsible for water resources and quality, and the consumer (reuser) have a valid and applicable basis on which to make engineering and financial decisions and therefore can carry out operational and management duties.

For these judgements and decisions to be applicable to treatment and allocation systems and to reuse, both small scale and large scale problems must be considered. Most of the models published are complex, large scale networks. The generalizations made to allow computational efficiency make the large scale problems unrealistic. Solutions to complex small scale problems for many discrete time intervals can provide a more realistic basis on which to plan. This does not mean that many reusers or many treatment processes cannot be considered, but the more components considered the more computer time and data space is needed. Accessibility of the software and a computer to handle the program is therefore an important factor in the planning process.

If the model is not flexible, e.g., cannot incorporate different types of reusers, or does not present results in a systematic and understandable form, it is of no use to the engineers, advisors, public participants or decision makers.

This research consisted of developing two major types of optimization models:

1. Single and multi-period models considered as "macro-level" models considering regional planning aspects for relatively large planning periods. These mathematical models are nonlinear programming (NLP) models and are referred to as Large-Scale Nonlinear Programming (LSNLP) models. These models are solved using two methods:

    a) the Large Scale Generalized Reduced Gradient (LSGRG) Method and

    b) The Successive Linear Programming Rejection (SLPR) Method. These models can consider a large regional system using a large planning horizon discretized into a few large planning periods (e.g., 15 or 20 years).

2. The second type of models are for the detailed optimal selection of treatment alternatives (processes). In comparison to the LSNLP models, this overall DP model considers only a portion of the regional system over a planning horizon with many small time increments (e.g., 1 year time increments). Two models, each based upon dynamic programming, were developed. These models are

   a) an allocation DP model, which is used for the optimal allocation of water to secondary users (reusers) and

   b) a treatment DP model, which performs the optimal selection of treatment schemes (processes) over time. The overall model provides a minimum cost (present dollars) water allocation and wastewater treatment scheme considering water reusing for a discretized planning period.

Overview of Models Developed

Large Scale Nonlinear Programming Models--A single period model was developed during the first stage of the research. This model can be used to determine the optimal reuse alternative within the framework of water allocation in a region with various sources, users and treatment plants during one time period. The size and type of treatment plants are known in advance and are incorporated in the model. The model includes several planning aspects such as:

1. The economies of scale in treatment and transportation cost;

2. The water quality and quantity requirements for each user individually, allowing a user with low quality requirements to satisfy its demand from low quality sources (untreated or partially treated wastewater);

3. Every source of fresh water in the system individually, and

4. The interactions among users and among treatment plants and users.

The optimization model considers all water reuse alternatives by allowing each user to specify its own quality requirements.

During the second stage of the research, the single period model was extended to incorporate capacity expansion and the growing nature of the system, with increasing demands and increasing number of users, as well as the changes in the users quality requirements, the sources, water availability, and other system parameters.

Both the single period and the multi-period models are deterministic, and assume the system parameters as constant during the entire planning period, with the variation of the system parameters occurring only at the beginning of a planning period.

Optimal location and type of treatment for the water and wastewater treatment plants is not achieved by the models. In real situations, the number of possible locations is drastically reduced once the availability of land, costs, legal and social aspects, etc., are considered, therefore the model does not include locations as decision variables in an effort to simplify the problem. The type of treatment is not implicitly considered by the models, but it considers the effect that the pre-determined treatments have on the water. Different planning scenarios may include different treatment alternatives for the treatment plants and hence their impact on the water allocation within the system can be evaluated.

The resulting models consist of a nonlinear objective function subject to linear and nonlinear constraints. A nonlinear programming algorithm is required for the solution. The proposed solution techniques are the large scale generalized reduced gradient (LSGRG) method, developed by Lasdon et al. (1979), and the successive linear programming with rejection (SLPR) algorithm, developed by Palacios and Lasdon (1980). Both methods have proved useful for solving large nonlinear problems rather efficiently. They were developed to deal with large, sparse nonlinearly constrained problems.

A secondary objective of the research was the development of the computer software to aid the models user in the solution of different problem situations. The software allows the evaluation of any regional configuration, where the number of sources, users and water and wastewater treatment plants conform to the regional structure. The software provides the user with the necessary tools to carry out the optimization procedure using the LSGRG or the SLPR algorithms.

Mathematical optimization models which determine the least cost solution to water allocation within a region, including water reuse, are developed. Both models, the single period planning - which considers a static system - and the multi-period planning - which incorporates the dynamic nature of the problem - are best suited for planning purposes where different planning scenarios can be evaluated allowing the decision maker a broad spectrum of solutions to consider. The models consist of a nonlinear objective function, where economies of scale in transportation and treatment cost are included, and both linear and nonlinear constraints. The solution of the planning problems require nonlinear optimization techniques. Two methods are used during the course of the research, one is the large scale generalized reduced gradient (LSGRG) method, and the other is the succesive linear programming rejection (SLPR) algorithm.

The application of the developed mathematical models, the single period planning and the multi-period planning, is illustrated through a series of both hypothetical and realistic problems. The solution of the hypothetical examples provided useful information for the selection of the best solution strategy. During the course of the research, the following computer programs were developed: (1) matrix generators for both the single period and the multi-period models, (2) general subroutines for each model which evaluate the objective function and the nonlinear constraints. These subroutines are required by both optimization codes (LSGRG and SLPR) used during the research, (3) a procedure to determine good initial solutions which have a significant effect on the performance of the optimization codes.

The San Antonio region area is used for the application of both models. Various proposed water and wastewater planning alternatives, such as the development of new surface sources, and improvement and expansion of the wastewater system are included during the modeling process. The results obtained from the San Antonio application provide answers to many planning questions involved in the planning scenarios evaluated.

Dynamic Programming Models--The objectives of the DP models presented in this report are to meet some of these needs through minimizing capital and operation and maintenance costs for any region's wastewater reclamation treatment and distribution system while also considering:

1. Discrete time intervals;
2. Different treatment processes;
3. Different treatment efficiencies as a function of quantity;
4. A range of quantity demands for each reuser; and
5. Transport distances to the reuser.

The optimal feasible reuser strategy determined by the DP models delineate the reuse schemes that minimize the costs involved in a regional reuse system.

Two new models were developed based upon dynamic programming (DP). The models are 1) allocation DP model, which is used for the optimal allocation of water to secondary users; and 2) treatment DP model, which performs the optimal selection of treatment schemes (processes) over time.

The allocation DP model determines how the available water is to be optimally (minimum cost) allocated considering water reuse. In this model the stages are represented as users; the decision variable is the amount of water allocated to a user from each possible treatment alternative considered; and the state variable is the amount of water left to allocate from the treatment alternative. The allocation DP model is solved for each possible treatment alternative and for each time period in the planning horizon. The objective function is to minimize costs. Once the allocation DP model has been solved for each possible treatment alternative and each time period in the planning horizon, the treatment DP model is solved. The allocation DP model essentially is used to define the state space in the treatment DP model.

The stages in the treatment DP model are represented by the time periods in the planning horizon, the state variable is the various combinations of treatment alternatives defined by the allocation DP model and the decision variable is the choice of treatment alternatives. The overall model then provides a minimum cost (present dollars) water allocation and wastewater treatment scheme, considering water reuse, over a planning horizon.

A word statement of the combined or overall DP models would be:

Objective--To find the minimum cost solution which identifies water allocation and water treatment scheme that promotes water reuse in a potentially water short area. Real costs include the costs of wastewater treatment and transportation costs including piping and pumping costs.

Constraints

1. Demand Constraints (minimum and maximum user demands);

2. Capacity constraints;

3. Mass balance constraints;

4. Water availability constraints (freshwater and supplementary sources);

5. Water quality constraints;
   a. User's quality requirements,
   b. Quality changes produced by wastewater treatment plants.

The final results of the overall model include:

1. An allocation scheme to define water reuse over the planning period in question;

2. The treatment alternative(s) and level needed for each time interval;

3. The minimum cost treatment scheme for the planning period in question.

1.4 REVIEW OF LITERATURE: PREVIOUS MODELS

During the past decade, systems analysis has been employed extensively in water quality and water supply management. Most of this work can be classified in two categories, the optimal distribution of the water resources to satisfy the demands within a region, and the optimal treatment of the wastewater produced in a region to satisfy water quality management policies. However, little work has been focused on both problems simultaneously and taking advantage of water reuse practices.

Both static and dynamic systems have been considered, where the latter deals primarily with the capacity expansion of the system. Water supply systems were first treated as a single period problem rather than as a multiperiod or dynamic system. Capacity expansion has been approached by a large variety of modelling techniques, including dynamic programming, mixed interger programming, network flow programming and nonlinear programming. Each technique, requiring different assumptions and input information, will approach the problem differently and so will be their solutions procedure. The proper selection of either technique relies on the actual system and the expectation required from the model. Therefore, it is the responsibility of the modeler to consider all the techniques available along with the system characteristics before making the final decision as to how the problem should be approached.

Regional Wastewater Systems

Economies of scale were among the main factors influencing researchers to consider regional wastewater treatment systems, where the locations of sources and their waste flows were fixed in advance as were the regional treatment plant locations and the allowable pipeline routes. These economies of scale imply concavity when the functions are continuous. Deininger and Su (1971) used the fact that since the cost functions are concave the optimum solution must be at an extreme point of the constraint set (which was linear). Using a piecewise linear approximation of the cost functions and an algorithm that ranks the extreme points, a hypothetical case was solved with seven sources located on a single branch network configuration.

Bhalla and Rikkers (1971) presented a heuristic technique for solving the regional plant location problem as part of an effort to plan the capacity expansion of regional systems. At each stage in the algorithm, the facility which could serve all unassigned sources most cheaply was identified. Then for each location still available, the subset of the unassigned sources which presented the most savings when served by a facility of this location (and corresponding assigned sources) was found. The location with the greatest savings was added to the solution and the process was repeated. Additional rules were given for dropping facilities from the solution and sending their sources elsewhere.

Graves et al. (1970) developed a nonlinear model which involved a potential network of piping between waste sources, regional treatment plants and river reaches. The optimal waste flows through this network plus the level of BOD removal at each treatment plant was determined so that a dissolved oxygen standard was met. Split flows and bypass piping of waste discharges to other reaches were allowed. Treatment plant costs were nonlinear functions of the size of the plant and percent of BOD removed, while piping costs were functions of flow. The BOD-Dissolved Oxygen model developed by Thomann (1972) was used to construct the quality constraints which involved nonlinearities. The nonlinear programming model was solved using a linearization algorithm based on the feasible directions approach which requires the cost function to be continuous. From a given solution, a direction search is found by solving a linear program derived from a first order Taylor series expansion. The step size for this direction is then chosen from quadratic approximations so as to give the greatest improvement while maintaining feasibility. Notable features of this algorithm include parametric adjustments of the error term in Taylor series expansion to maintain consistency in the linear program and insure gain in the optimization, and the use of priority classes of variables (a form of restriction strategy) to reduce computational effort. For nonconvex problems only local optimality is guaranteed.

Converse (1972) considered a simplified version of the regionalization of wastewater treatment systems. The model assumed that all wastewater sources are along the main stem of a river and that there was a finite number of locations at which wastewater treatment plants could be located. The optimal solution would determine whether to install a treatment plant at a particular location or to install a trunk sewer to allow treatment at another location. The model did not consider the constraint that river quality standards were met. The objective function considered the trade-off between the number of treatment plants and the extent of trunk sewers. Hence, economies of scale which favored installation of a few large treatment plants were offset by the greater distance that wastewater had to be piped. A dynamic programming procedure was used in which the stages were the number of plants built, the state was the number of source locations away from the last source and the decision was whether to install a treatment plant at a particular location or a trunk sewer to the next location.

Rossman and Liebman (1974) solved the problem of how a group of waste dischargers along a river should plan and construct a regionalized system of treatment facilities so that a water quality standard would be met at minimum cost. They used the carbonaceous biochemical oxygen demand (CBOD) of wastewater and its effect on stream dissolved oxygen (DO) as quality measures. The model consisted of two sets of decision variables, one concerning the facility location decision and another concerning the degree of treatment. The resulting nonlinear model was solved by a method based on a particular dual approach to nonlinear programming which makes use of Generalized Lagrange Multipliers, GLM, also known as Everett's Method. The basic idea is to incorporate the "complicating" constraints in the objective function and then solve a series of less constrained and hopefully easier problems until a certain optimization criterion is met.

Lauria (1975) developed a mixed integer programming model for determining the location, timing and scale of regional wastewater treatment plants, sewers and pumping stations. The model takes account of existing wastewater facilities and includes operational and construction costs. Concave cost equations were approximated by fixed charge functions. The solution of the mixed integer programming model required a branch and bound technique. The algorithm used was developed by Dakin (1965) and is as follows: (i) treating the binary (0, 1) variables as continuous, solve the problem by linear programming (LP); (ii) if any binary variables are not integral, select one on which to branch and form two new LP problems retaining all other constraints, one with the binary variable set equal to zero and the other with it equal to one; (iii) examine the solutions (terminal nodes) and find the one with the best objective function value; (iv) if all binary variables for this solution are integral, the problem is solved; otherwise return to step (ii). Along the process, each LP problem provides upper bounds to the original problem, and the smallest upper bound currently available is used to eliminate branches, avoiding the enumeration of all possible LP problems.

Location and Capacity Expansion Models

Since planning over a time horizon is an important aspect in water quality and water resources management, where the optimal strategies for capital investments for the construction of waste treatment facilities, distribution and sewage networks, and other projects associated with water management are essential although not trivial; the problem of capacity expansion has received considerable attention of researchers in the last ten to fifteen years.

Capacity expansion problems are usually associated with discrete facility location problems, where projects are to be located in time. Sa (1969) described a capacitated plant location problem where a finite number of limited capacity plants had to be located to satisfy the demand of a finite number of demanders. A branch and bound technique was used to solve the problem.

Curry and Skerth (1969) considered the problem of facility location-allocation where the objective is to allocate a given number of facilities in a finite number of possible locations and assign to each facility a number of demanders to be satisfied. The application of the model may range from machinery location in a manufacturing plant to satisfy the demands for the use of the equipment, to treatment plant location in a region to satisfy the demands for wastewater treatment. The model was formulated as a mathematical programming problem and decomposed into the recursive equations of dynamic programming where the objective function was to minimize the total distance travelled. This problem differed from the problem described by Sa in that the demand that each facility had to satisfy were not fixed.

Wesolowsky (1973) extended the static single location problem to a model that permitted location changes within a planning horizon of r periods. This dynamic facility location problem was solved by an algorithm called "incomplete dynamic programming" where the sequence of locations of destinations was optimized.

Leondes and Nandi (1975) suggested a simple algorithm for solving capacity expansion problems defined by a network with uncertain demands. The cost of expanding the capacity of each arc (an arc may represent the flow of passengers in a transportation network between two nodes, or the flow of information in a communication network) was assumed to be a convex function and a concave salvage value was associated with excess capacity. The problem was formulated as a nonlinear stochastic programming problem, and the method was based on the concept that the capacity of each arc was expanded by a given amount (to satisfy an assumed first stage demand) in the first stage when the demand was unknown; and then when the demand became known deterministically, a corrective action was taken by solving a second stage problem either to increase or reduce the capacity.

Butcher, Haimes, and Hall (1969) presented one of the first papers on sequencing water resources projects to meet a given demand schedule. A dynamic programming approach was used to minimize the present value of total cost. The solution procedure required that the projects being sequenced have their capacity and cost selected in advance. The model was deterministic, with discrete time, discrete size for the expansion projects, and a single demand center. The objective function consisted of the discounted cost of the construction of the projects. The construction cost was considered as an equivalent single payment at the time when the project is available. No operational costs were included.

Becker and Yeh (1974) presented a dynamic programming approach for the optimal timing, sequencing and sizing of multiple reservoir surface water supply facilities. In this approach, firm water is demanded, however, the reservoir capacity at a site is costed. It was recognized that the firm water increment added by construction of a reservoir depends strongly on several factors. These include the existing firm water level, the hydrologic behavior of the streams in the system, and the site and size of existing reservoirs, as well as on the capacity of the added reservoir.

Martin (1975) developed a model for determining the minimum cost capacity expansion of a general surface water supply system. The algorithm is comprised of a dynamic programming project scheduling model which selects likely optimal expansion policies, and a generalized network model which analyzes in detail the operation of the selected development policies. These models interact in an iterative manner to select the set of reservoirs, pumps, canals and pipelines, the capacity or sizing, and the construction sequencing which minimizes the present value of capital and operating costs of the entire system over a fixed time horizon under a deterministic set of hydrology and demands.

Scarato (1969) presented a minimum cost method to time and size water system component expansion to meet a determined population growth rate. The method analyzes the economic impact both of the economies of scale in construction and of the real cost of capital. Water pipelines and general water treatment facilities were selected for illustration which demonstrate economies of scale. Initial construction costs favor building single large units to take full advantage of the effect of economies of scale on costs, however, building several smaller units at different points in time allows a savings in interest on capital investment until the capacity is needed. Therefore, these two opposing factors, economies of scale and the time value of capital, were considered simultaneously. The result is a capacity expansion model which determines when and how much excess capacity shoud be installed to meet increasing demands. The model assumed a linearly increasing demand $q = Dt$, where q is the demand, D is a constant, and t is time. The capacity cost was assumed concave, $c(x) = kx^a$, where $c(x)$ is the cost for capacity x, k is a constant and a is an ecoNcomy of scale factor. The model assumed a continuous discount factor, an infinite penalty cost (demand is always satisfied) and an infinite time period. The method is based on classical optimization for unconstrained problems, where the equation for total discounted cost is expressed as a function of time only. Differentiating this cost equation and solving for minimum cost yields t^*, the optimal time phasing. This time phasing represents the time period for which the optimal capacity increase, Dt^*, will at least meet the demand. Therefore, the solution suggests a uniform capacity increase over time every t^* years. The economic effects of timing and sizing decisions on total costs were plotted for various interest rates and levels of economies of scale.

Tsuo, Mitten and Russell, (1973) presented a search technique for project sequencing, i.e., finding the sequence for construction of a set of projects that will meet a growing demand at minimum discounted cost. The procedure they presented is best suitable for preliminary planning and feasibility studies for problems that can be reduced to that of project sequencing. A heuristic rule was suggested using a search procedure which involves computing an index which is indicative of the discounted cost of a partial sequence. This partial sequence starts by choosing the first project to be constructed, i.e., with the lowest index, and at every iteration a new project is added to the sequence; namely, the project which, along with the sequence at that iteration, provides the lowest discounted cost. This technique can be applied in the preliminary screening phase of a large complex study. The technique finds an optimal (or near optimal) sequence of projects to meet a projected demand at mimimum discounted cost, and then provides a sensitivity test of the results, making it appealing for use with approximate cost instead of the usually hard to obtain accurate costs.

Hinomoto (1972) investigated the multi-stage capacity expansion of a municipal water treatment system to determine the sizes of new treatment plants and times at which these new plants are added to the system. The capital and operating costs of these plants are given by concave functions reflecting economies of scale available with an increase in capacity. To determine the optimum sizes and installation times of the new plants, the expansion problem was formulated as a dynamic programming model. The treatment plants of the system were assumed to be interlocked and to function as an integral unit. The investment and operating costs were significantly affected by the source of water, surface or underground.

Haimes and Nainis (1974) proposed a water resource planning framework in which the scheduling, construction and expansion of water resource projects, such as water supply projects, were chosen with economically based objectives in mind. A long-range capacity expansion planning model was formulated that provides a projection of future water supply availabilities based on economic efficiency. The planning model provided a least-cost schedule of development projects that also maximized the net benefits of the outputs of those projects to a regional economy. A dynamic programming algorithm was used to schedule the projects, and a Leoutief input-output linear program was used to model the response to project supplies. Multi-level theory was used to provide a dynamic coordination scheme that sought the best joint solution of the scheduling problem and the input-output linear program. A regional planning example was presented and the results were discussed.

Lesso et al. (1977) developed a model for determining the selection and scheduling of waste treatment plants in a river basin. The solution procedure consisted of an enumeration procedure (branch and bound) where sets of local plants were to be located in time along the river and their feasibility was determined by a simulation model where dissolved oxygen was used as the quality control parameter. The model was applied to a large river basin in Brazil.

Water Reuse Systems

Implementation of water reuse policies has received little attention from a management viewpoint. Research on water reuse has been focused almost entirely on the development of treatment processes in order to achieve a "good quality" water. However, since water reuse will become a necessity for many regions throughout the world, finding the best methodology for its implementation becomes essential.

Bishop and Hendricks (1971) presented one of the first applications of system analysis to the problem of water reuse. The problem was considered a transshipment problem and used linear programming for the solution procedure. This rather simple model did take into account the cost of treatment plus the cost of transportation, however, these costs were assumed linear. The question of when and what size to build any new projects was not considered.

Bishop, Hendricks and Milligan (1971) presented a methodology to assess water supply alternatives, such as water reallocation, reuse and importation, by examining the amount and staging of water sources development. A model for the water system was formulated as a transportation problem in linear programming depicting the possible sources of supply which can be used to satisfy the requirements of various water users. The objective of the model was to minimize the cost of water under various assumptions for operating the system. This model was a simplified approach to the problem of water allocation at minimum cost, however, the use of linear functions to represent the cost of water is a major simplification which made the problem less realistic.

Bishop and Narayanan (1977) presented a model which included seasonal and stochastic factors in water planning. The problem of optimal allocation of water resources in a region was considered, which shows a paralles resemblance to the transportation and transshipment problem of linar programming. economies of scale in the cost functions were considered using a piece-wise linear approximation to the concave nonlinar function. The randomness associated with the right hand side values of some of the constraints was incorporated. The procedure involved was as follows: Consider the water availability constraint for source k, $\sum_k x_{ki} \leq b_k$, where x_{ki} is the flow sent from source k to node i, and b_k is the source water availability. It is desired to hold this constraint with at least a probability of β_k when b_k is assumed to be a normal random variate. The constraint can now be written as $\Pr[\sum_k x_{ki} \leq b_k] \geq \beta_k$. Subtracting $E(b_k)$, in which E denotes the expectation operator, from both sides of the inequality within the square brackets and dividing by the standard deviation of b_k, σ_{b_k}, the constraint becomes

$$\Pr \left[\frac{\sum_k x_{ki} - E(b_k)}{\sigma_{b_k}} \leq \frac{b_k - E(b_k)}{\sigma_{b_k}} \right] \geq \beta_k \tag{1.1}$$

The quantity $Z = b_k - E(b_k)/\sigma_{b_k}$ is a standard normal variate. Let the cumulative distribution function of Z be $\theta(z)$. Then $\theta^{-1}(\beta_k) = P_{\beta_k}$ can be obtained from a table of distribution of standard, normal, random variables. The original constraint is therefore converted to the equivalent linear constraint incorporating randomness,

$$\sum_k x_{ki} \leq E(b_k) + P_{\beta_k} \sigma_{b_k} \tag{1.2}$$

This procedure was applied to all sources availability constraints. The resulting model was applied to a study region in Utah.

Rios, Sherman and Malina (1975) approached the problem of water supply to a region where the availability of fresh water was limited. An attempt was made to minimize the demand of fresh water by enhancing water recycling and reuse practices. The model consisted of one super source, which could supply fresh water to every user in the region; one super sink, where wastewater would eventually be discharged; and the users which could recycle their water or send the water to other users, treating first to comply with any quality requirements. Figure 1.2 shows a schematic representation of the model. The objective function consisted of a nonlinear part which included transmission costs, water treatment costs and wastewater treatment costs, and a linear part consisting of the fresh water cost and the disposal cost. The constraints were all linear and included the water quantity requirements for every user, water availability constraints for the one super-source and the mass balance constraints for every user. The solution algorithm used linearizes the objective function and a linear programming problem is executed to find an optimal solution. Thus, a procedure for ranking the extreme points of the linear problem is used to generate upper and lower bounds on the nonlinear problem until both bounds converge at the optimum. The disadvantage of this procedure is that there is no way to predict beforehand how many points will have to be ranked before the optimal solution is obtained, making the procedure rather inefficient.

The mathematical model does allow for interactions among users, considering the possibility of direct reuse of treated effluents; however, the model does not allow for regional treatment plants, where different wastewater effluents culd be treated and sent back to the users. Hence, the model assumes that an effluent would be treated as to meet the quality requirement of the receiver. This allows the possibility that a fraction of a single effluent receives a certain treatment while another fraction receives a different treatment; however, no construction costs were included in order to incorporate this situation. Other disadvantages of the model are that only one source is considered and that once an effluent is discharged out of the system it is no longer available.

A major simplification of this model is the assumption that the effluent concentration of a user is constant, regardless of the quality of the water entering the user, therefore no quality constraints are incorporated. The model is approached as a static system where only a single period can be

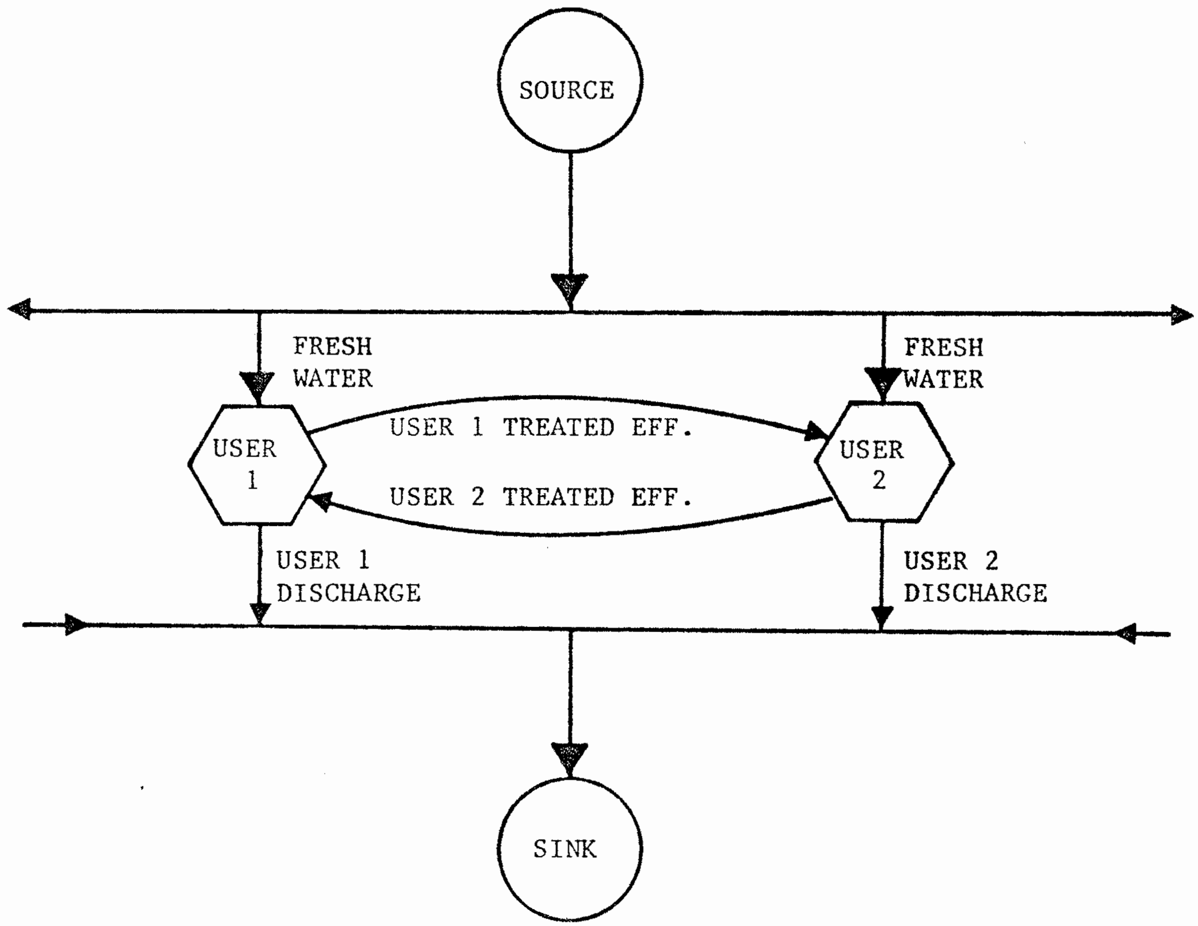


Figure 1.2 Rios, Sherman and Malina (1975) Schematic Model

evaluated at a time, therefore no capacity expansion of the elements in the system is considered, neither time related variations of certain parameters, such as increasing demands, water availability, etc. are considered.

Mulvihill and Dracup (1974) formulated a mathematical model of a conjunctively operated urban water supply and wastewater system. The objectives of their model were: (1) to minimize the cost of supplying water from several sources, including the provision for recycling reclaimed water and (2) to determine the capacity expansion schedule of the water and wastewater treatment processes. Figure 1.3 shows a conjunctive water supply and wastewater system. The problem consisted of (1) several sources available for water supply that have varying quantity, quality and cost, (2) a series of water treatment processes that remove or reduce various quality constituents, (3) stipulated water quality standards and quantity requirements, (4) a series of wastewater treatment processes that remove or reduce various quality constituents, (5) wastewater effluent standards, and (6) provision for recyclying reclaimed water. These factors would determine (1) the optimal mix of servral water sources, (2) the capacity expansion (timing and sizing) of the water and wastewater treatment unit processes and (3) the quantity of reclaimed water to be recycled.

The mathematical formulation consisted of a nonlinear objective function subject to linear constraints. The objective function being the present worth summation of water supply costs and wastewater treatment expansion costs. The constraint set consisted of: (1) the water requirements constraints, (2) the water availability constraints, (3) the water user quality constraints, (4) the effluent standard constraints, (5) the water treatment plant constraints, (6) the wastewater treatment plant constraints, and (7) the continuity constraints.

The proposed solution technique was a multi-level solution procedure, consisting of two levels. The first level of the algorithm is an iterative process in which the objective function is successively linearized and a series of linear programs is solved. When this level yields no improvement, the second level, a search of neighboring extreme points, is initiated. If there is an improvement at this level, the procedure returns to the first level and continues.

Two major disadvantages of this model are: (1) the lack of interactions among users, since the model assumes all users as only one big user with a unique water quality requirement for all users and with one demand equal to the total of all users, and (2) the lack of interaction between users and wastewater treatment plants, disallowing for any possible direct water reuse of the treatment effluents.

Pingry and Shaftel (1979) presented a nonlinear model which takes into account both flow requirements and water quality considerations. This model considered water sources, water treatment plants, water users, and water disposal sites. Water was allowed to flow from sources to treatment plants and users, from treatment plants to users and disposal sites, and from users to treatment plants, disposal sites and other users. Each flow had an associated quality as defined by measured concentrations of various pollutants.

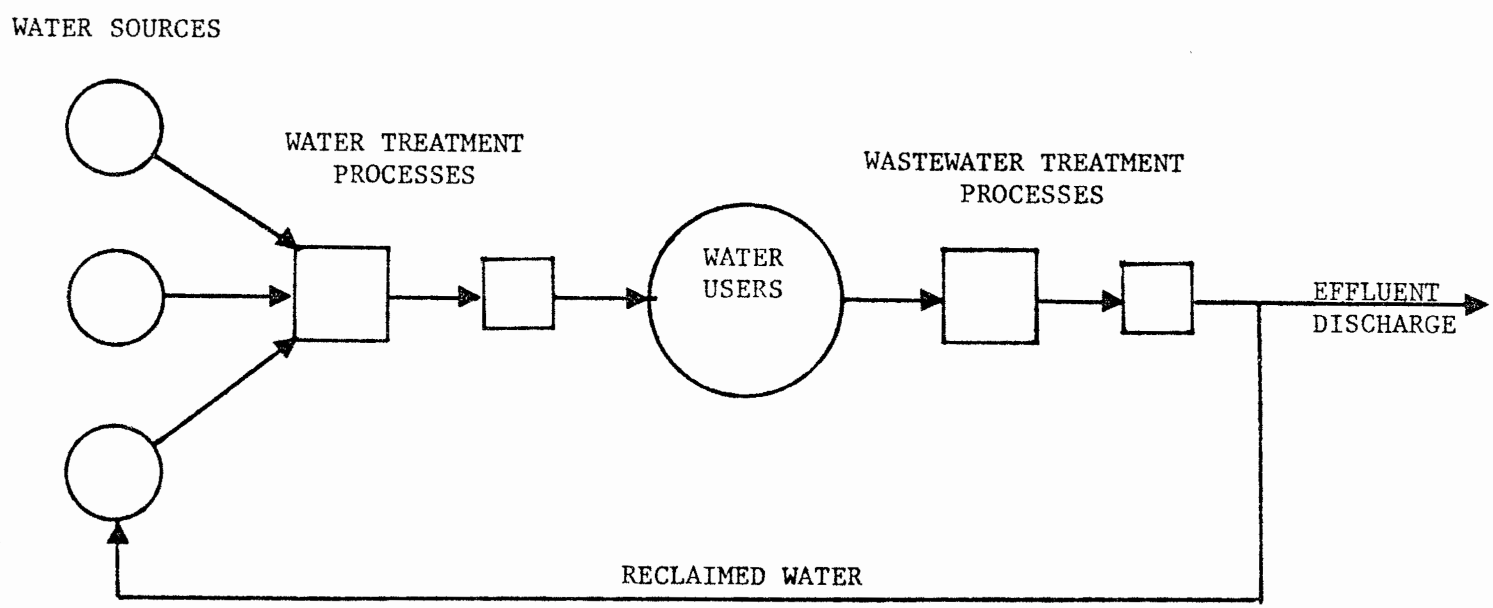


FIGURE 1.3 Conjunctive Urban Water Supply and Wastewater System

Each treatment plant, user and disposal site was allowed to have minimum acceptable quality standards for influent flow. Treatment plants could affect the water quality by removing some percentage of the pollutant while users could change the water quality by adding a given mass of pollutant.

The mathematical model consisted of a nonlinear objective function subject to linear and nonlinear constraints. The objective function included: (1) piping costs which were nonlinear with respect to flow to show economies of scale; (2) treatment costs which were concave functions with respect to flow but convex functions with respect to degree of treatment; (3) source costs which were linear with respect to flow; and (4) disposal costs which were linear with respect to the weight of material being disposed.

The constraints included: (1) sources water availability, (2) mass balance for treatment plants, (3) treatment plants capacity, (4) users demands, (5) wastewater availability, (6) sink capacity, (7) pollutant mass balance for treatment plants, (8) pollutant mass balance for users, (9) quality requirements for all water elements. Constraints (1) through (6) were linear constraints while the remaining were nonlinear. The model also includes upper bounds on the concentrations. The model did not consider recycling of water to the sources and the problem was approached as a single period system with no capacity expansion or time variation of certain system parameters. The solution technique consisted of an iterative method where a transshipment problem with a nonlinear objective function was solved for a given set of quality parameters at every iteration; these quality parameters consisted of the effluent concentrations of users and treatment plants and were predetermined for every iteration, and included the quality constraints as linear equations. The algorithm stops when a search over the quality parameters converges to the minimum. This search is based on the work done on parametric transportation problems. The model was applied to a small hypothetical example to show its application.

SECTION 2

LARGE-SCALE NONLINEAR PROGRAMMING MODELS

2.1 SINGLE PERIOD MODEL

Interest in water recycling and reuse has been stimulated recently by the recognition that in a growing number of regions large quantities of high quality water may no longer be available at low cost. The possible economic feasibility of reuse and recycling dramatically increases the complexity of designing an optimal water allocation system. The water system designer has to choose from a number of sources of water, treatment plants, users, and disposal sites, the allocation which produces the least cost delivery strategy while satisfying quality, environmental, and legal constraints.

Previous water system mathematical models have been mostly applications of linear programming such as transportation and transshipment models, with linear cost functions which do not represent the real problem. Some models have incorporated economies of scale in the objective function, but the quality constraints have been somewhat simplified and do not allow the possibility of various flows with different concentrations to satisfy given water quantity and quality requirements.

The models described in this chapter allow for economies of scale in transportation and treatment costs. The quality of the flows is explicitly considered by taking into account the effect that each element in the system has on the quality of the water. The models also allow for interactions among water and wastewater treatment plants, water sources, and water users. The water quality and quantity requirements for each user are considered individually, allowing users with low quality requirements to satisfy their demand from low quality sources (untreated or partially treated wastewater). Every surface and groundwater source in the region is also considered individually. The first model described is suitable for single period planning and in Section 2.2 is extended to a multi-period model to incorporate capacity expansion and true variations of the system parameters.

System Description - Physical Characteristics

A region consists of many water related elements; water sources, water treatment plants, water users, wastewater treatment plants, and water sinks. Each element has its own characteristics, and interacts with other elements in the system as follows:

1. Water Sources and Sinks. Surface and groundwater sources, as well as sinks, are in this category. Each source is defined by (i) location in the region, (ii) water availability, which is expressed as the maximum water yield, (iii) pollutant assimilation, as defined by the maximum acceptable mass discharge of pollutant, and (iv) quality profile, which is defined in terms of the concentrations of each pollutant being used to enforce the water quality constraints during the planning process. Water sources can send water to water treatment plants, where the quality is improved, and to users without previous treatment. Water sources and sinks can receive wastewater treatment plant effluents and user discharges. See Figure 2.1.

2. Water Treatment Plants. Each plant is defined by: (i) location in the region, (ii) maximum flow capacity, (iii) water losses and (iv) treatment performance, as given by the removal efficiencies that the plant achieves for each pollutant under consideration. The model considers each plant as having a predetermined and constant removal efficiency which is used in evaluating the impact the plant has on the quality of the water as it passes through. Water treatment plant effluents are sent only to users in the region and only receive water from surface and groundwater sources. See Figure 2.2.

3. Water Users. A user can be defined as any element in the system which exerts a demand of water. Industries, municipalities, agricultural sites, and energy complexes are major users. A user is defined by: (i) location, (ii) water demand, (iii) water losses or consumption, (iv) water quality degradation, as given by the concentration increment caused during the use of the water, (v) treatment facilities which are defined in terms of the removal percentage achieved for each pollutant under consideration, and (vi) water quality requirements, which represent the maximum acceptable concentration for each pollutant. The model allows a user to recycle water or to send the water directly to another user in the region. A user can also discharge its wastewater to a source or sink and can send the effluents to a wastewater treatment plant. A user can receive water from a surface or groundwater source, a water treatment plant, a wastewater treatment plant or another user. See Figure 2.3.

4. Wastewater Treatment Plants. Each plant is defined by: (i) location in the region, (ii) maximum flow capacity, (iii) water losses and (iv) treatment performance, as given by the removal efficiencies achieved for each pollutant under consideration. The model considers each plant with its predetermined and constant removal efficiency which is used in evaluating the impact of the plant on the quality of the water. Wastewater treatment plants receive water from users in the region and send their effluents to water sources (sinks) and users. See Figure 2.4

Problem Statement

A word statement of the problem is as follows:

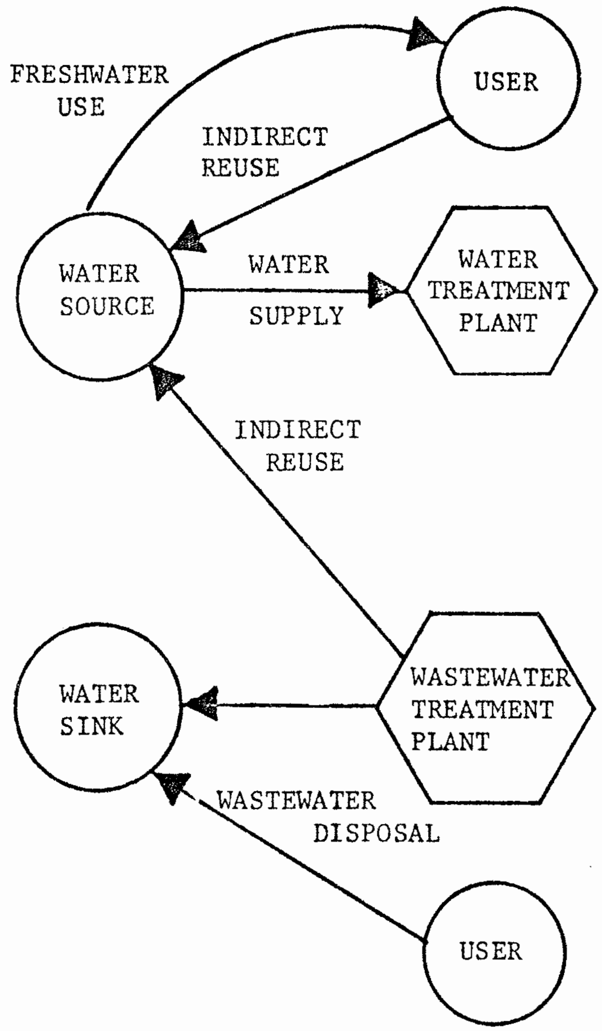


FIGURE 2.1 Schematic Representation of Water Source and Sink Interactions With Other System Components

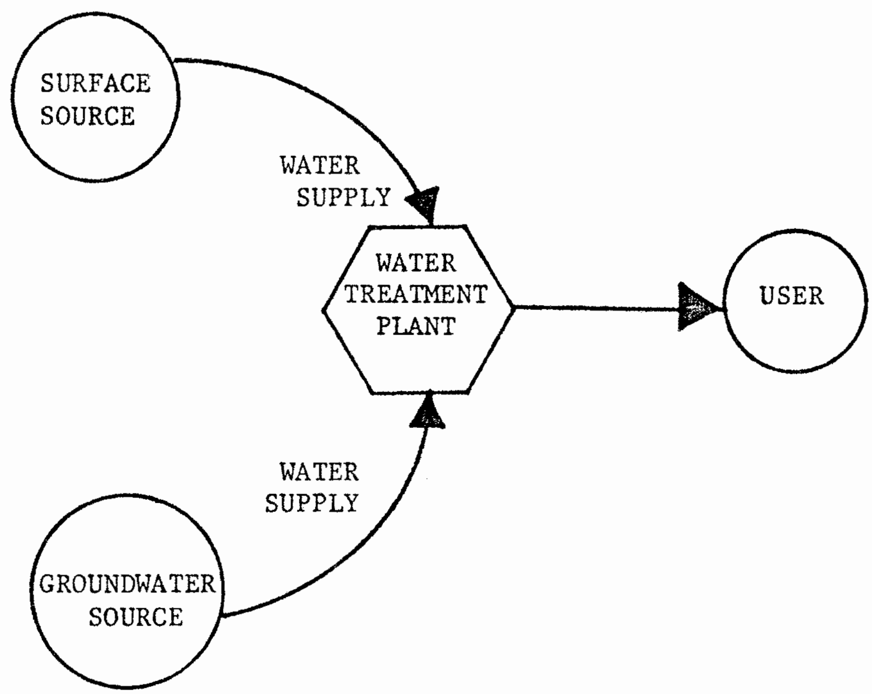


FIGURE 2.2 Schematic Representation of a Water Treatment Plant Interaction With Other System Components

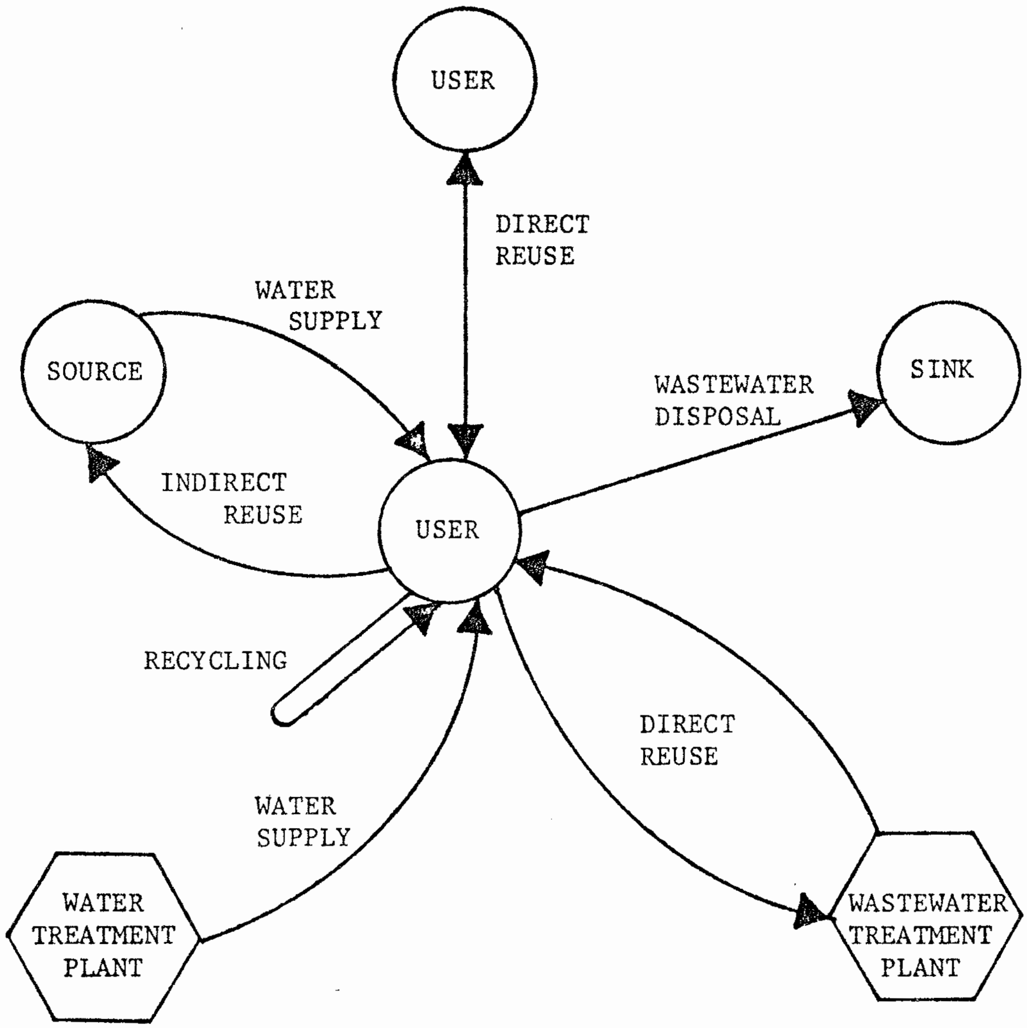


FIGURE 2.3 Schematic Representation of a User Interaction With Other System Components

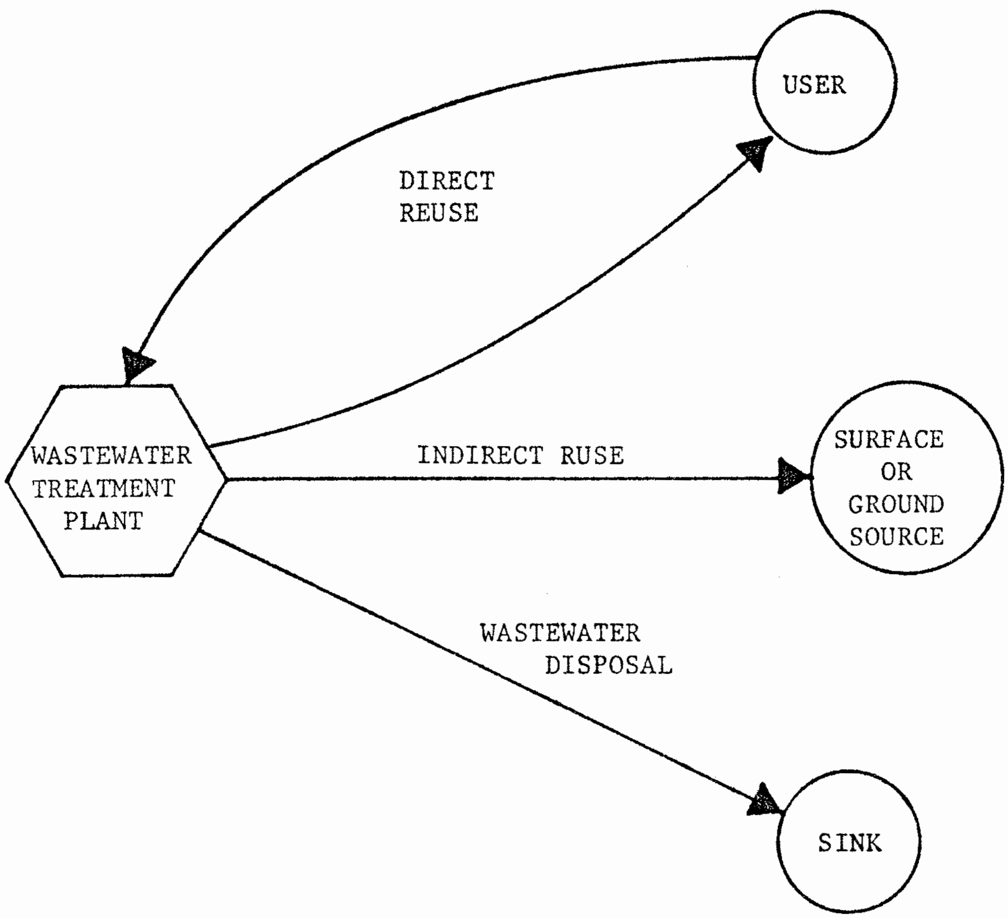


FIGURE 2.4 Schematic Representation of a Wastewater Treatment Plant Interaction In The System

Objective--To find the minimum cost solution to the problem of supplying water from different types and locations of sources to every user in the region, allowing for water recycling (returning to same user) and reuse. The cost includes the cost of water and wastewater treatment and the transportation cost, including piping and pumping.

Constraints--

1). Demand constraints

2). Capacity constraints

3). Water mass balance constraints

4). Water availability constraints

5). Water quality constraints

   a). User's quality requirements
   b). Maximum discharge of pollutant to sources
   c). Quality changes produced by users
   d). Quality changes produced by water treatment plants
   e). Quality changes produced by wastewater treatment plants

Nomenclature

Decision Variables

XS_{ij} = Amount of surface water sent from source i to user j without treatment

XG_{kj} = Amount of groundwater sent from source k to user j without treatment.

$XT_{\ell j}$ = Amount of treated water sent from water or wastewater treatment plant ℓ to user j.

XSR_{ji} = Amount of wastewater sent from user j to surface source i.

XGR_{jk} = Amount of wastewater sent from user j to ground source k.

$XTR_{j\ell}$ = Amount of wastewater sent from user j to wastewater treatment plant ℓ.

$XST_{i\ell}$ = Amount of freshwater sent from surface source i to water treatment plant ℓ.

$XGT_{k\ell}$ = Amount of freshwater sent from ground source k to water treatment plant ℓ.

$XSTR_{\ell i}$ = Amount of wastewater sent from wastewater treatment plant ℓ to surface source i.

$XGTR_{\ell k}$ = Amount of wastewater sent from wastewater treatment plant ℓ to ground source k.

X_{tj} = Amount of water sent from user t to user j.

$CT(P_n)_\ell$ = Pollutant n concentration leaving water or wastewater treatment plant ℓ.

$C(P_n)_t$ = Pollutant n concentration leaving user t.

The units for flows are in million gallons per day (MGD) and the units for pollutant concentrations are mg/liter.

Required Input

NU	=	Number of users.
NT 1	=	Number of water treatment plants.
NT 2	=	Number of wastewater treatment plants.
NS	=	Number of surface sources.
NG	=	Number of ground sources.
DEM_j	=	Water required by user j.
L_j	=	Water losses by user j.
LT_ℓ	=	Water losses at water or wastewater treatment plant ℓ.
SWA_i	=	Maximum withdrawal permitted at surface source i.
GWA_k	=	Maximum withdrawal permitted at surface source k.
$CS(P_n)_i$	=	Pollutant n concentration of surface source i.
$CG(P_n)_k$	=	Pollutant n concentration of groundwater source k.
$STD(P_n)_j$	=	Pollutant n standard required by user j.
$\Delta C(P_n)_j$	=	Pollutant n concentration increment produced by user j.
$QS(P_n)_i$	=	Maximum mass discharge of pollutant n acceptable by surface source i.
$QG(P_n)_k$	=	Maximum mass discharge (MGD x mg/l) of pollutant n acceptable by ground source k.
CAP	=	Capacity of water or wastewater treatment plant ℓ.
P_n	=	Pollutant n = 1, 2, . . . , NPOL.
NPOL	=	Number of pollutants
$\eta_{j,n}$	=	Removal efficiency for user j of pollutant n
$\eta_{\ell,n}$	=	Removal efficiency for water or wastewater treatment plant of pollutant n.
WT	=	Set of water treatment plants
WWT	=	Set of wastewater treatment plants.

Model Constraints

The model constraints are presented using the notation described in the preceding section.

Demand Constraints--This set of linear constraints forces the demand for each user j to be satisfied

$$\sum_i XS_{ij} + \sum_k XG_{kj} + \sum_\ell XT_{\ell j} + \sum_t X_{tj} \geq DEM_j \qquad j = 1, 2, \ldots, NU \tag{2.1}$$

Every user demand may be satisfied from every other element in the system, including surface and groundwater sources, water treatment plants, wastewater treatment plants and other users.

Mass Balance Constraints--These sets of linear constraints prevent violation of any mass balances throughout the system for the users, water treatment plants and wastewater treatment plants

a). For users

$$\sum_i XS_{ij} + \sum_k XG_{kj} + \sum_\ell XT_{\ell j} + \sum_t X_{tj} - \sum_i XSR_{ji} - \sum_k XGR_{jk} - \tag{2.2}$$

$$\sum_{\ell \varepsilon WWT} XTR_j - \sum_t X_{jt} = L_j \qquad j = 1, 2, \ldots, NU$$

The water losses, including consumption, are assumed constant for each user.

b). For water treatment plants

$$\sum_i XST_{i\ell} + \sum_k XGT_{k\ell} - \sum_j XT_{\ell j} = LT \qquad \underset{\ell \varepsilon WT}{\ell = 1, 2, \ldots, NT1;} \tag{2.3}$$

Water treatment plants are assumed to receive water only from surface and groundwater sources, and send water to users.

c). For wastewater treatment plants

$$\sum_j XTR_{j\ell} - \sum_j XT_{\ell j} - \sum_i XSTR_{\ell i} - \sum_k XGTR_{\ell k} = LT \qquad \underset{\ell \varepsilon WWT}{\ell = 1, 2, \ldots, NT2;} \tag{2.4}$$

Wastewater treatment plants only receive water from the users. Treated water can be returned to users or discharged to the surface and groundwater sources and water sinks.

Capacity Constraints--These linear constraints limit the water entering a treatment plant to its capacity.

a). Water treatment plants

$$\sum_j XST_{j\ell} + \sum_k XGT_{k\ell} \leq CAP_\ell \qquad \underset{\ell \varepsilon WT}{\ell = 1, 2, \ldots, NT1;} \tag{2.5}$$

b). Wastewater treatment plants

$$\sum_j XTR_{j\ell} \leq CAP_\ell \qquad \underset{\ell \varepsilon WWT}{\ell = 1, 2, \ldots, NT2;} \tag{2.6}$$

Water Availability Constraints--These linear constraints prevent withdrawal of water from exceeding the maximum allowable for each source.

a). Surface water sources

$$\sum_j XS_{ij} + \sum_{\ell \varepsilon WT} XST_{i\ell} - \sum_j XSR_{ji} - \sum_{\ell \varepsilon WWT} XSTR_i \leq SWA_i \qquad i = 1, 2, \ldots, NS \tag{2.7}$$

b). Groundwater sources

$$\sum_j XG_{kj} + \sum_\ell XGT_{k\ell} - \sum_j XGR_{jk} - \sum_\ell XGTR_{\ell k} \leq GWA_k \qquad k = 1, 2, \ldots, NG \tag{2.8}$$

Water Quality Constraints for Users--This set of non-linear constraints forces the flow distribution in the system to satisfy the quality requirements of each user. These constraints are derived from the fact that the concentration of the water being used by the user is the result of the mixture of all flows entering the user node. Therefore, the constraint requires that the resulting concentration of the mixture be less than or equal to the user's criteria, i.e.,

$$\begin{aligned} &\sum_i CS(P_n)_i XS_{ij} + \sum_k CG(P_n)_k XG_{kj} + \sum_\ell CT(P_n)_\ell XT_{\ell j} + \sum_t C(P_n)_t X_{tj} - \\ &STD(P_n)_j \left[\sum_i XS_{ij} + \sum_k XG_{kj} + \sum_\ell XT_{\ell j} + \sum_t X_{tj}\right] < 0 \qquad \begin{array}{l} j = 1, 2, \ldots, NU \\ n = 1, 2, \ldots, NPOL \end{array} \end{aligned} \tag{2.9}$$

Water Quality Constraints for Maximum Discharge to Sources--These nonlinear constraints consider the legal aspects associated with the water allocation and reuse planning process. The limitation on the amount of pollutants which can be discharged is achieved by controlling the mass of pollutant per unit time to be less than or equal to the maximum acceptable mass discharge.

a). Surface water sources

$$\sum_j C(P_n)_j \, XSR_{ji} + \sum_{\ell \varepsilon WWT} CT(P_n)_\ell \, XSTR_{\ell i} \leq QS(P_n)_i \tag{2.10}$$

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

$$n = 1, 2, \ldots, NPOL$$

b). Groundwater sources

$$\sum_j C(P_n)_j \, XGR_{jk} + \sum_{\ell \varepsilon WWT} CT(P_n)_\ell \, XGTR_{\ell k} \leq QG(P_n)_k \tag{2.11}$$

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

$$n = 1, 2, \ldots, NPOL$$

It should be noted that constraint Eqs. 2.10 and 2.11 limit the total mass of pollutant being discharged. However, in some instances the concentration, rather than the mass of pollutant, is of primary concern. In such cases, in addition to the above constraints, an upper limit is placed on the pollutant concentration variables. Such an upper bound would be determined by legal agencies, and it would prevent the discharge of any effluent with a higher concentration.

The following constraints account for the quality changes produced on the water as it passes through the system. These constraints are basically mass balance equations for the pollutants under consideration during the planning process.

Water Quality Changes Produced by Users--The concentration leaving the user is expressed in terms of the pollutant concentration entering the user, the pollutant concentration increment due to usage, and the pollutant removal achieved if an "in-situ" treatment facility is available prior to discharge. The concentration of pollutant n entering user j, $\hat{C}(P_n)_j$, is expressed as:

$$\hat{C}(P_n)_j = \frac{\sum_i CS(P_n)_i \; XS_{ij} + \sum_k CG(P_n)_k \; XG_{kj} + \sum_\ell CT(P_n)_\ell \; XT_{\ell j} + \sum_t C(P_n)_t \; X_{tj}}{\sum_i XS_{ij} + \sum_k XG_{kj} + \sum_\ell XT_{\ell j} + \sum_t X_{tj}} \tag{2.12}$$

and the concentration of pollutant n leaving user j, $C(P_n)_j$, is expressed as the penetration fraction of the concentration of the mixture entering user j, plus the concentration increment produced by user j:

$$C(P_n)_j = \left[\hat{C}(P_n)_j + \Delta C(P_n)_j\right](1 - \eta_{j,n}) \tag{2.13}$$

Substituting Eq. 2.12 in Eq. 2.13 and rearranging, the following constraint is obtained

$$\left[\frac{C(P_n)_j}{1 - \eta_{j,n}} - \Delta C(P_n)_j\right] \left(\sum_i XS_{ij} + \sum_k XG_{kj} + \sum_\ell XT_{\ell j} + \sum_t X_{tj}\right) - \tag{2.14}$$

$$\left[\sum_i CS(P_n)_i \; XS_{ij} + \sum_k CG(P_n)_k \; XG_{kj} + \sum_\ell CT(P_n)_\ell \; XT_{\ell j} + \sum_t C(P_n)_t \; X_{tj}\right] = 0$$

$$\forall \; j,n$$

Water Quality Changes Produced by Water Treatment Plants--The concentration leaving a water treatment plant is expressed in terms of the concentration of the water entering the plant and the removal efficiency of the plant. The concentration of pollutant n entering water treatment plant ℓ, $\hat{CT}(P_n)_\ell$, is the resulting concentration of the mixture of all influents from various freshwater sources entering the plant:

$$\hat{CT}(P_n) = \frac{\sum_i CS(P_n)_i XST_{i\ell} + \sum_k CG(P_n)_k \; XGT_{k\ell}}{\sum_i XST_{i\ell} + \sum_k XGT_{k\ell}} \quad ; \quad \ell \varepsilon WT \tag{2.15}$$

and the concentration of pollutant n leaving water treatment plant ℓ, $CT(P_n)_\ell$, is the fraction not removed by the treatment process;

$$CT(P_n)_\ell = [\hat{CT}(P_n)_\ell] \; (1 - \eta_{\ell,n}) \quad ; \quad \ell \varepsilon WT \tag{2.16}$$

Substituting Eq. 2.15 in Eq. 2.16 and rearranging, the following constraint is obtained:

$$\left[\frac{CT(P_n)_\ell}{1-\eta_{\ell,n}}\right] (\sum_i XST_{i\ell} + \sum_k GT_{k\ell}) - [\sum_i CS(P_n)_i\ XST_{i\ell} + \sum_k CG(P_n)_k XGT_{k\ell}] = 0 \tag{2.17}$$

$\ell = 1, 2, \ldots, NT1; \ell \varepsilon WT$

$n = 1, 2, \ldots, NPOL$

Water Quality Changes Produced by Wastewater Treatment Plants--The concentration leaving the wastewater treatment plant is expressed in terms of the concentration of the water entering the plant and the removal efficiency of the plant. The concentration of pollutant n entering wastewater treatment plant ℓ, $\hat{CT}(P_n)_\ell$, is the concentration of the mixture of all flows entering the plant;

$$\hat{CT}(P_n)_\ell = \frac{\sum_t C(P_n)_t\ XTR_{t\ell}}{\sum_t XTR_{t\ell}} \quad ; \ell \varepsilon WWT \tag{2.18}$$

and the concentration of pollutant n leaving wastewater treatment plant ℓ, $CT(P_n)_\ell$, is the fraction not removed by the treatment process;

$$CT(P_n)_\ell = [\hat{CT}(P_n)_\ell]\ (1-\eta_{\ell,n}) \quad ; \quad \ell \varepsilon WWT \tag{2.19}$$

Substituting Eq. 2.18 in 2.19 and rearranging, the following constraint is obtained:

$$\left[\frac{CT(P_n)_\ell}{1-\eta_{\ell,n}}\right] (\sum_t XTR_{t\ell}) - [\sum_t C(P_n)_t\ XTR_{t\ell}] = 0 \tag{2.20}$$

$\ell = 1, 2, \ldots, NT2; \ell \varepsilon WWT$

$n = 1, 2, \ldots, NPOL$

The model constraints are Eqs. 2.1 to 2.11, 2.14. 2.17 and 2.20. They involve two basic types of decision variables, flows through each arc and concentration leaving each node.

Objective Function

As previously stated, the objective is to determine the minimum cost solution to the problem of supplying water to every user in the region considering water reuse. The costs include the water and wastewater treatment costs and the transportation costs consisting of piping and pumping costs. The pipe costs are expressed in the form αQ^β, where Q is the flow transported through the pipe. α and β are coefficients with β expressing the economies of scale, where $0<\beta<1$, EPA (1978). The piping costs are expressed as

$$\text{Piping Costs} = \sum_{w \in W} \alpha_w Q_w^{\beta_w} \tag{2.21}$$

where W is the set of possible pipes connecting sources, users, treatment plants, and disposal sites (sinks). The pumping costs are expressed in a similar form as

$$\text{Pumping Costs} = \sum_{z \in Z} \alpha_z Q_z^{\beta_z} \tag{2.22}$$

where Z is the subset of pipes requiring pumping, with $Z \varepsilon W$. For notational simplicity, the flows, Q_w and Q_z, are used in place of the "X" variables which represent flows between various sources, users, treatment plants and disposal sites.

The water and wastewater treatment plant costs are expressed in a similar form as, EPA (1978, 1979):

$$\text{Water Treatment Plant Cost} = \sum_{\ell \varepsilon WT} \alpha_\ell [\sum_i XST_{i\ell} + \sum_k XGT_{k\ell}]^{\beta_\ell} \tag{2.23}$$

$$\text{Wastewater Treatment Plant Cost} = \sum_{\ell \varepsilon WWT} \alpha_\ell [\sum_j XTR_{j\ell}]^{\beta_\ell} \tag{2.24}$$

These cost functions for the water and wastewater treatment plants are only a function of flow through the plants and do not include the percentage removal achieved by the treatment process.

The objective function is the minimization of the sum of piping, pumping, and water and wastewater treatment costs, i.e.,

$$\text{Min} \left[\sum_{w \varepsilon W} \alpha_w Q_w^{\beta_w} + \sum_{z \varepsilon Z} \alpha_z Q_z^{\beta_z} + \sum_{\ell \varepsilon WT} \alpha_\ell (\sum_i XST_{i\ell} + \sum_k XGT_{k\ell})^{\beta_\ell} + \sum_{\ell \varepsilon WWT} \alpha_\ell (\sum_j XTR_{j\ell})^{\beta_\ell} \right] \tag{2.25}$$

Other costs relating to source development such as intake structures for surface water sources and well construction, development for groundwater sources, and construction of new pipe lines can be included. This also applies to disposal costs. The moded is to minimize costs, Eq. 2.25, subject to the constraints, Eqs. 2.1 - 2.11, 2.14, 2.17 and 2.20. The solution will indicate flows through the network and concentrations of pollutants in the water leaving each node.

The model has a nonlinear objective function and both linear and nonlinear constraints requiring the use of a solution technique such as the large scale generalized reduced gradient (LSGRG) technique by Lasdon, Waren, Jain and Saunders (1979). LSGRG has proved to be useful for solving large nonlinear problems rather efficiently. The LSGRG was developed to deal with large, sparse, nonlinearly constrained problems.

2.2 MULTI-PERIOD MODEL

This section describes the multi-period model for the water allocation planning problem which is an extension of the single period model described in Section 2.1. The multi-period model considers the time variations of the system parameters, such as water demands, water availability, quality consideration, number of users in the region, etc. This model allows for capacity expansion considerations in the planning process where the final solution indicates when and to what extent treatment facilities should be built. The chapter describes the physical characteristics of the system and its components, giving a detailed explanation of how these elements are interrelated.

System Description - Physical Characteristics

The system consists of many water related elements; water sources, water treatment plants, water users, wastewater treatment plants, and water sinks. Each element has its own characteristics and interacts with other elements in the system as described below:

(1) Water Sources and Sinks. Surface and groundwater sources, as well as sinks, are in this category. Each source is defined by: (i) its location in the region, (ii) its water availability, which is expressed in terms of the maximum flow which can be consistently withdrawn during each planning period, (iii) its pollutant assimilation, which is defined in terms of the maximum mass of pollutant per unit time the source can accept during each planning period, and (iv) its quality profile, which is defined in terms of the concentrations of each pollutant being considered during the planning process. A quality profile for each planning period must be provided, which implies an estimation of the source behavior subject to a pollutant discharge. Water sources can send water to water treatment plants, where its quality can be improved; or to users without previous treatment. Sources can receive wastewater treatment plant effluents and user discharges.

(2) Water Treatment Plants. Existing and potential water treatment plants are included. Each plant is defined by: (i) the location in the region, (ii) the flow capacity if a water treatment plant exists at the beginning of the planning horizon, (iii) the water losses during each period, (iv) the removal efficiencies for each period, which implies that the planner selects "a priori" the type of treatment each treatment plant will provide during each period. This leaves the flow capacity as the only decision for each plant. The water treatment plant flow capacity is used as an upper bound on the variable representing the capacity of the plant in the initial period. This implies that added capacity may be built no sooner than the second period. If the water treatment plant is a potential one, then no upper bound is placed on the variable and the capacity can be built in the first period as large as the solution to the model requires. Water treatment plants may send their effluents only to users in the region, and may only receive water from water sources.

(3) Water Users. A user can be defined as any element in the system which exerts a demand of water. Industries, municipalities, agricultural sites are major users. Besides its water demand during each period, a user is defined by: (i) its location, (ii) the water losses during each period, (iii) the quality requirements for each period, which represent the maximum concentrations that can be accepted, (iv) the impact on the quality of the water, which is defined in terms of the concentration increment caused by the user after being utilized, (v) the treatment facilities during each period, as given by the removal percentage achieved. This considers the possibility of a user having its own treatment plant prior to discharging its effluent and considers the possibility of recycling its own wastewater. A concentration increment must be provided for each pollutant considered during each period. A user may interact with every other element in the system. It can receive water directly from a source, a water treatment plant, a wastewater treatment plant, or another user. It can send its effluents to another user, to a wastewater treatment plant or to a sink or source.

(4) Wastewater Treatment Plants. Existing and potential wastewater treatment plants are included. Each plant is defined by (i) the location, (ii) the flow capacity if a wastewater treatment plant exists at the beginning of the planning horizon, (iii) the water losses during each period, depending on the treatment process involved, and (iv) the removal efficiencies for each period. The wastewater treatment plant flow capacity is used as an upper bound on the variable representing the capacity, $CAP_{\ell 1}$, of plant ℓ in the initial period. This implies that additional capacity for such plant may be built no sooner than period two. If the wastewater treatment plant is a potential one, no upper bound is placed on the variable $CAP_{\ell 1}$, and the capacity may be built in the first period and as large as the solution to the model requires.

A wastewater treatment plant may send its effluent to a sink or source or to a user. It may receive water from the users.

Problem Statement

A word statement of the problem is a follows:

Objective--To find the minimum total present cost solution to the problem of supplying water from different types and locations of sources to every user in the region during each planning period, allowing for water recycling and reuse. The cost includes the water cost, the operation and maintenance cost of piping, pumping and treatment facilities, the construction cost of new pipes and treatment facilities, and the cost of additional capacity of the treatment plants.

Constraints--(1) demand constraints; (2) capacity constraints; (3) mass balance constraints; (4) water availability constraints; (5) water quality constraints (a) users quality requirements; (b) maximum mass discharge of pollutant to sources; (c) quality changes produced by users;

(d) quality changes produced by water treatment plants; (e) quality changes produced by wastewater treatment plants; (6) capacity expansion constraints.

Nomenclature

The following nomenclature is useful for the interpretation of the multi-period model:

Decision Variables

XS_{ijp} = Amount of freshwater sent from surface water source i to user j in period p without treatment.

XG_{kjp} = Amount of freshwater sent from groundwater source k to user j in period p without treatment.

$XT_{\ell jp}$ = Amount of treated water sent from water or wastewater treatment plant ℓ to user j in period p.

XSR_{jip} = Amount of wastewater sent from user j to surface water source i in period p.

XGR_{jkp} = Amount of wastewater sent from user j to groundwater source k in period p.

$XTR_{j\ell p}$ = Amount of wastewater sent from user j to wastewater treatment plant ℓ in period p.

$XST_{i\ell p}$ = Amount of freshwater sent from surface water source i to water treatment plant ℓ in period p.

$XGT_{k\ell p}$ = Amount of freshwater sent from groundwater source k to water treatment plant ℓ in period p.

$XSTR_{\ell ip}$ = Amount of treated water sent from wastewater treatment plant ℓ to surface water source in period p.

$XGTR_{\ell kp}$ = Amount of treated water sent from wastewater treatment plant ℓ to groundwater source k in period p.

X_{jrp} = Amount of water sent from user j to user r in period p.

$CAP_{\ell p}$ = Flow capacity of water or wastewater treatment plant ℓ in period p.

$CT(P_n)_{\ell p}$ = Concentration of pollutant n leaving water or wastewater treatment plant ℓ in period p.

$C(P_n)_{jp}$ = Concentration of pollutant n leaving user j in period p.

The units for all flows are in million gallons per day (MGD) and the units for pollutant concentration are milligrams per liter, mg/ℓ.

Required Input

NU_p = Number of users in period p.

$NT1_p$ = Number of water treatment plants in period p.

$NT2_p$ = Number of wastewater treatment plants in period p.

NS_p = Number of surface water sources in period p.

NG_p = Number of groundwater sources in period p.

$CS(P_n)_{ip}$ = Concentration of pollutant n leaving surface water source i in period p.

$CG(P_n)_{kp}$ = Concentration of pollutant n leaving groundwater source k in period p.

DEM_{jp} = Demand of water required by user j in period p.

L_{jp} = Water losses or consumption by user j in period p.

$LT_{\ell p}$ = Water losses at water or wastewater treatment plant ℓ in period p.

SWA_{ip} = Maximum flow rate which can be withdrawn from surface water source i in period p.

GWA_{kp} = Maximum flow rate which can be withdrawn from groundwater source k in period p.

$STD(P_n)_{jp}$ = Maximum concentration of pollutant n accepted by user j in period p.

$\Delta C(P_n)_{jp}$ = Concentration increment of pollutant n produced by user j in period p.

$QS(P_n)_{ip}$ = Maximum mass discharge of pollutant n acceptable by surface water source i in period p.

$QG(P_n)_{kp}$ = Maximum mass discharge of pollutant n acceptable by groundwater source k in period p.

P_n = Pollutant n = 1, 2, . . . , NPOL.

NPOL = Number of pollutants.

NPP = Number of planning periods.

η_{jnp} = User j removal efficiency of pollutant n in period p.

$\eta_{\ell np}$ = Water or wastewater treatment plant ℓ removal efficiency of pollutant n in period p.

WT_p = Set of water treatment plants in period p.

WWT_p = Set of wastewater treatment plants in period p.

The units of the mass discharge of pollutants are in MGD mg/ℓ, and all others are previously described.

Model Constraints

The model constraints are presented using the notation described in the preceding section.

Demand Constraints--This set of linear constraints forces the demand for each user j to be satisfied during each planning period p.

$$\sum_i XS_{ijp} + \sum_k XG_{kjp} + \sum_\ell XT_{\ell jp} + \sum_r X_{rjp} \geq DEM_{jp} \tag{2.26}$$

$$j = 1, 2, \ldots, NU_p$$
$$p = 1, 2, \ldots, NNP$$

Mass Balance Constraints--These sets of linear constraints prevent violation of any mass balances throughout the system for the users, water treatment plants, and wastewater treatment plants.

a). For users

$$\sum_i XS_{ijp} + \sum_k XG_{kjp} + \sum_\ell XT_{\ell jp} + \sum_r X_{rjp} - \sum_i XSR_{jip} -$$

$$\sum_k XGR_{jkp} - \sum_{\ell \varepsilon WWT} XTR_{j\ell p} - \sum_r X_{jrp} = L_{jp} \tag{2.27}$$

$$j = 1, 2, \ldots, NU_p$$
$$p = 1, 2, \ldots, NPP$$

The water losses, including consumption, are assumed constant for each user during each period.

b). For water treatment plants

$$\sum_i XST_{i\ell p} + \sum_k XGT_{k\ell p} - \sum_j XT_{\ell jp} = LT_{\ell p} \tag{2.28}$$

$$\ell = 1, 2, \ldots, NT1_p; \ell \varepsilon\ WT_p$$
$$p = 1, 2, \ldots, NPP$$

Water treatment plants may receive water from surface water and groundwater sources and send water to users.

c). For wastewater treatment plants

$$\sum_j XTR_{j\ell p} - \sum_j ST_{\ell jp} - \sum_i XSTR_{\ell ip} - \sum_k XGTR_{\ell kp} = LT_{\ell p} \tag{2.29}$$

$$\ell = 1, 2, \ldots, NT2_p; \ell \varepsilon\ WWT_p$$
$$p = 1, 2, \ldots, NPP$$

Wastewater treatment plants receive water from users in the system. Their treated effluents can be returned to the users or they can be discharged on the surface or groundwater sources.

Capacity Constraints--These linear constraints limit the water entering a water or wastewater treatment plant to the capacity during that period.

a). Water treatment plants

$$\sum_i XST_{i\ell p} + \sum_k XGT_{k\ell p} - CAP_{\ell p} \leq 0 \tag{2.30}$$

$$\ell = 1, 2, \ldots, NT1_p;\ \ell \varepsilon WT_p$$

$$p = 1, 2, \ldots, NPP$$

b). Wastewater treatment plants

$$\sum_j XTR_{j\ell p} - CAP_{\ell p} \leq 0$$

$$\ell = 1, 2, \ldots, NT2_p;\ \ell \varepsilon WWT_p \tag{2.31}$$

$$p = 1, 2, \ldots, NPP$$

If a water or wastewater treatment plant is existent at the beginning of the planning horizon, then the variable $CAP_{\ell 1}$, which represents the flow capacity of treatment plant ℓ in period 1, is upper bounded to the existing capacity and does not contribute any construction cost, with only operation and maintenance costs. This implies that such an existing plant can be expanded no sooner than the beginning of the second period.

Capacity Expansion Constraints--These linear constraints represent the linkage between successive planning periods for water and wastewater treatment plants. These constraints prevent the selection of flow capacities that decrease with time for water or wastewater treatment plants.

a). Water treatment plants

$$CAP_{\ell,p} - CAP_{\ell,\ p+1} \leq 0$$

$$\ell = 1, 2, \ldots, NT1_p;\ \ell \varepsilon WT_p \tag{2.32}$$

$$p = 1, 2, \ldots, NPP-1$$

b). Wastewater treatment plants

$$CAP_{\ell,p} - CAP_{\ell,p+1} \leq 0 \tag{2.33}$$

$$\ell = 1, 2, \ldots, NT2_p;\ \ell \varepsilon WWT_p$$

$$p = 1, 2, \ldots, NPP-1$$

Water Availability Constraints--These linear constraints prevent withdrawal of water from exceeding the maximum allowable for each source during each time period.

(a) Surface water sources

$$\sum_j XS_{ijp} + \sum_{\ell \varepsilon WT} XST_{i\ell p} - \sum_j XSR_{jip} - \sum_{\ell \varepsilon WT} XSTR_{\ell ip} \leq SWA_{ip} \tag{2.34}$$

$$i = 1, 2, . . . , NS_p$$

$$p = 1, 2, . . . , NPP$$

b). Groundwater sources

$$\sum_j XG_{kjp} + \sum_{\ell \varepsilon WT} XGT_{k\ell p} - \sum_j XGR_{jkp} - \sum_{\ell \varepsilon WWT} XGTR_{\ell kp} \leq GWA_{kp} \tag{2.35}$$

$$k = 1, 2, . . ., NG_p$$

$$p = 1, 2, . . ., NPP$$

Water Quality Constraints for for Users--This set of nonlinear constraints forces the flow distribution in the system to satisfy the quality requirements of each user. These constraints are derived from the fact that the concentration of the water being used by the user is the result of the mixture of all flows entering the user node.

Therefore, the constraint requires that the resulting concentration of the mixture be less than or equal to the user's quality criteria. This is required for each period, allowing the user to change its quality requirements with time by changing its criteria at the beginning of each planning period.

$$\sum_i CS(P_n)_{ip} \, XS_{ijp} + \sum_k CG(P_n)_{kp} \, XG_{kjp} + \sum_\ell CT(P_n)_{\ell p} XT_{jp} +$$

$$\sum_r C(P_n)_{rp} \, X_{rjp} - STD(P_n)_{jp} \, [\sum_i XS_{ijp} + \sum_k XG_{kjp} + \sum_\ell XT_{\ell jp} + \sum_r X_{rjp}] \leq 0 \tag{2.36}$$

$$j = 1, 2, . . . , NU$$

$$p = 1, 2, . . . , NPP$$

$$n = 1, 2,, . . . , NPOL$$

Each user may determine its own quality criteria for each period, allowing the model to allocate water of different qualitities at minimum cost while satisfying the criteria of each user.

Water Quality Constraints for Maximum Discharge to Sources--These nonlinear constraints consider the legal aspects associated with the water allocation and reuse planning process. The limitation on the amount of pollutants which can be discharged is achieved by controlling the mass of pollutant per unit time to be less than or equal to the maximum discharge acceptable during each planning period. Surface water sources

$$\sum_j C(P_n)_{jp} \; XSR_{jip} + \sum_{\ell \varepsilon WWT} CT(P_n)_{\ell p} \; XSTR_{\ell ip} \leq QS(P_n)_{ip} \tag{2.37}$$

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

$$p = 1, 2, \ldots, NPP$$

$$n = 1, 2, \ldots, NPOL$$

Groundwater sources

$$\sum C(P_n)_{jp} XGR_{jkp} + \sum_{\ell \varepsilon WWT} CT(P_n)_{\ell p} \; XGTR_{\ell kp} \leq QG(P_n)_{kp} \tag{2.38}$$

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

$$p = 1, 2, \ldots, NPP$$

$$n = 1, 2, \ldots, NPOL$$

It should be noted that constraints 2.37 and 2.38 shown above limit the total mass of pollutant being discharged. However, in some instances the concentration, rather than the mass of pollutant, is of primary concern. In such cases, in addition to the above constraints, an upper limit should be placed on the concentration variables. Such an upper bound would be determined by legal agencies and would prevent any effluent with a higher concentration to be discharged.

These constraints account for the quality changes produced on the water as it passes through the system.

These constraints are basically mass balance equations for the pollutants under consideration.

Water Quality Changes Produced by Users--The concentration leaving the user is expressed in terms of the pollutant concentration entering the user, the pollutant concentration increment due to usage, and the pollutant removal achieved if an "in-situ" treatment facility is available prior to discharge. The concentration of pollutant n entering user j in period p, $\hat{C}(P_n)_{jp}$, is expressed as the result of the mixture of all flows entering user j,

$$\hat{C}(P_n)_{jp} = [\sum_i CS(P_n)_{ip} \; XS_{ijp} + \sum_k CG(P_n)_{kp} \; XG_{kjp} + \sum_\ell CT(P_n)_{\ell p} \; XT_{\ell jp} + \sum_r C(P_n)_{rp} \; X_{rjp}] \; / \; [\sum_i XS_{ijp} + \sum_k XG_{kjp} + \sum_\ell XT_{\ell jp} + \sum_r X_{jr}] \tag{2.39a}$$

and the concentration of pollutant n leaving user j in period p, $C(P_n)_{jp}$, is expressed as the penetration fraction of the concentration of the mixture entering user j, plus the concentration increment produced by user j;

$$C(P_n)_{jp} = [\hat{C}(P_n)_{jp} + \Delta C(P_n)_{jp}] (1 - \eta_{jnp}) \tag{2.39b}$$

Substituting Eqs. 2.39a and 2.39b and rearranging, the following constraint is obtained:

$$\frac{C(P_n)_{jp}}{1 - \eta_{jnp}} - \Delta C(P_n)_{jp} \left(\sum_i XS_{ijp} + \sum_k XG_{kjp} + \sum_\ell XT_{\ell jp} + \sum_r X_{rjp}\right) -$$

$$\left[\sum_i CS(P_n)_{ip} XS_{ijp} + \sum_k CG(P_n)_{kp} XG_{kjp} + \sum_\ell CT(P_n)_{\ell p} XT_{\ell jp} + \sum_r C(P_n)_{rp} X_{rjp}\right] = 0 \tag{2.40}$$

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

$$p = 1, 2, \ldots, NPP$$

$$n = 1, 2, \ldots, NPOL$$

Water Quality Changes Produced by Water Treatment Plants--The concentration leaving the water treatment plant is expressed in terms of the concentration of the water entering the plant in period p and the removal efficiency of the plant in period p. The concentration of pollutant n entering water treatment plant ℓ in period p, $\hat{CT}(P_n)_{\ell p}$, is the resulting concentration of the mixture of all influents from various freshwater sources entering the plant;

$$\hat{CT}(P_n)_{\ell p} = \frac{\sum_i CS(P_n)_{ip} XST_{i\ell p} + \sum_k CG(P_n)_{kp} XGT_{k\ell p}}{\sum_i XST_{i\ell p} + \sum_k XGT_{k\ell p}} \quad ; \quad \ell \varepsilon WT_p \tag{2.41}$$

and the concentration of pollutant n leaving water treatment plant in period p, $CT(P_n)_{\ell p}$, is the fraction not removed by the treatment process;

$$CT(P_n)_{\ell p} = [\hat{CT}(P_n)_{\ell p}] \quad (1 - \eta_{\ell np}) \ ; \quad \ell \varepsilon WT_p \tag{2.42}$$

Substituting Eq. 2.41 into 2.42 and rearranging, the following constraint is obtained:

$$\frac{CT(P_n)_{\ell p}}{1 - \eta_{\ell np}} \quad (\sum_i XST_{i\ell p} + \sum_k XGT_{k\ell p}) -$$

$$[\sum_i CS(P_n)_{ip} \; XST_{i\ell p} + \sum_k CG(P_n)_{kp} \; XGT_{k\ell p}] = 0 \tag{2.43}$$

$$\ell = 1, 2, \ldots, NT1_p; \ell \varepsilon WT_p$$

$$p = 1, 2, \ldots, NPP$$

$$n = 1, 2, \ldots, NPOL$$

Water Quality Changes Produced by Wastewater Treatment Plants--The concentration leaving the wastewater treatment plant is expressed in terms of the concentration of the water entering the plant in period p, and the removal efficiency of the plant during period p. The concentration of pollutant n entering wastewater treatment plant ℓ in period p, $\hat{C}T(P_n)_{\ell p}$, is the concentartion of the mixture of all flows entering the plant;

$$\hat{C}T(P_n)_{\ p} = \frac{\sum_r C(P_n)_{rp} \; XTR_{r\ell p}}{\sum_r XTR_{r\ell p}} \quad ; \quad \ell \varepsilon WWT_p \tag{2.44}$$

and the concentration of pollutant n leaving the wastewater treatment plant ℓ in period p, $CT(P_n)_{\ell p}$, is the fraction not removed by the treatment process;

$$CT(P_n)_{\ell p} = [\hat{C}T(P_n)_{\ell p}] \; (1 - \eta_{\ell np}) \; ; \; \ell \varepsilon WWT_p \tag{2.45}$$

Substituting Eqs. 2.44 into 2.45 and rearranging, the following constraint is obtained:

$$\frac{CT(P_n)_{\ell p}}{1 - \eta_{\ell np}} \quad (\sum_r XTR_{r\ell p}) - [\sum_r C(P_n)_{rp} \; XTR_{r\ell p}] = 0 \tag{2.46}$$

$$\ell = 1, 2, \ldots, NT2_p; \ell \varepsilon WWT_p$$

$$p = 1, 2, \ldots, NPP$$

$$n = 1, 2, \ldots, NPOL$$

The model constraints are Eqs. 2.26 to 2.38, 2.40, 2.43 and 2.46. They involve three basic types of decision variables, flows through each arc during each period and concentrations leaving each node during each period, and treatment plant capacities.

Objective Function

The objective of the multi-period model is to determine the minimum total discounted cost. The costs include the operation and maintenance of piping, pumping, and treatment facilities; and the construction of new facilities, e.g., new treatment plants, additional capacity of existent plants, upgrading of existent treatment facilities and new pipelines. The operational piping costs are expressed in the form Q , where Q is the flow transported through the pipe, and and are coefficients with expressing the economies of scale. The total operational costs associated with piping during a given planning period are

$$\text{Operational Piping Costs} = \sum_{w \in W_p} \alpha_w Q_w^{\beta_w} \tag{2.47}$$

where W_p is the set of possible pipes connecting sources, users, water and wastewater treatment plants, and disposal sites during period p. The construction costs for new pipes are expressed in a similar form as

$$\text{Construction Pipes Costs} = \sum_{w \in NW_p} \alpha_w Q_w^{\beta_w} \tag{2.48}$$

where NW_p is the set of possible pipes to be constructed for operation during period p.

The operational pumping costs are expressed in a form similar to the operational piping costs as follows:

$$\text{Operational Pumping Costs} = \sum_{z \varepsilon Z_p} \alpha_z Q_z^{\beta_z} \tag{2.49}$$

where Z_p is the subset of pipes requiring pumping during period p, with $Z_p \varepsilon W_p$. Q_w and Q_z, which represent the flows between various sources, users, treatment plants, and disposal sites are the decision variables; XS_{ijp}, XG_{kjp}, $XT_{\ell jp}$, XSR_{jip}, XGR_{kip}, $XTR_{j\ell p}$, $XST_{i\ell p}$, $XGT_{k\ell p}$, $XSTR_{\ell ip}$, $XGTR_{\ell kp}$, and X_{rjp}. The above notation, using W_p and Z_p, is introduced for simplicity in describing the piping and pumping costs involved in the objective function.

The water and wastewater treatment plant costs are of the similar form, $\alpha[\Sigma Q]^{\beta}$. The water and wastewater treatment plant operational costs during period p are expressed respectively as

$$\begin{array}{l}\text{Water Treatment Plant} \\ \text{Operational Costs}\end{array} = \sum_{\ell \varepsilon WT_p} \alpha_\ell [\sum_i XST_{i\ell p} + \sum_k XGT_{k\ell p}]^{\beta_\ell} \tag{2.50}$$

$$\begin{array}{l}\text{Wastewater Treatment} \\ \text{Plant Operational Cost}\end{array} = \sum_{\ell \varepsilon WWT_p} \alpha_\ell [\sum_j XTR_{j\ell p}]^{\beta_\ell} \tag{2.51}$$

The construction costs associated with new water or wastewater treatment plants at the beginning of a planning period are also expressed by exponential functions similar to Eq. 2.48, to represent economies of scale involved in the selection of the design capacity of a new facility.

$$\text{Water Treatment Plant Construction Costs} = \sum_{\ell \varepsilon WT_p} \theta_\ell [CAP_{\ell p}]^{\phi_\ell} \tag{2.52}$$

$$\text{Wastewater Treatment Plant Construction Costs} = \sum_{\ell \varepsilon WWT_p} \theta_\ell [CAP_{\ell p}]^{\phi_\ell} \tag{2.53}$$

If a water or wastewater treatment plant is already existent at the the beginning of period p, and an expansion of its current capacity is required, then the costs associated with the added capacity are a function of the increment in capacity and also show economies of scale. These costs can be expressed as a single payment cost at the beginning of period p as follows:

$$\text{Expansion Construction Costs for Existent Water Treatment Plants} = \sum_{\ell \varepsilon WT_p} \Omega_\ell [CAP_{\ell,p+1} - CAP_{\ell,p}]^{\gamma_\ell} \tag{2.54}$$

$$\text{Expansion Construction Costs for Existent Wastewater Treatment Plants} = \sum_{\ell \varepsilon WWT_p} \Omega_\ell [CAP_{\ell,p+1} - CAP_{\ell,p}]^{\gamma_\ell} \tag{2.55}$$

The costs associated with treatment plants, operational, construction of new plants, or expansion of old ones are only a function of flow through the plants and do not include the percentage removal achieved by the treatment process directly, but rather indirectly in the selection of the function coefficients: θ_ℓ, ϕ_ℓ , Ω_ℓ ,γ_ℓ . In order for the model to achieve valid comparisons among different alternatives, it is necessary to refer all costs to a common and unique basis. Operational costs, which are expressed in $/time, equally distributed during the length of a planning period, should be converted to an equivalent total cost at the beginning of the period. This total cost should then be converted to an equivalent cost at the beginning of the planning horizon, as shown in Figure 2.5.

The model handles these costs as follows:

a). Compute the annual cost during planning period p, A_p, using cost data for the operation of the facilities in turn.

b). Compute the total cost equivalent at the beginning of period p;

$$TC_p = A_p \left[\frac{(1 + i)^{N_p} - 1}{i(1 + i)^{N_p}}\right] \tag{2.56}$$

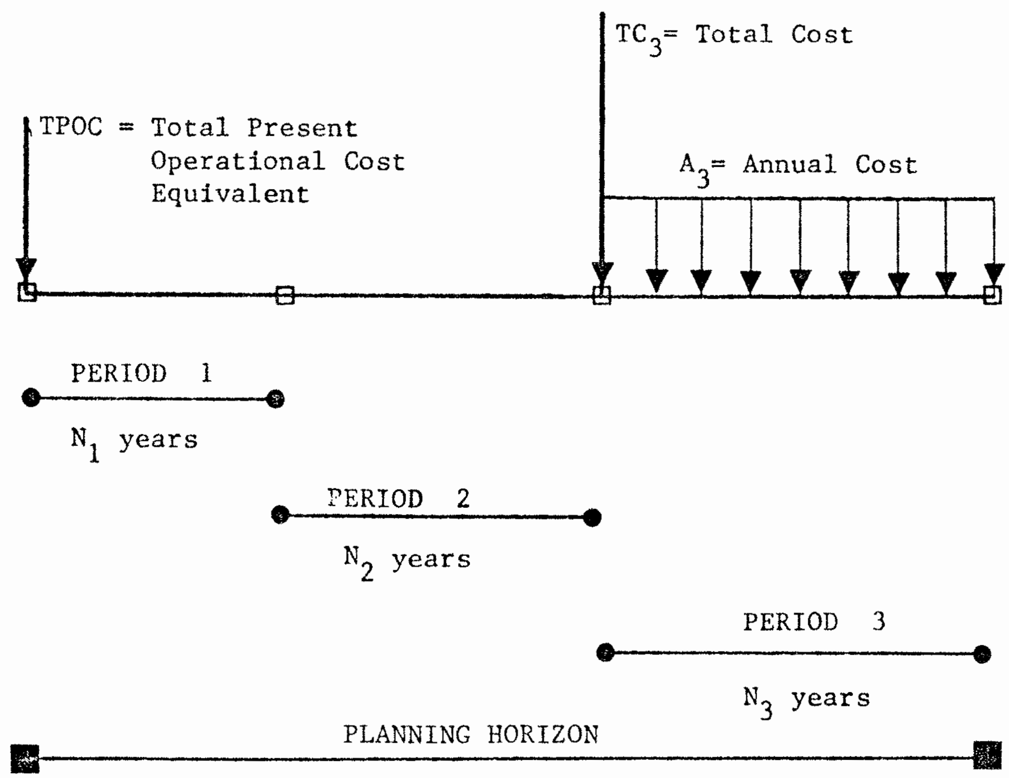


FIGURE 2.5 Operational Costs Present Cost Equivalent

where

TC_p = the total cost equivalent at the beginning of period p,

A_p = the annual operational cost during peirod p,

N_p = the duration of period p in years, and

i = the interest rate in %.

c). Compute the total present cost equivalent at the beginning of the planning horizon.

$$TPOC_p = TC_p[1/(1 + i)^{NCUM_{p-1}}] \tag{2.57}$$

where

$TPOC_p$ = the total present cost equivalent for operational cost in period p,

TC_p = the total cost equivalent at the beginning of period p, and

$NCUM_{p-1} = \sum_{j=1}^{p-1} N_j$

Construction costs are handled in a similar manner as the operational costs. Since construction costs are expressed in a single payment as $ at the beginning of the planning period in which the new facility is already available; this single payment should be converted to an equivalent total single payment at the beginning of the planning horizon, as shown in Figure 2.6. The model handles the construction costs as follows:

a). Compute the single payment construction cost at the beginning of period p, CC_p, using the cost data for the construction of new facilities.

b). Compute the total present construction cost equivalent at the beginning of the planning horizon

$$TPCC_p = CC_p\ [1/(1 + i)^{NCUM_{p-1}}] \tag{2.58}$$

where

$TPCC_p$ = total present cost equivalent for construction cost at the beginning of period p and

CC_p = Construction cost single payment at the beginning of period p and

$NCUM_{p-1} = \sum_{j=1}^{p-1} N_j$; N_j = duration of period j in years.

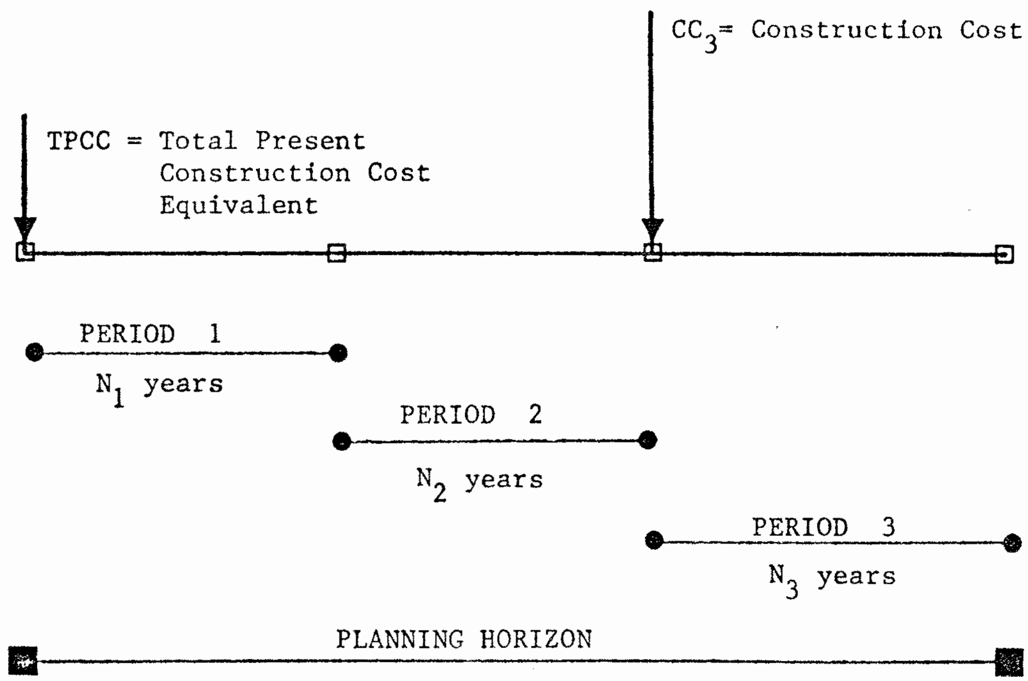


FIGURE 2.6 Construction Costs Present Cost Equivalent

The objective function can then be expressed to minimize the sum of the present cost equivalent for operational piping, pumping, and treatment costs plus the present cost equivalent for the construction of new pipes and treatment facilities or capacity expansion for every planning period as

$$\text{Min} \sum_{p=1}^{NPP} TPOC_p + \sum_{p=1}^{NPP} TPCC_p \tag{2.59}$$

Other costs relating to source development such as intake structures for surface water sources and well construction, development for groundwater sources, and water costs can be included. This also applies to disposal costs.

2.3 SOLUTION TECHNIQUES

This section describes two alternative methods for solving large scale nonlinear problems. One, the large scale generalized reduced gradient (LSGRG) method is based on a search procedure over a reduced number of variables. The method is capable of starting from either feasible or non-feasible points. The second is the successive linear programming (SLP) method which linearizes the nonlinar terms in the objective function and in the nonlinear constraints, and a set of linear problems is solved at each iteration until an optimum criterion is satisfied.

This section also presents a procedure to find "good" initial solutions for the nonlinear single period planning model. The procedure exploits the structure of the model and uses widely accepted standard optimization techniques such as, linear programming (LP) and a network programming code, out-of-kilter algorithm (OKA). These initial solutions are then used to solve the nonlinear optimization codes. Using the initial solutions from the OKA-LP procedure results in considerable reductions in computer time, since both LSGRG and SLP codes are strongly dependent upon initial solutions.

Reduced Gradient Methods

Reduced gradent methods find the optimum of a function of n variables subject to linear constraints. The objective function can be nonlinear with continuous first partial derivatives. From a computational viewpoint, the method is closely related to the simplex method of linear programming in that the problem variables are partitioned into basic and non-basic groups.

The idea of the reduced gradient method is to consider, at any stage, the problem only in terms of the non-basic variables. Since the vector of basic variables can be determined by solving the set of linear constraints, the objective function can be considered to be a function of the non-basic variables only. From this viewpoint, the only constraints are the bound constraints on the non-basic variables; and a simple modification of steepest descent accounting for these constraints can be executed, which uses the gradient vector of the function as the search direction. The gradient with respect to the independent or non-basic variables is called the reduced gradient.

In the case where not only the objective function but also some of the constraints are nonlinear, the reduced method is modified to include these nonlinearities. This method is known as the generalized reduced gradient (GRG) method. Abadie and Carpenter (1969) first presented a GRG method to solve nonlinear problems with nonlinear constraints. Recently, a number of other versions of GRG have been developed and implemented. GRG algorithms are designed to solve problems of the following form:

$$\text{minimize } f(x) \tag{2.60}$$

$$\text{subject to } h(x) = 0 \tag{2.61}$$

$$\ell \leq x \leq u \tag{2.62}$$

where x is a vector of n variables, f is the objective function, h is a vector of m equality constraints, linear or nonlinear, and and u are vectors of lower and upper bounds. GRG algorithms use the m equality constraints to solve for m of the variables, called basic variables, in terms of the remaining n-m non-basic variables. Denoting the basic and non-basic variables as X_b and X_{nb}, respectively, the constraint equations become

$$h(X_b, X_{nb}) = 0 \tag{2.63}$$

The Jacobian matrix of h, $\frac{\partial h}{\partial x}$, may be similarly partitioned as

$$\frac{\partial h}{\partial x} = \left(\frac{\partial h}{\partial X_b}, \frac{\partial h}{\partial X_{nb}}\right) = (B, b_{nb}) \tag{2.64}$$

where, for simplicity, the variables are assumed renumbered so the basics are the first m components of x.

A feasible point, $\overline{X}$, is one that satisfies constraint Eqs. 2.61 and 2.62. Assuming that a feasible point is known, the specific variables chosen as the basic variables are selected so that B is non-singular when evaluated at $\overline{X}$. In this case, the constraints (2.61) can be solved for X_b in terms of X_{nb} to yield the basics as a function of the non basics, $X_b(X_{nb})$. The representation is valid for all X_{nb} sufficiently near $\overline{X}_{nb}$. The objective function is then reduced to a function of X_{nb} only,

$$f(X_b(X_{nb}), X_{nb}) = F(X_{nb}) \tag{2.65}$$

and the original problem, Eqs. 2.60 - 2.62, is transformed to a simplier reduced problem:

$$\text{minimize } F(X_{nb}) \tag{2.66}$$

subject to the bounds on X_{nb}. The functions $F(X_{nb})$ is called the reduced objective and its gradient, ∇F, the reduced gradient.

GRG algorithms solve the original problems, Eqs. 2.60 to 2.62 by solving a sequence of reduced problems, usually by a gradient method. At a given iteration, the reduced gradient is computed in terms of the non-basic variables as follows:

$$\nabla F = \frac{\partial F(X_{nb})}{\partial X_{nb}} = \frac{\partial f(x)}{\partial X_{nb}} - \pi^T \left[\frac{\partial h(x)}{\partial X_{nb}}\right] \tag{2.67}$$

where π, the simplex multiplier vector, is determined from

$$\left[\frac{\partial h(x)}{\partial X_b}\right]^T \pi = \frac{\partial f(x)}{\partial X_b} \tag{2.68}$$

with all partial derivatives being evaluated at the current feasible point $\bar{X} = (\bar{X}_b, \bar{X}_{nb})$.

Reduced Gradient Methods for Large Problems

Reduced gradient methods for large problems are designed to solve large, sparse nonlinear problems with both a nonlinear objective function and nonlinear constraints. Most large problems of this nature are sparse and mostly linear (i.e., most elements of the Jacobian matrix are constant). Lasdon, et al. (1979) developed a large scale generalized reduced gradient (LSGRG) algorithm which attempts to exploit the nature of the Jacobian matrix by implementing features developed for large and sparse matrices.

In the LSGRG algorithm, following Saunders (1976), the non-basic variables are further partitioned into s superbasic variables, X_s, which are strictly between their bounds and n-m-s remaining non-basic variables, X_n, which are at one of their bounds, see Figure 2.7. The reduced gradient with respect to the non-basic variables, $\partial F/\partial X_n$, is used only to determine if one of these variables should be reduced from a bound to join the superbasic set. This decision is made after an optimization over the current set of superbasic variables is completed.

The reduced gradient with respect to the current superbasic variables, $\partial F/\partial X_s$, is then used to form a search direction, $\bar{d}$. Both conjugate gradient and variable metric methods have been used to determine $\bar{d}$. Then a one dimensional search is initiated to solve the problem

$$\text{minimize } F(X_{nb} + \alpha\bar{d}) \tag{2.69}$$

$$\alpha \geq 0$$

subject to the bounds on $X_{nb} = (X_s, X_n)$. The search direction, $\bar{d}$, is extended to include zero components for the non-basic variables at their bound. This minimization is done only approximately, and is accomplished by choos-

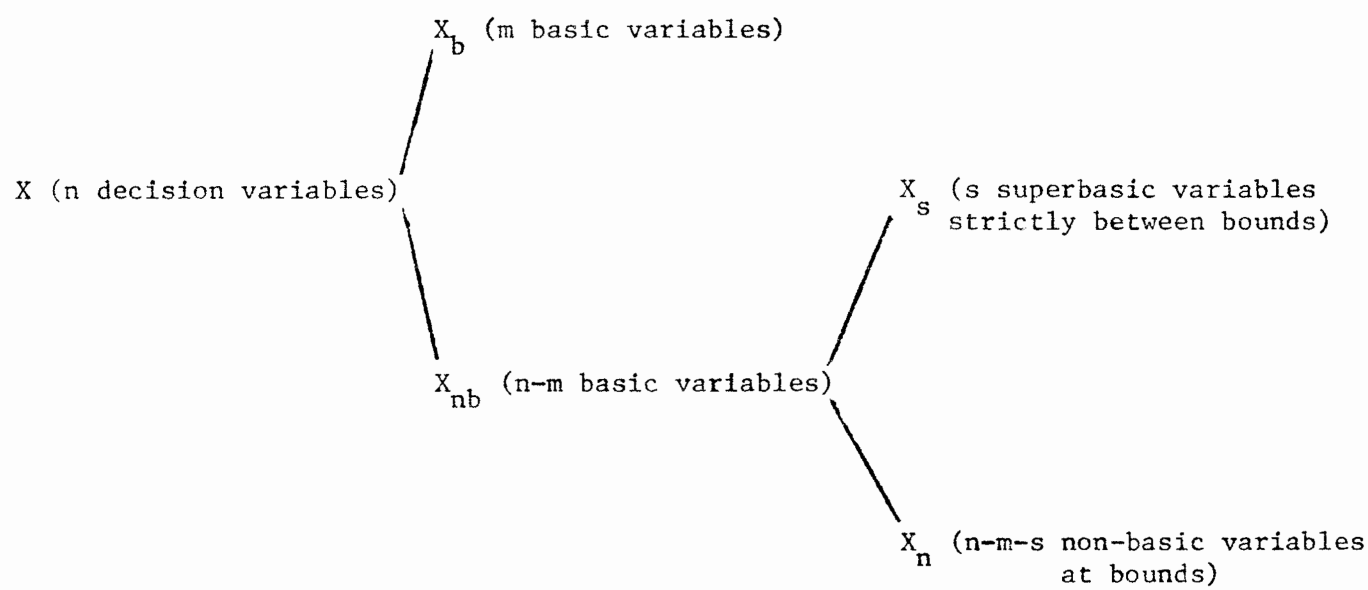


Figure 2.7 Variable classification used in LSGRG algorithm

ing a sequence of positive values $\{\alpha_1, \alpha_2, \alpha_3 \ldots\}$ for α. For each α_i, $F(X_{nb} + \alpha_i \bar{d})$ is evaluated. By Eq. 2.65, this is equal to

$$F(X_{nb} + \alpha_i \bar{d}) \quad = \quad f\,[X_b(X_{nb} + \alpha_i \bar{d}),\ X_{nb} + \alpha_i \bar{d}\ ] \tag{2.70}$$

This implies the determination of the basic variables which are dependent upon the non-basic variables. These satisfy the system of constraint equations

$$h(X_b,\ X_{nb} + \alpha_i \bar{d}) \quad = \quad 0 \tag{2.71}$$

where X_{nb}, α_i, and $\bar{d}$ are known and X_b is to be found. If X_b appears nonlinearly in any constraint then this system must be solved by an iterative procedure using a variant of Newton's method.

In the case of nonlinear constraints, the one dimensional search can terminate in three different ways. first, Newton's method may not converge. If this occurs on the first step, α_1 is reduced and the search is performed again; otherwise, the search is terminated. Second, if Newton's method converges, some basic variables may be in violation of their bounds. Then, a new value of α is determined such that at least one basic variable is at its bound and all others are within their bounds. If, at this new point, the objective is less than all previous points, the one dimensional search is terminated, a new set of basic variables is determined and a new reduced problem is initiated. Finally, the search may continue until an objective value is found which is larger than the previous one; then a quadratic is fitted to the three α_i values bracketing the minimum, and the search terminates with the lowest value for the reduced objective F. Ocanas and Mays (1980) present a more detailed description of the one dimensional search.

An important feature of this algorithm is its attempt to return to the constraint surface at each step in the one dimensional search, which differs from earlier strategies which involve linear searches on the tangent plane to the constraint surface prior to returning to the surface.

The Large Scale Generalized Reduced Gradient (LSGRG) Method LSGRG Algorithm

1. The LSGRG algorithm starts by evaluating the constraints and objective values at the initial point. This initial point is either supplied by the user, or determined in the code by setting the variables to their lower bound.

2. If any constraints are violated, a phase I procedure is entered in which the objective is the sum of absolute values of constraint violations. This provisional objective has a minimum of zero if there is a feasible solution to the problem. When a feasible solution is found, the actual objective takes over and the optimization procedure is continued.

3. An initial basis (basic matrix B) is chosen to be strictly triangular. The code allows the user to select from three options; (i) an all slack variable basis, (ii) all columns of the constraint matrix are considered as candidates for the initial basis, (iii) all columns of the constraint matrix corresponding to linear variables are considered for the initial basis.

4. Divide the current point, vector X, into basic (X_b), superbasic (X_s), and non-basic (X_n) variables. See Figure 2.7.

   (a) Use the m equality constraints to solve for the m basic variables in terms of the remaining n-m non-basic variables.

   (b) Define the superbasic variables as those non-basic variables strictly between their bounds.

5. Compute or update the basis inverse, B^{-1}, and compute the reduced gradient, F, using Eqs. 2.67 and 2.68, respectively.

6. Test for optimality. The current point, X, is considered optimal if either of two tets is satisfied. The first checks the Kuhn-Tucker condition. The second test checks if the fractional change in the objective is less than a small positive value for consecutive iterations. If X is optimal, STOP.

7. If X is not optimal, compute a search direction, d, and the tangent vector, V, corresponding to d, using the following equations:

$$d = -(R^T R)^{-1} \frac{\partial F}{\partial X_s} \tag{2.72}$$

where $R^T R$ is an approximation to $\dfrac{\partial^2 F}{\partial X_s^2}$

$$V = -B^{-1} \left[\frac{\partial h}{\partial X_s}\right] d \tag{2.73}$$

The tangent vector contains the directional derivatives of the basic variables in the direction d. After computing V, the largest step that can be taken in the direction d before any basic variable violates a bound is computed, assuming that all variables change linearly with derivates V.

8. Test for the largest step size. If it is too small, the current basis is termed degenerate, and a superbasic variable is selected to replace one of the basic variables threatening to violate a bound. Since the basis is changed, the new reduced gradient and the corresponding new search direction d are computed. If the basis is not degenerate, go on to (9).

9. Start the one dimensional search to find a minimum for the reduced problem, Eq. 2.69. This is accomplished by choosing a sequence

of positive values $\{\alpha_1, \alpha_2, . . .\}$ for , and evaluating the objective at every step.

10. If the search finds a new and better point, a new iteration is started, (go to 5). If the search fails to find a better point, two actions are attempted sequentially: (i) the search direction is set to the negative of the reduced gradient, ∇F, $d = -\partial F/\partial X_{nb}$, and (ii) some non-basic variable whose reduced gradient component indicates it wants to leave its bound is released. If both actions fail, the LSGRG code stops with appropriate error messages.

LSGRG Software Features

Some desirable features of the LSGRG computer implementation include:

Input Features

1. Ability to assign names to variables and constraints;

2. Ability to specify the type of each problem function and variable independently of their order;

3. Option for user provided subroutine for derivative evaluations, or system forward-differencing routine for the purpose;

4. Ability to modify some problem data, leaving the rest unchanged, enabling a sequence of different problems to be solved in one run;

5. Error checking and echo output of all input data;

6. Default values for all controllable program tolerances and parameters; and

7. Input decks in standard MPS linear programming format.

Output Features

1. Multiple print levels;

2. Tabular output formats;

3. Dump and restart capabilities; and

4. Availability to check user provided derivative computations.

Operational Features

1. dynamic storage allocation based on problem requirement;

2. Easy to use as part of a larger system; and

3. Ability to start from feasible or infeasible points, and to generate a sequence of improved feasible points, once feasible.

Successive Linear Programming (SLP) Method

Successive Linear Programming Algorithms--Successive linear programming (SLP) algorithms solve nonlinear optimization problems via a sequence of linear programs (LP's). SLP algorithms are particularly attractive for large, sparse nonlinear programs in which only some variables appear nonlinearly in the objective and/or in one or more constraints. In addition, there might be a subset of linear constraints. SLP has been extensively used in practice, mainly because: (1) SLP was developed by practitioners, (2) it is relatively easy to implement if an efficient, flexible LP code is available, and (3) it is capable of solving large problems.

In comparative studies, SLP has often performed poorly, but industrial users have reported successful performances. This disagreement is difficult to resolve. Algorithm performance is usually strongly dependent on the strategies used on the computer implementation, and on various parameter values used in the algorithm. Such information is missing or incomplete in most SLP references. The performance of SLP during the solution of the multi-period planning problems presented in Chapter 3 gives optimistic expectations for the solution of fairly large problems.

SLP algorithms solve nonlinear problems of the form

$$\text{minimize } g_o(x) + a_o y \tag{2.74}$$

$$\text{subject to } g(x) + A_1 y = b_1 \ (m_1 \text{ rows}) \tag{2.75}$$

$$A_2 x + A_3 y = b_2 (m_2 \text{ rows}) \tag{2.76}$$

$$\ell_1 \leq x \leq u_1;\ \ell_2 \leq y \leq u_2 \tag{2.77}$$

The n_1 "nonlinear" variables x may be involved in the objective, through the nonlinear function $g_o(x)$, or in the first m_1 constraints, via the vector of functions $g(x) = (g_1(x), \ldots, g_{m_1}(x))$. The n_2 dimensional vector y contains the "linear" variables, and the last m_2 constraints are linear. Both linear (y) and nonlinear (x) variables may have upper and lower bounds (Eq. 2.77), which may be $+\infty$ or $-\infty$, respectively. Any problem can be expressed in such a form (perhaps with $m_2 = 0$), by choosing y to include the slack or "logical" variables. All nonlinear functions $g_o, \ldots, g_{m_1}$ are assumed differentiable everywhere but are not required to be separable.

Given a base point $\bar{x}$, each function g_i is approximated by its Taylor series linearization about $\bar{x}$. Defining the Jacobian matrix of g(x) as

$$J(x) = (\partial g_i / \partial x_j) \tag{2.78}$$

$g(x)$ is replaced by $g(\bar{x}) + J(\bar{x})d$, and similarly for $g_o(x)$. This linearization is only accurate for d "small", so upper and lower bounds

$$-s \leq d \leq s \tag{2.79}$$

are imposed, where s is an n_1 vector with all positive components and is referred to as the "step bounds." The successor point $\bar{x} + d$ must also satisfy

$$\ell_1 \leq \bar{x} + d \leq u_1 \tag{2.80}$$

Combining Eqs. 2.79 and 2.80 with the Taylor series approximations and eliminating x using the relation

$$x = \bar{x} + d \tag{2.81}$$

leads to the linear program

$$\text{minimize } [\nabla g_o(\bar{x})\, d + a_o y] \tag{2.82}$$

$$\text{subject to } J(\bar{x})d + A_1 y = b_1 - g(\bar{x}) \tag{2.83}$$

$$A_w d + A_3 y = b_2 - A_2\bar{x} \tag{2.84}$$

$$\max\ (\ell_1 - \bar{x}, -s) \leq d \leq \min\ (u_1 - \bar{x}, s) \tag{2.85}$$

$$\ell_2 \leq y \leq u_2 \tag{2.86}$$

In the SLP algorithm, the LP (Eqs. 2.82 - 2.86) is solved, yielding a solution $(\bar{d}, \bar{y})$. If the candidate successor point $(\bar{x} + \bar{d}, \bar{y})$ meets certain criteria, it is accepted, and the step bounds s may be increased. Otherwise, s is reduced to s' and the above LP is solved. Palacios and Lasdon (1980) have an excellent discussion of the properties and insights into SLP algorithms.

<u>SLPR Computer Implementation Features</u>--A FORTRAN IV implementation of the SLPR algorithm was obtained from palacios and Lasdon (1980). The software features include:

<u>Input Features</u>

1. Ability to assign names to variables and constraints;

2. Ability to specify the type of each problem function and variable, independently of their order;

3. Ability to assign user provided upper and lower bounds for all variables;

4. Input decks in standard MPS linear programming format; and

5. Requirement of a user provided subroutine to compute the nonlinear functions (constraints and objective function) at a given point.

<u>Output Features</u>

1. Multiple print levels;

2. Execution statistics; and

3. Final iteration report.

<u>Operational Features</u>

1. Ability to use a simplex LP code as the main program subroutine;

2. Ability to store only non-zero elements of the linearized problem;

3. Dynamic storage allocation based on problem requirement; and

4. Ability to start from feasible or infeasible points.

Finding Initial Solutions

Generalized reduced gradient (GRG) and successive linear programming (SLP) methods require an initial solution to start out the optimization search. Both LSGRG and SLPR algorithms have the option of using an initial solution provided by the user or to start from an arbitrary solution, as determined by the lower bounds of the decision variables. If the initial solution happens to be an infeasible solution, a phase I optimization is initiated which minimizes an objective function consisting of the sum of infeasibilities until a feasible point is found. Once this is achieved, the actual objective function replaces the sum of infeasibilities and the actual optimization phase is initiated.

Past experiences have shown that for the problem of water allocation planning with water reuse included, starting from a "highly" infeasible point results in large amounts of computational time, most of which is consumed during the search for a feasible point. The size of the problem and the large proportion of equality constraints are the main reason for this behavior. Using the lower bounds as initial values for the decision variables results in a "highly" infeasible initial point.

Using an initial point provided by the user allows the inclusion of engineering judgment in selecting a good initial solution which may or may not be feasible. In either case, experience has shown that a good user provided initial point results in less computer time than initializing the algorithm from the lower bounds. This suggests that a procedure solution is well worth the effort.

The procedure developed considers the structure of the mathematical model developed in Sections 2.1 and 2.2 and makes use of standard optimization techniques such as network flow programming and linear programming in an effort to find an initial solution which, if not feasible to the nonlinear model, is a good initial point for the nonlinear programming algorithm.

The Out of Kilter-Linear Programming (OKA-LP) Procedure

The major steps in the OKA-LP procedure for finding initial solutions are as follows:

a). From the nonlinear model consider only the linear constraints. These include the users water demands, the sources water availability, the treatment plant capacities and the users and treatment plants mass balances.

b). From the nonlinear objective function, which consists of piping, pumping and treatment costs, represented by exponential functions, obtain linear functions which approximate the actual costs.

c). Solve the approximated linear objective function in (b) subject to the set of linear constraints in (a). Use an Out-of-Kilter algorithm, (OKA), to find a flow distribution which minimizes the approximated piping, pumping and treatment costs.

d). Use the flow distribution obtained from OKA to linaerize the nonlinear constraints of the nonlinear model (water alloation model). This set of contraints, the water quality constraints, consists of the users quality requirements, the maximum mass of pollutant discharges, and the water quality changes by the users and treatment plants. Recall that the nonlinearities involved in the water quality constraints result from the product of the flow variables and the concentration variables. Therefore, by fixing the flow variables to the values obtained from OKA, the nonlinear constraints become only linear functions of the concentration variables.

e). Select an arbitrary objective function which is to be minimized subject to the linear constraints resulting from (d). A linear programming (LP) algorithm is used to solve this problem. The use of an arbitrary objective function is justified since the concentration variables have no impact on the nonlinear cost function of the water allocation model and therefore, any feasible solution to the LP problem consists of the set of concentration values for the fixed flow distribution.

It is obvious that a feasible solution to the LP model in (e) is also a feasible solution to the nonlinear model since the flow distribution used to set up the LP model satisfies all the linear constraints of the nonlinear model, as shown in (a). However, this procedure does not guarantee that a feasible solution will be found every time, since the flow distribution obtained from the OKA run may represent an infeassible solution to the nonlinear problem when the quality requirements are included. It will produce, however, a point which is close to a feasible solution, from which the nonlinear codes LSGRG or SLPR can initiate the solution procedure.

To implement the OKA-LP procedure described above, a program which sets up the nonlinear model network into an OKA network was developed. This program is described in section 2.3.5.2. To solve the linear problem of the procedure, a program was developed which uses the flow distribution obtained from the Out-of-Kilter algorithm and sets up the constraint matrix and all the required information in standard MPS format. This program is described in section 2.3.5.3.

Program SETOKA

The purpose of this program is to take the actual network, as defined by the user, and to transform it to an Out-of-Kilter Algorithm (OKA) network (see Figure 2.8). An OKA network, also known as the minimum cost circulation problem, can be defined as a set of nodes and arcs in which a given amount of flow is required to be sent from a super source to a super sink in such a way that the total cost of flow is minimized. The constraints (flow conservation constraints and arc capacity constraints) under which this problem is solved are:

(1) The flow conservation constraint requires a total flow input into every node to be equal to its total flow output. In order to satisfy these conditions for the super source and the super sink, an arc is created from the sink to the source to carry the amount of flow equal to that sent from the source to the sink.

(2) The arc capacity constraints require that the flow through every arc be within its upper and lower bounds.

OKA problems require that the cost of flow in any arc be linearly proportional to the amount of flow in the arc. The objective function used during the OKA run attempts to represnt the costs of the nonlinear model. Since OKA requires the objective cost to be a linear function of flow through the arc, the approximation made is as follows:

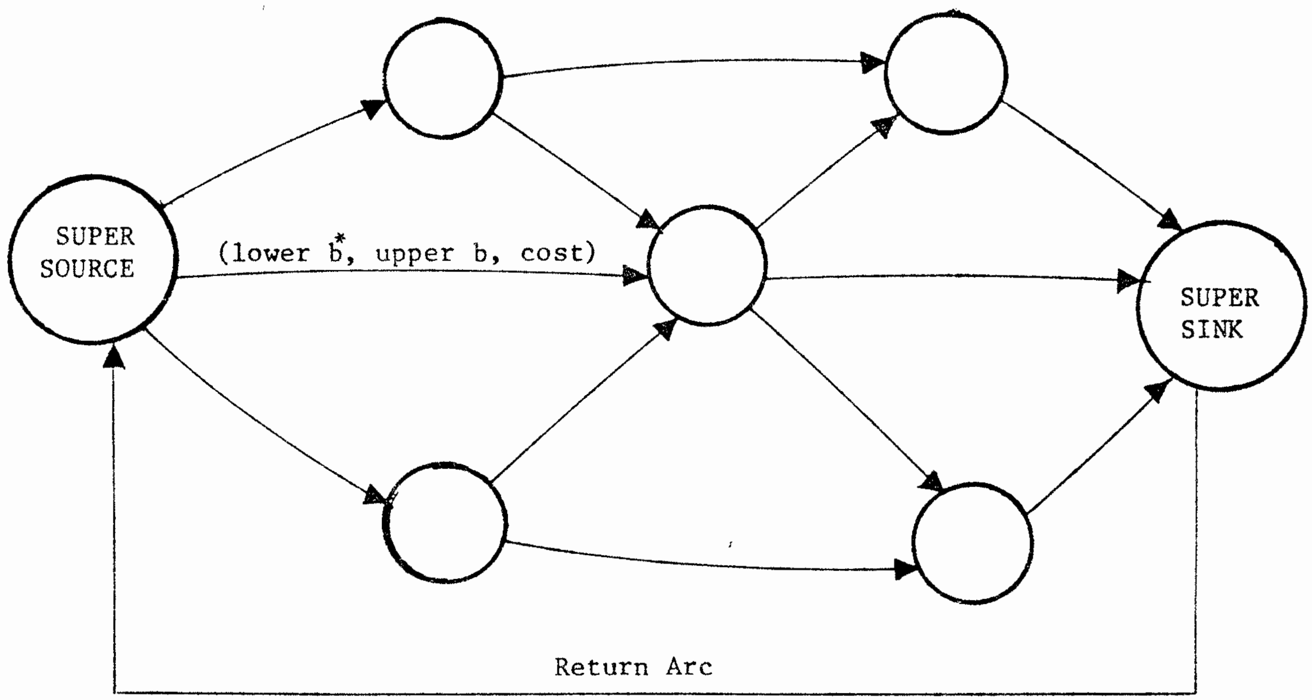
(i) From the cost equations, Cost = αQ^{β}, use α and β and a specified flow through the arc, Q^*.

(ii) Compute the cost associated with Q^*, Cost (Q^*).

(iii) Compute the slope of a line connecting the points (0,0) and (Q^*, Cost (Q^*)), see Figure 2.9).

(iv) The slope is the approximated linear function for the exponential cost.

This procedure is used for the equations representing piping, pumping and treatment costs. The total cost associated with each arc is the sum of



* b = bound

FIGURE 2.8 An Out of Kilter Algorithm Network

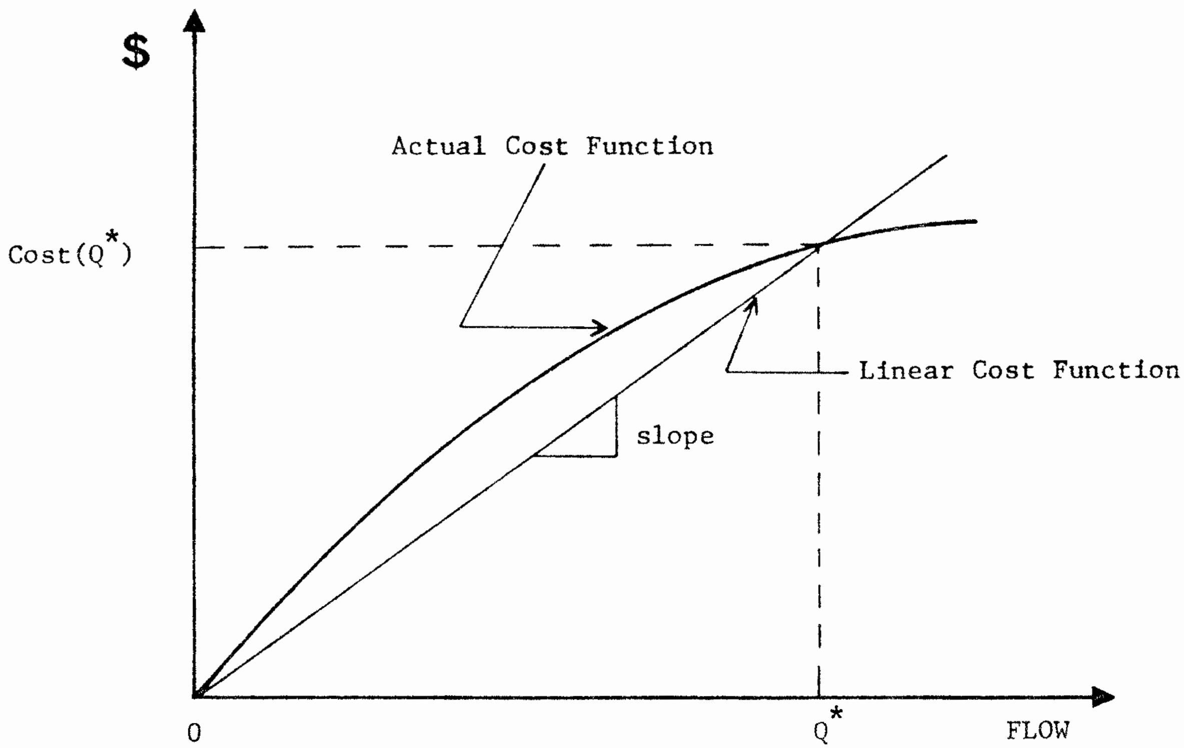


FIGURE 2.9 Linear Approximation of Exponential Cost Functions

the approximated piping and pumping linear costs. Since treatment costs are associated with flow through the plant, a fraction of the approximated linear treatment cost is included if the arc is leaving a node which provides treatment. Therefore, the total cost associated with an arc is

$$\text{Total Linear Cost} = (\text{Linear Piping \$}) + (\text{Linear Pumping \$}) + \theta(\text{Linear Treatment \$}) \tag{2.87}$$

where θ represents the fraction of treatment cost assigned to the arc. θ is usually defined as the inverse of the number of arcs leaving the node, i.e., θ = 1/Number of Arcs. In this case, the treatment cost is uniformly distributed among the arcs leaving the plant. Since the purpose of the water allocation models developed differs somewhat with that of OKA problems, several adjustments are required to assure that a feasible solution to the OKA satisfies the constraints of the water allocation models.

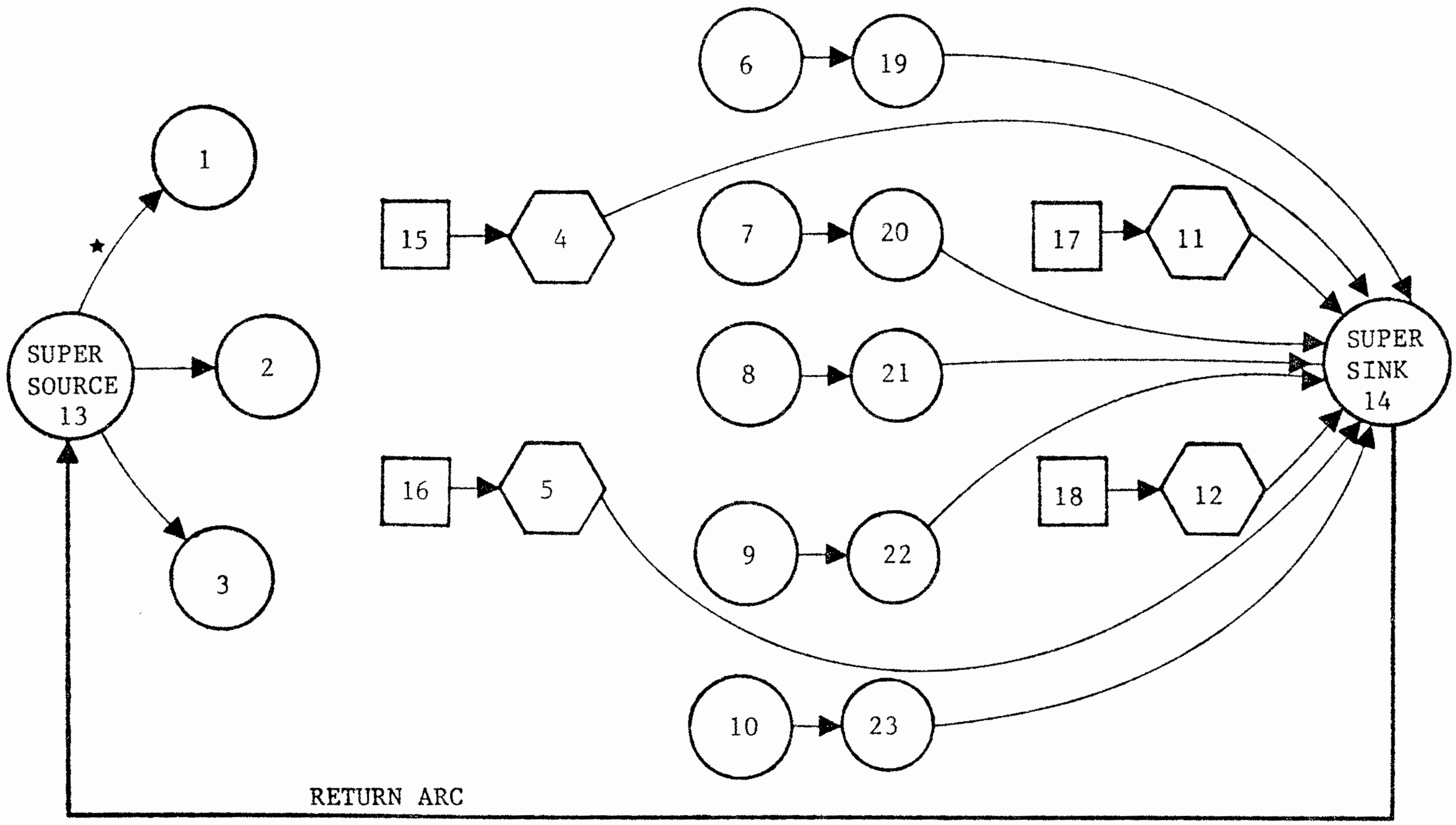
Recall that the constraints under which the water allocation problems have to be solved are (1) the users water demands, (2) the sources water availabilities, (3) the treatment plants capacities, (4) the mass balance for users and treatment plants, and (5) the water quality constraints.

Leaving out the sets of nonlinar constraints, i.e., the water quality constraint sets, it is possible to model the water allocation problem with an OKA network, given that some adjustments are made. The network shown in Figure 2.10 is used in the remaining discussion to illustrate the procedure.

(1) Users Water Demands: Create an artificial node for each user in the system, (nodes 19, 20, 21, 22 and 23) and create an artificial arc for each user connected to its respective artificial node. These arcs will have a lower bound equal to the user water demand, an arbitrary upper bound and a cost of zero. This forces the amount of water leaving the user to be greater than or equal to its water demands; therefore, the input flow to each user, which is equal to the output flow by means of the flow conservation constraints, will satisfy its water demands.

(2) Sources Water Availabilities: Create an artificial node (13), a super source node, which will supply the actual sources in the system. There will be an arc from this super source to each source with the following characteristics: (a) the lower bound will be zero, (b) the upper bound will be set equal to the maximum withdrawal allowed for the source and (c) a cost will be zero. This assures that the water leaving each source will not exceed its water availability by means of the flow conservation on the sources nodes and the arc capacity limitation on each arc entering the sources nodes.

(3) Treatment Plants Capacities: Create an artificial node for each treatment plant in the system, (nodes 15, 16, 17, 18) and create an artificial arc from the artificial node to its corresponding treatment plant node. This arc should have the following characteristics: (a) the lower bound is set at zero, (b) the upper bound is set at the treatment plant



★ Only artificial arcs are shown

FIGURE 2.10 Modified Network For The OKA Run capacity and (c) the cost is set at zero. This forces the input flow into the treatment plants to be less than or equal to their capacity by means of the arc capacity constraints in the OKA network.

(4) Mass Balance for Users and Treatment Plants: Create artificial arcs leaving each treatment plant node and each user artificial node and entering the super sink with the characteristics listed as follows: (a) lower bound equal to their water losses, (b) upper bound equal to the lower bound and (c) cost equal to zero. This is done because the flow continuity constraints of the OKA problem neglect any losses in the system.

Additional rules:

a). The acutual arcs entering a treatment plant should enter the corresponding artificial node. This is required to satisfy the capacity constraints.

b). The actual arcs leaving a user should leave their corresponding artificial node. This allows for the possibility of water reuse and recycling.

c). Create an arc from the super sink to the super source with a lower bound of zero, an upper bound of $+\infty$ and a cost of zero.

d). All the actual arcs, as defined by the user, should have a lower bound of zero, an upper bound of $+\infty$ and a cost which is the linear approximation to the exponential cost associated with the arc, as given by Eq. 2.74.

The user's manual and program listing for program SETOKA are presented in Ocanas and Mays (1980). Input requirements, output characteristics and a program listing are included.

Program SETLP

The purpose of this program is to develop a linear programming set up to find the values of the concentration variables for a given flow distribution.

Linear programming models require all the mathematical functions, constraints and objective function to be linear. A highly efficient code, called SIMPLEX, is available for solving such linear problems. This procedure moves from one feasible solution to another, at each step improving the value of the objective function. If no feasible solution is pre-determined, the code generates a feasible solution by replacing the actual objective by the sum of the absolute value of constraint violations, and minimizing this objective until it reaches a value of zero. This latter feature is exploited when searching for a feasible solution to the single period planning nonlinear model.

The nonlinear constraints in the nonlinear model consist of the water quality constraint sets, one set for each pollutant. A water quality constraint set consists of the following: (1) water quality requirements for each user, (2) maximum mass discharge of pollutant to the sources, (3) water quality changes by each user, (4) water quality changes by each water treatment plant, and (5) water quality changes by each wastewater treatment plant.

All the nonlinearities in such constraints are the result of the product of a flow variable times a concentration variable. If flow values are assigned "a priori," then the constraints become linear and only the concentration variables are considered as decision variables. It is clear that if the flow distribution assigned complies with all the linear constraints of the nonlinear model, then the set of concentration variables which satisfy the nonlinear constraints along with the flow variables already pre-assigned constitute a feasible solution to the nonlinear water allocation planning model.

The purpose of this program is: (1) to transform the nonlinear constraints into linear constraints by rearranging them using a given flow distribution; (2) to create an artificial linear objective function, and (3) to produce an MPS file which can be used as input for any commerical simplex code available. Since the main concern is to find a feasible solution, the objective function is meaningless since the simplex code will replace it by the sum of infeasibilities.

Program SETLP performed the following transformations:

(1) Water quality requirements for each user are expressed as

$$\sum_i \mathrm{FLOW}_{ij} \times \mathrm{Conc}_i \leq \sum_i \mathrm{Flow}_{ij} \times \mathrm{CSTD}_j \tag{2.88}$$

where Flow_{ij} is the flow from i to j (input), CSTD_j is the maximum concentration accepted by user j (input), and Conc_i is the concentration value leaving the node which sends its efluent to user j (decision variable).

(2) Maximum mass discharges of pollutant to soruces are expressed as

$$\sum_i \mathrm{FLOW}_{ik} \times \mathrm{Conc}_i \leq (\mathrm{Max.\ Dis.\ Acc.})_k \tag{2.89}$$

where Flow_{ik} is the flow from node i to node k (input), $(\mathrm{Max.\ Dis.\ Acc.})_i$ is the maximum mass discharge of pollutant accepted by node k (input), and Conc_i is the concentration value leaving the node which discharges its effluent to source k (decision variable).

(3) Water quality changes by each user are expressed as

$$\left(\frac{\sum_i \mathrm{FLOW}_{ij}}{\beta_j}\right) \mathrm{Conc}_j - \sum_i \mathrm{Flow}_{ij}\ \mathrm{Conc}_i = \sum_i \mathrm{Flow}_{ij}\ \Delta\mathrm{Conc}_j \tag{2.90}$$

where $Flow_{ij}$ is the flow from i to j (input), $\Delta Conc_j$ is the concentration increment produced by user j after one use (input), $_j$ is (1 - efficiency of treatment) given by user j (input), $Conc_j$ is the concentration value leaving user j (decision variable), and $Conc_i$ is the concentration value leaving the node which sends its effluent to user j (decision variable).

(4) Water quality changes by each water treatment plant are expressed as

$$\left(\frac{\sum_i FLOW_i}{\beta_\ell}\right) Conc_\ell = \sum_i Flow_{i\ell}\ Conc_i \tag{2.91}$$

where $Flow_{i\ell}$ is the flow from source i to water treatment plant ℓ (input), $Conc_i$ is the concentration leaving source i (input), β_ℓ is (1 - removal efficiency) of water treatment plant ℓ (input), and $Conc_\ell$ is the concentration leaving water treatment plant ℓ (decision variable).

(5) Water quality changes by each wastewater treatment plant are expressed as

$$\left(\frac{\sum_i FLOW_i}{\beta_\ell}\right) Conc_\ell - \sum_i Flow_{i\ell}\ Conc_i = 0 \tag{2.92}$$

where $Flow_{i\ell}$ is the flow from user i to wastewater treatment plant ℓ (input), β_ℓ is (1 - removal efficiency) of wastewater treatment plant ℓ (input), $Conc_\ell$ is the concentration leaving wastewater treatment plant ℓ (decision variable), and $Conc_i$ is the concentration leaving user i which sends its effluent to wastewater treatment plant ℓ (decision variable).

The user's manual for program SETLP which includes the input requirements, output characteristics and the program listing is presented in Ocanas and Mays (1980).

The interaction among programs SETOKA, SETLP, and the Out-of-Kilter Algorithm (OKA) and Linear Programming (LP) codes when searching for initial solutions are shown in Figure 2.11. As observed in Figure 2.11, the initial solution will be composed of the flow distribution obtained from the OKA run and the concentration values associated with such flow distribution as given by the LP solution.

2.4 SOFTWARE DEVELOPMENT

This section provides instructions for solving problems and presents a small example problem to show how the computer model handles the numbering of the decision variables during the solution of a problem. The notation developed in this Chapter is for the description of the single period and multi-period models, respectively, and consists of various sets of decision variables, as they were classified in accordance with their characteristics. For instance, there is a set consisting of flows from sources to users in a given planning period, XS_{ijp}, and there is another set consisting of the concentration of the water leaving a water treatment plant,

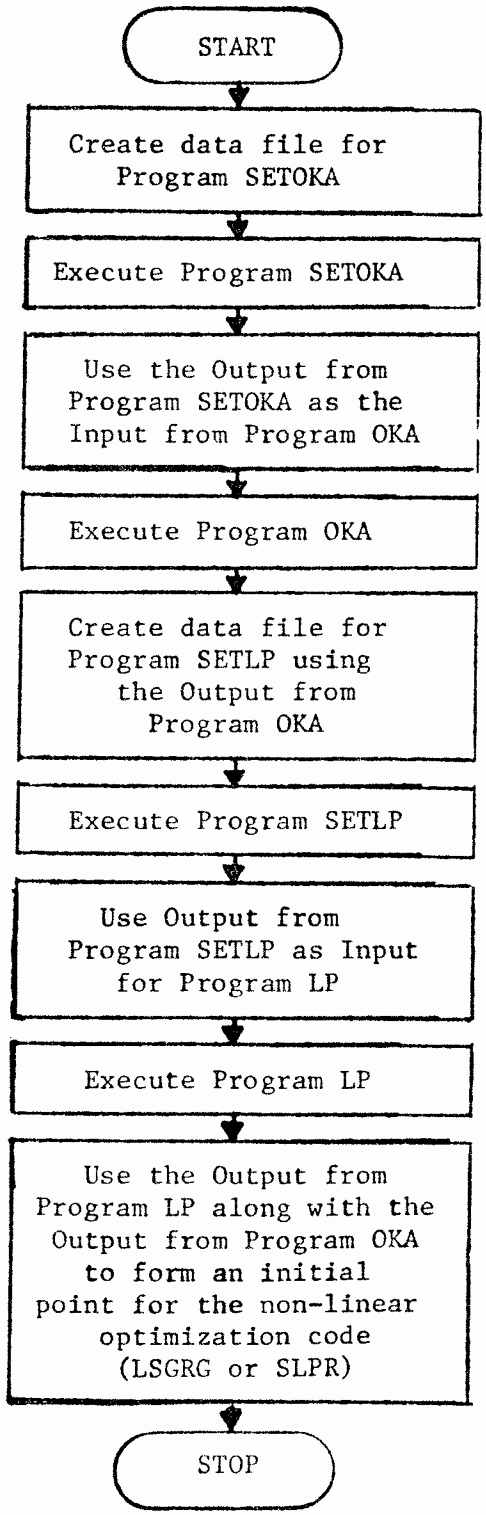


Figure 2.11 OKA-LP Procedure for Finding Initial Solutions

$CT(P_n)_p$, and so on. In the solution procedure, the optimization algorithm only deals with a unique vector of decision variables. This requires an organized procedure to transform the multiple set form of decision variables to a unique vector of decision variables used in the optimization algorithm.

This organized procedure is able to handle any possible region consisting of sources, sinks, users, water treatment plants and wastewater treatment plants. The procedure first considers all the elements in the region as nodes in a network where water flows through. It numbers all the nodes sequentially, starting with the sources and sinks, then the water treatment plants, users, and wastewater treatment plants.

Second, the procedure develops and numbers the whole set of possible links (from now on referred to as arcs) among the elements.

From this spectrum of arcs, the model user is able to select those arcs economically or technically feasible, including existent and potential ones.

Finally, the decision variables (flows, concentrations and capacities) are defined and numbered considering only those arcs which have been defined feasible by the planner, therefore creating the decision vector which the optimization algorithm requires. For the single period model, the decision vector consists of the flows through feasible arcs, and the concentration of each node.

For the multi-period model, the decision vector follows a similar structure as the single period, however, capacity variables are included and several planning periods are considered.

Computer Program for Single Period Planning Model

The computer programs to solve the single period model consists of four different components interacting to obtain the optimal water allocation for a region. Listings of each program and detailed user's manuals are given in Ocanas and Mays (1980).

Computer Programs (Figure 2.12)

a). Program GENARC. This program defines the arc numbering used to help generate the input for programs MPSGENS. The input for program GENARC consists of the number of sources, sinks, users, water treatment plants, wastewater treatment plants and pollutants.

b). Program MPSGENS. This program generates an output consisting of the input information required by program LSGRG.

c). Program LSGRG. This is the program which performs the actual optimization of the system. In general, this program requires the use of the SPECS FILE, the MPS file and subroutine FCOMP. This program produces

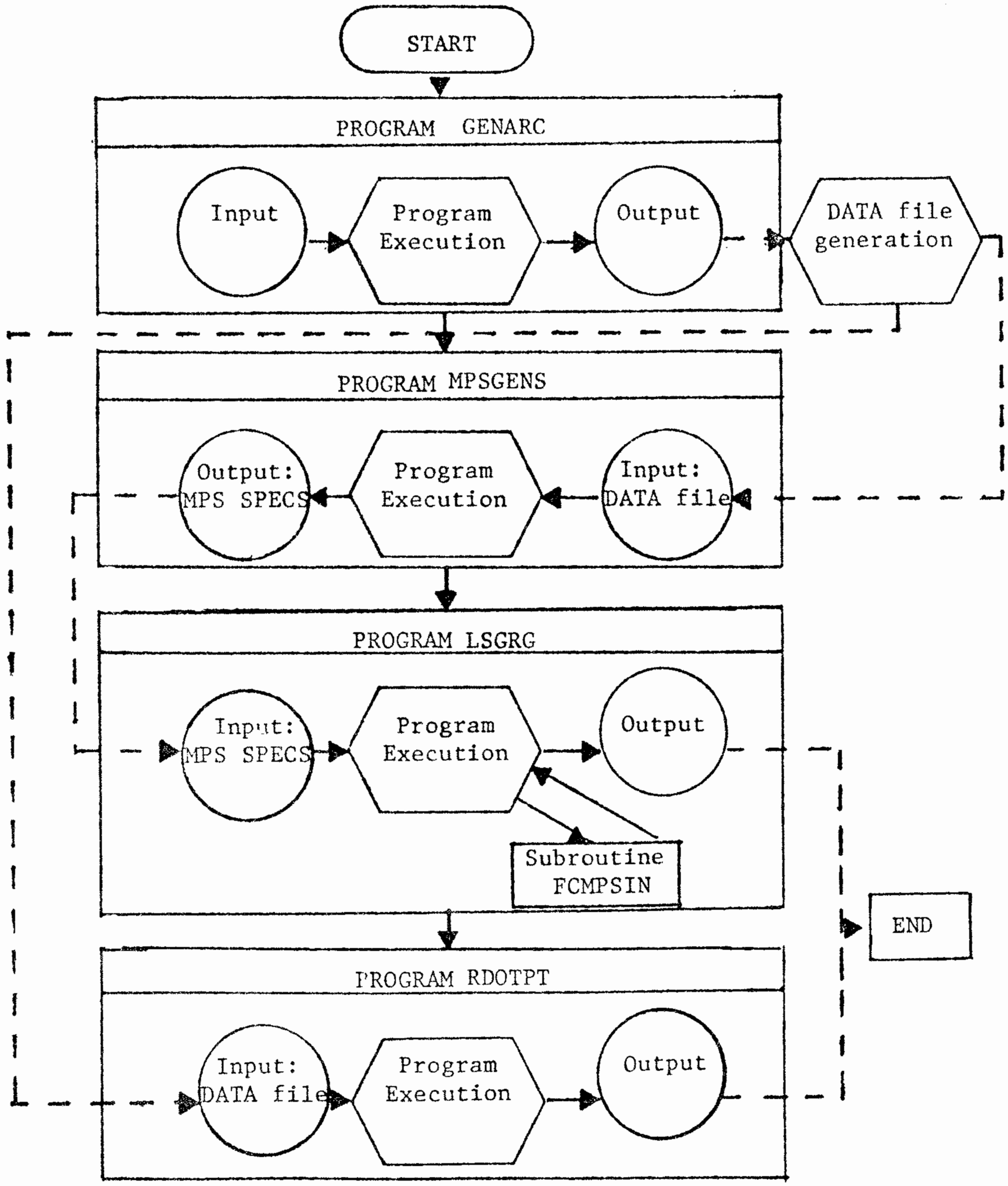


Figure 2.12 Computer Program Components Interaction for Single Period Planning

an output containing the status of the solution found (optimal, infeasible, unbounded, etc.) and the value of each decision variable.

d). Program RDOTPT. This program produces a table to aid in the interpretation of the LSGRG output. By using the LSGRG output the value of each decision variable is known, and by using the output from RDOTPT the meaning of each decision variable is determined.

Solution Procedure (Figure 2.12)

a). Run program GENARC.

b). Use the output from program GENARC to help generate a DATA file for program MPSGENS.

c). Run program MPSGENS.

d). Run LSGRG.

e). Run program RDOTPT.

f). Use the output from program LSGRG along with the output from program RDOTPT to find the optimal flows through each arc, and the concentrations leaving each node.

Computer Program for Multi-Period Planning Model

The decision variables of a multi-period planning problem can be classified into flows through arcs, capacities of water and wastewater treatment plants, and concentrations leaving each node during every planning period. Because the optimization code requires a single vector of decision variables, they must be numbered from one to the total number of variables, as for the single period model. The computer model developed consists of four different components interacting to obtain the optimal planning for a region. Listings of each program and detailed user's manuals are given in Ocanas and Mays (1980).

Computer Programs (Figure 2.13)

a). Program MULGARC. This program produces the arc numbering of all the arcs allowed by the model during each planning period. It also numbers the capacity and concentration variables. The output of program MULGARC is used during the creation of the input file required by program MPSGENM. The input for program MULGARC consists of the number of planning periods and pollutants, and, for every period, the number of sources, sinks, users, water and wastewater treatment plants.

b). Program MPSGENM. This program generates an output consisting of the information required by program LSGRG.

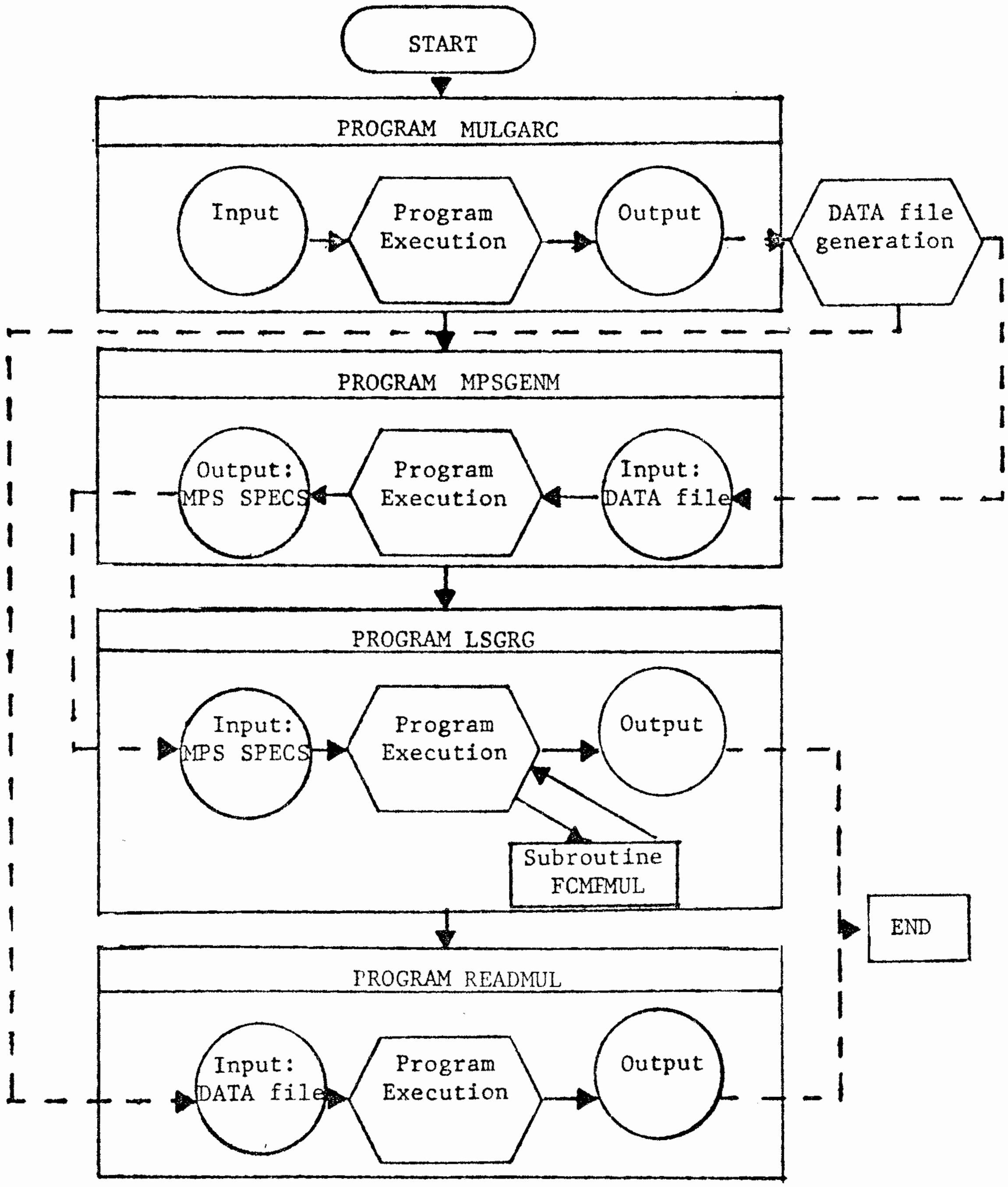


Figure 2.13 Computer Program Components Interaction for Multi-Period Planning

c). Program LSGRG. This program performs the optimization phase. The input requirements for LSGRG consist of a SPECS file, an MPS file and a subroutine FCOMP. This program produces an output containing the status of the final solution (optimal, infeasible, unbounded, etc.), and the value of each decision variable.

d). Program READMUL. This program produces an output which aids the user in the interpretation of the LSGRG output. By using the LSGRG output, the value of each decison variable is known, and by using the READMUL output, the meaining of each decision variable is determined.

Solution Procedure (Figure 2.13)

a). Run program MULGARC.

b). Use the output from program MULGARC while creating a DATA file for program MPSGENM.

c). Run program MPSGENM.

d). Run program LSGRG, using the MPSGENM output and subroutine FCMPMUL.

e). Run program READMUL.

f). Use the output from program LSGRG along with the output from program READMUL to find the optimal planning for the region. The above procedure is further illustrated in Figure 2.13.

Computer Program for Finding Initial Solutions

Non-linear programming optimization codes, LSGRG and SLPR, are very dependent upon the initial point used during the execution of the program. The selection of a bad initial point may cause the algorithm to fail in its search for an optimal point or to consume large amounts of time before reaching the optimum. Section 2.3 describes a procedure to find initial points for water reuse planning problems. It exploits the structure of the model and uses standard optimization techniques in determining such point. This appendix provides a description of the computer programs developed to aid the user in the task of searching for initial points. It involves the interaction of four computer programs, SETOKA, SETLP, OKA and LP, with the two latter programs being standard network flow and linear programmming codes. Figure 2.11 shows the interaction of the four programs.

Program SETOKA

The program transforms the actual network structure, as defined by the user, to an Out-of-Kilter Algorithm (OKA) network, which can then be solved by a standard OKA optimization code. The program requires the creation of various artificial nodes and arcs to carry out the transformation.

The artificial nodes numbering should follow these rules:

(1) Create two artificial nodes, a super source and a super sink node, and assign the numbers N+1 and N+2, respectively, where N is the total number of nodes in the actual network (sources, users, etc.).

(2) Create NTP artificial nodes, where NTP is the number of water and wastewater treatment plants, and number them N+3, N+4, . . . , N+2+NTP.

(3) Create NU artificial nodes, where NU is the number of water users, and number them starting from N+2+NTP.

(4) The total number of artificial nodes must be NTP+NU+2.

The program also requires the creation of some artificial arcs as follows:

(1) Each actual source must receive an artificial arc from the super source artificial node.

(2) Each actual treatment plant must receive an artificial arc from its respective artificial node.

(3) Each actual water user must send an artificial arc to its respective artificial node.

<u>How to Find a Feasible Flow Distribution</u>

To find a feasible flow distribution (a flow distribution that satisfies all linear constraints of the single period planning model), programs SETOKA and OKA should be used. The following procedure is proposed.

(1) Create a data file for program SETOKA.

(2) Run program SETOKA to obtain the input file for program OKA.

(3) Run program OKA to obtain an optimal flow distribution which satisfies all the linear constraints of the single period model and uses a linear approximation of the objective function.

The output obtained using the above procedure provides a flow distribution which satisfies the linear constraints of the single period planning model; namely, the user demands, plants capacity, sources water availability, and mass balance constraints. The flow distribution is an optimal solution to the approximated linear objective function. This flow distribution is used in creating the input file for program SETLP.

<u>Program SETLP</u>--The purpose of this program is to develop a linear programming set up to determine the concentration values associated with a given flow distribution. The nonlinear constraints in the single period planning

model consist of the water quality constraint sets, one set for each pollutant. A water quality constraint set consists of the following: (1) water quality requirements for each user, (2) maximum mass discharge of the pollutant to the sources, (3) water quality changes by each user, (4) water quality changes by each water treatment plant, and (5) water quality changes by each wastewater treatment plant. All the nonlinearities in such constraints are the result of the product of a flow variable times a concentration variable. By fixing the flow variables, the constraints become linear.

Program SETLP transforms the nonlinear constraints to linear constraints using the flow distribution obtained from program OKA, and creates an MPS file which can be used as input for any commercial linear programming code.

Program SETLP Input Requirements--The input for program SETLP consists of the following sets:

(1) A set which defines the number of sources, water treatment plants, users, wastewater treatment plants, sinks, pollutants, and existent or valid arcs (as defined by the user).

(2) A set which defines the existent or valid arcs in the network, (i.e., where water can flow through). The arcs are defined in terms of the node they connect. The set also includes the flow through each of the arcs as given by the output of program OKA.

(3) A set which contains information about the sources; the water quality, (concentration of each pollutant used for the quality criteria) and the maximum discharge of mass of pollutant that can be accepted.

(4) A set containing information about the users; the quality requirements, (max. concentration of each pollutant they can accept); the quality degradation, (the concentration increment of each pollutant that is produced by using the water); the treatment they provide their effluents prior to discharge (in terms of the penetration fraction of each pollutant. It must be recalled that penetration fraction + removal fraction = 1.); and the water demands.

(5) A set containing the information about water and wastewater treatment plants, by defining the treatment efficiency; by giving the fraction of each pollutant which is not removed during treatment.

How to Find an Initial Solution--To find an initial solution to the single period water reuse planning model, programs SETLP and LP use the feasible flow distribution obtained by program OKA and solves for the concentration variables. If the resulting concentration variables satisfy all the quality constraints included in the LP formulation, then the flow distribution along with the concentration variables constitute a feasible initial point. However, if the LP fails to find a feasible set of concentration variables, then the initial solution of the flow distribution and the resulting concentration variables represent a "good" infeasible initial point.

To run programs SETLP and LP, the following procedure is suggested.

(1) Create a data file for program SETLP.

(2) Run program SETLP to obtain the input file for program LP.

(3) Run program LP to obtain the concentration values for the flow distribution from program OKA.

It should be noted that program LSGRG can be used to solve the LP problem, since it incorporates an LP phase. The output from program SETLP develops an MPS file which can be used with any commercial LP code plus a SPECS file that goes along with program LSGRG.

The Output--The output obtained using the above procedure provides the concentration variables for the flow distribution from program OKA. The initial solution which must be specified for the single period planning model is the union of the flow distribution from OKA and the concentration variables from LP.

SECTION 3

APPLICATION OF THE LSNLP MODELS: THE SAN ANTONIO CASE

3.1 DESCRIPTION OF THE APPLICATION REGION

The study area included Bexar County, Texas and part of the San Antonio River Basin, consisting of most of Leon Creek, Salado Creek and the upper San Antonio River water shed (Figure 3.1). The City of San Antonio is drained by these three major watersheds of the San Antonio River Basin. Flow in the three streams is normally small, with the seven day - two year low flows ranging from 0 cfs in Leon Creek to 11 cfs in San Antonio River. However, several miles south of San Antonio, where the Medina River empties into San Antonio River, low flows are much greater. The seven day - two year low flow of the San Antonio River just below the Confluence of the Medina River is 53 cfs. Cost functions for the application are given in Table 3.1. The system elements (users, sources and treatment plants) considered in the application are shown schematically in Figure 3.1 and additional details are listed in Table 3.2.

The City of San Antonio is experiencing a tremendous population growth, primarily in the northern portion of Bexar County. In 1975 the City of San Antonio had a population of 777,000 and the projected population for the year 2000 is 1.1 million people. Currently the Edwards Aquifer is the only source of drinking water for the City of San Antonio. The average annual yield of the aquifer without withdrawing significant quantities of water from storage is equal to the annual rate of recharge (500,000 ac-ft/year), assuming that all flow to the springs is intercepted and the springs are allowed to dry up. For application of the models it is assumed that the maximum discharge from the Edwards Aquifer that can be used by the City of San Antonio is 285,000 ac-ft/year, in order to prevent depletion of the aquifer.

The Texas Water Plan (1968) states that surface water will be required to meet further water demands of the San Antonio area. According to the Plan, San Antonio's initial supplemental surface water supply would come from Canyon Lake, which is located in the Guadalupe River Basin approximately 35 miles northeast of the city (Figure 3.1). Other prospective sources of surface water supply are the Applewhite Reservoir, which would be located on the Medina River approximately 11 miles southwest of San Antonio, and the Cibolo Reservoir, which would be located on Cibolo Creek about 32 miles southeast of San Antonio.

The City of San Antonio is serviced by three major wastewater treatment facilities, the Rilling Road plant, the Salado Creek plant and the Leon Creek plant (Figure 3.1, Table 3.2). Seven small package or temporary

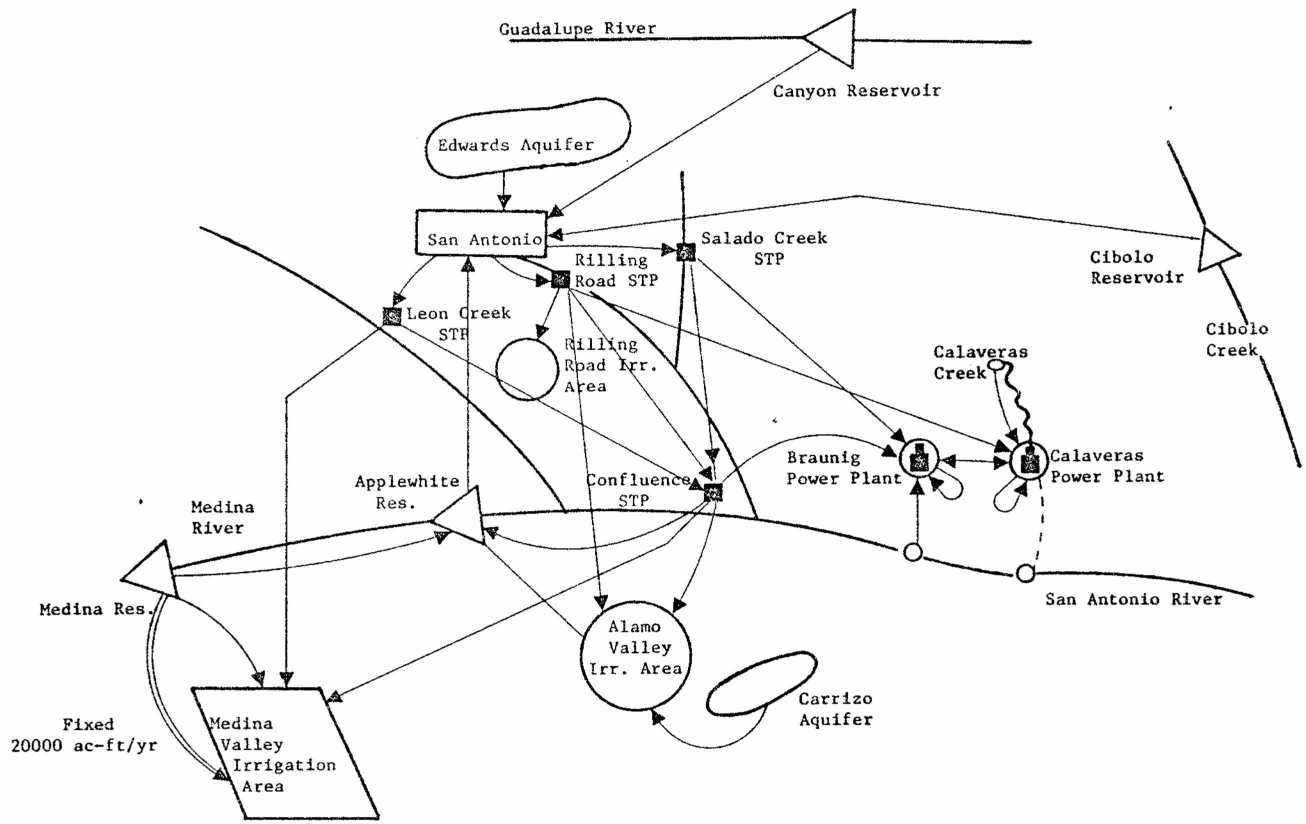


FIGURE 3.1 Schematic Representation of the System Elements Components

TABLE 3.1 Cost Functions

Description	Model Cost Function	Cost Functions for Application (10^6\$)	Source
1. Freshwater operation and/or capital	αQ^{β}	α and β values in Table 2	Texas Department of Water Resources
2. Pipelines			
a. Operation and maintenance	$\sum_{w \in W_p} \alpha_w Q_w^{\beta_w}$	4.56×10^{-3} distance (mi) $Q^{0.495}$ (refer to Table 2)	*McConagha and Converse* [1973]
b. Construction	$\sum_{w \in NW_p} \alpha_w Q_w^{\beta_w}$	$80Q^{0.461}$	*U.S. EPA* [1978*d*]
3. Pumping			
a. Operation and maintenance	$\sum_{z \in Z_p} \alpha_z Q_z^{\beta_z}$	$8.39 \times 10^{-4} H^{0.482} Q^{0.482}$	*U.S. Army Corps of Engineers* [1973]
b. Recycling	$\sum_{z \in Z_p} \alpha_z Q_z^{\beta_z}$	$0.002Q^{0.737}$	*U.S. Army Corps of Engineers* [1973]
4. Water treatment plants			
a. Operation and maintenance	$\sum_{l \in WT_p} \alpha_l [\sum_i XST_{ilp} + \sum_k XGT_{klp}]^{\beta_l}$	$0.0150Q^{0.798}$	*U.S. EPA* [1979]
b. New plant construction	$\sum_{l \in WT_p} \theta_l [CAP_{lp}]^{\phi_l}$	$1.50Q^{0.810}$	*U.S. EPA* [1979]
c. Expansion of existing plants	$\sum_{l \in WT_p} \Omega_l [CAP_{l,p+1} - CAP_{l,p}]^{\gamma_l}$	$1.20Q^{0.810}$	*U.S. EPA* [1979]
5. Wastewater treatment plants			
a. Operation and maintenance	$\sum_{l \in WWT_p} \alpha_l [\sum_j XTR_{jlp}]^{\beta_l}$	$0.0825Q^{0.960}$	*U.S. EPA* [1978*b*]
b. New plant construction	$\sum_{l \in WWT_p} \theta_l [CAP_{lp}]^{\phi_l}$	$2.88Q^{0.99}$	*U.S. EPA* [1978*a*]
c. Expansion of existing plants	$\sum_{l \in WWT_p} \Omega_l [CAP_{l,p+1} - CAP_{l,p}]^{\gamma_l}$	$2.25\Delta Q^{0.890}$	*U.S. EPA* [1978*a*]

All flow rates Q and ΔQ are in million gallons per day.

TABLE 3.2 System Elements (Existing and Proposed)

Freshwater Sources	Maximum Yield, ac ft/yr	Water Lost (Excluding Treatment and Transportation), \$/ac ft	Cost, 10^6 \$/yr, $\alpha Q_\alpha{}^\beta$, $\beta = 1.0$
Canyon Lake	30,000	35	0.0392
Cibolo Reservoir (proposed)	20,000	95	0.1064
Applewhite Reservoir (proposed)	14,900	75	0.0840
Medina Reservoir	40,000 (20,000 to irrigation)	10	0.0112
San Antonio River	12,000	0	0.0
Calaveras Creek	27,000	0	0.0
Carrizo Aquifer	15,000	10	0.0112
Edwards Aquifer	285,000	15	0.0168

User	Water Demand, ac ft/yr			Water Losses, ac ft/yr			Maximum BOD Acceptable, mg/l	BOD Increment, mg/l
	1980	2000	2030	1980	2000	2030		
City of San Antonio	240,000	363,000	575,000	82,000	174,000	201,000	5	260
Calaveras power plant	1,500,000	1,800,000	1,800,000	18,000	23,000	23,000	15	0
Braunig power plant	600,000	600,000	600,000	7,000	7,000	7,000	10	0
Medina Valley irrigation area	20,000	20,000	20,000	20,000	20,000	20,000	10	
Alamo Valley irrigation area	24,000	24,000	24,000	24,000	24,000	24,000	10	
Rilling Road irrigation area	4,000	4,000	4,000	4,000	4,000	4,000		

Wastewater Treatment Plants	Capacity, MGD	BOD Removal Efficiency, %
Leon Creek	24	97.2
Rilling Road	94	98.1
Salado Creek	24	97.2
Confluence (proposed)	154	98.1 (proposed)

wastewater treatment facilities are also operated by the city throughout newly developed areas until the waste flows are large enough to justify their incorporation into the regional facilities.

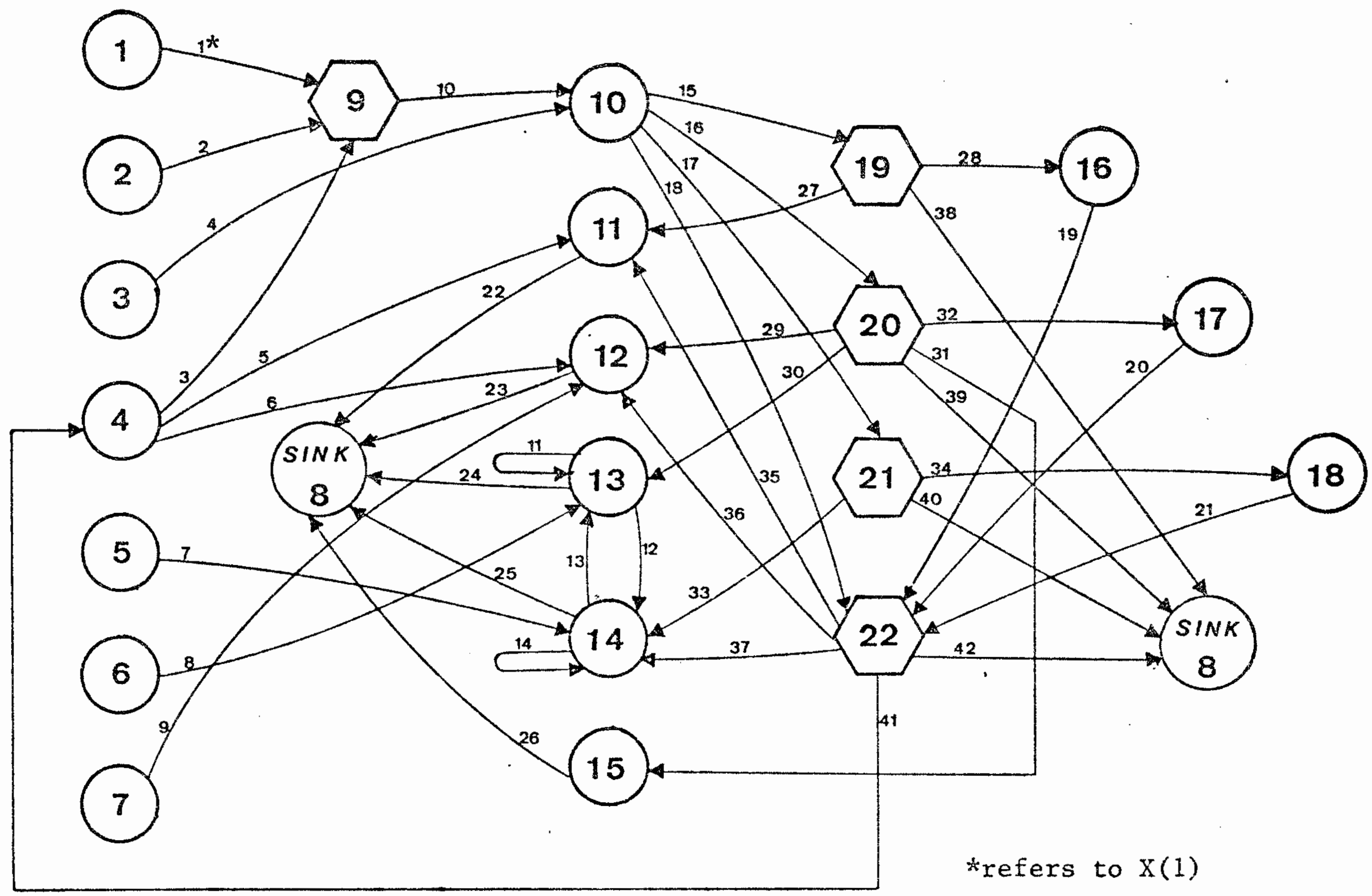
The City of San Antonio is considering several alternatives regarding the improvement of the treatment system. These include: (1) abandonment of the existing Rilling Road Treatment Plant, (2) expansion and improvement of the Leon Creek and Salado Creek plants, (3) construction of a new wastewater treatment facility (referred to as the Confluence Treatment Plant) near the confluence of the San Antonio and Medina Rivers, and (4) the construction of sewage transfer lines from the existing Rilling Road site and the Leon Creek and Salado Creek treatment plants to the new Confluence Treatment Plant.

The proposed confluence wastewater treatment plant would provide preliminary treatment, primary treatment, two-stage biological activated sludge treatment, filtration and disinfection (U.S. EPA, 1978c). Wastewater emanating from the present Rilling Road service area (approximately 83 MGD) would be treated through the first stage of activated sludge facilities. At that point, following the intermediate clarifers, this secondary treated wastewater would be combined with the secondary treated wastewater from the Leon Creek and Salado Creek plants and the total flow of approximately 154 MGD would be further treated to a tertiary level beginning with the second stage activated sludge process. The final treated wastewater effluent would be discharged to the San Antonio River approximately 1200 feet upstream of the confluence with the Medina River, or be used for some direct or indirect reuse alternatives within the region (U.S. EPA, 1978c).

3.2 MODEL NETWORK STRUCTURE

In order to model the water supply system, including the proposed facilities, all of the elements are numbered continuously, starting with the sources, followed by the water treatment plants, users, and wastewater treatment plants (Figure 3.2). Table 3.3 presents the node numbering system. Since the mathematical model is not suited for the direct interaction of sources (i.e., flow from one source to another), the Medina Reservoir and the Applewhite Reservoir are combined and considered as one source with a water yield equal to the sum of the two yields. This poses no problem in the analysis, as there is no cost associated with the transfer of water from the Medina Reservoir to the Applewhite Reservoir by way of the Medina River and the water quality is the same for both sources. The cost associated with the conveyance of water from the Medina Reservoir to the Medina Valley Irrigation Zone is assigned to an arc leaving Applewhite (which includes Medina Reservoir) and directed to the irrigation zone. The arc includes the freshwater cost of Medina Reservoir even though the arc is leaving Applewhite Reservoir, represented by node 4.

The use of a dummy sink allows the model more flexibility since the treated wastewater may be discharged if its reuse is not economical. The nodes which have an arc to the dummy sink (node 8) are those nodes representing the wastewater treatment plants (nodes 19, 20, 21 and 22) and the nodes representing users (nodes 11, 12, 13 and 14), except the City of San Antonio, because it is not allowed to discharge municipal wastewater without treatment.



*refers to X(1)

Figure 3.2 Feasible Arcs for the San Antonio Region and the Flow Variables Numbering as used in Cases 1, 2 and 3

TABLE 3.3 Node Numbering and Cost Data for Pipeline Construction and Operation and Maintenance (O and M) for Pipeliner and Pumping

						Pipeline Construction		Pumping O and M Cost Function, Cost, 10^6 \$/yr, αQ^β			
				Distance,	Pipeline O and M Cost Function αQ^β, $\beta = 0.495$	Initial Cost, 10^6 \$, αQ^β, $\beta = 0.461$	Cost, 10^6 \$/yr, αQ^β, $\beta = 0.461$			Elevation Difference,	
Node	From	Node	To	mi	α	α	α	α	β	ft	H, ft
19	Leon Creek WWTP	11	Medina irrigation area	25	0.1141	10.56	0.7498	0.0205	0.482	381	762
22	Confluence WWTP	11	Medina irrigation area	28	0.1278	11.83	0.8397	0.0221	0.482	444	888
22	Confluence WWTP	12	Alamo irrigation area	3	0.0137	1.27	0.0900				
22	Confluence WWTP	4	Applewhite Reservoir	6	0.0274	2.53	0.1799	0.0088	0.482	66	132
18	Leon Creek WWTP	22	Confluence WWTP	9	0.0411	3.80	0.2699				
20	Rilling Road WWTP	22	Confluence WWTP	6	0.0274	2.53	0.1799				
21	Salado Cr. WWTP	22	Confluence WWTP	6	0.0274	2.53	0.1799				
22	Confluence WWTP	14	Braunig power plant	2	0.0091	.84	0.060				
10	San Antonio City	22	Confluence WWTP	12	0.0548	5.07	0.3599				
4	Applewhite Reservoir*	10	San Antonio City	11	0.0502	4.65	0.3299				
4	Applewhite Reservoir	12	Alamo irrigation area	7	0.0319						
4	Medina Reservoir	11	Medina irrigation area	22	0.1004						
10	San Antonio City	19	Leon Creek WWTP	6	0.0274						
10	San Antonio City	21	Salado Creek WWTP	8	0.0365						
10	San Antonio City	20	Rilling Road WWTP	7	0.0319						
1	Canyon Lake	10	San Antonio City	36	0.1643	15.21	1.0797				
2	Cibolo Reservoir	10	San Antonio City	32	0.1460	13.52	0.9597				
7	Carrizo Aquifer	12	Alamo irrigation area	0	0.00			0.0217	0.482	425	850
20	Rilling Road WWTP	13	Calaveras power plant*	1.5	0.0068			0.0105	0.482	95	190
14	Braunig power plant	13	Calaveras power plant	3	0.0137	1.27	0.0900				
5	San Antonio River	14	Braunig power plant	0.5	0.0023			0.0111	0.482	107	214
14	Braunig power plant	14	Braunig power plant	0				0.0020	0.737		
13	Calaveras power plant	13	Calaveras power plant	0				0.0020	0.737		
6	Calaveras Creek	13	Calaveras power plant	0							
20	Rilling Road WWTP	15	Rilling Road irrigation area	0							
3	Edwards Aquifer	10	San Antonio City	0				0.0245	0.482	550	1100
4	Medina Reservoir	4	Applewhite Reservoir	0							
13	Calaveras power plant	14	Braunig power plant	3	0.0137	1.27	0.0900	0.0052	0.482	22	44
21	Salado Creek WWTP	14	Braunig power plant	8	0.0365	3.38	0.2399	0.0020	0.482	3	6

WWTP is wastewater treatment plant.
* Medina Reservoir and Applewhite Reservoir are considered as one source (node 4) with a water yield equal to the sum of the two yields.

The use of a dummy sink also allows the use of a penalty cost for waste discharges assigned to the arc entering the sink. Penalty costs were not included for this application, so that any recycling or reuse suggested in the optimal solution is the result of economic convenience. It should be pointed out that referring to node 8 as a dummy sink does not necessarily imply the non-existence of a sink; rather, the dummy sink can represent any allowable sink in the region, i.e., a natural stream, a lake or a reservoir, a disposal site, or any other feasible discharge point.

A dummy water treatment plant (node 9) was included in the model network structure to incorporate the effect of water treatment costs, since the freshwater costs provided by the Texas Department of Water Resources included neither treatment nor transportation costs. Therefore, in order to model the actual situation, it is assumed that all surface sources send their water to the dummy treatment plant before it is sent to the City of San Antonio. The transportation cost, which includes operation and maintenance for piping and pumping and pipe construction, is included in the arc from each surface source to the water treatment plant. The treatment cost is then included as the water passes through the plant (node 9). The arc from node 9 to node 10 has a zero cost.

One of the alternatives to be considered is the further treatment of the treated effluents from the Leon Creek, Rilling Road and Salado Creek wastewater at the new Confluence plant. Since the model does not consider interaction among wastewater treatment plants, each of three existing plants was assigned a dummy user which receives water from the plants, without producing any quality changes in the water as it passes through the node, and sends it to the new Confluence plant.

Using the modifications described above, the San Antonio region is schematically represented by the resulting model network structure shown in Figure 3.2. The same network structure is used for the San Antonio example application in the years 1980, 2000 and 2030; the users' water demands, as listed in Table 3.2 differs for the three years (Table 3.6).

The single period planning model minimizes the total annual cost which can include the annual operation and maintenance costs for piping, pumping and treatment, the annual cost equivalent for the capital cost of new pipe construction, and the cost of freshwater. The multi-period planning model minimizes the total present cost equivalent for all the factors described above, plus the capital cost associated with the construction of new treatment plants and expansion of existing facilities. Cost functions used for this example are listed in Table 3.1. Table 3.2 lists the freshwater cost function cofficiencts. Table 3.3 lists the length for each pipe link considered in the planning process as well as the resulting costs coefficients. Table 3.3 also lists the elevation difference for each arc requiring pumping and the annual operation and maintenance pumping cost coefficients. The cost coefficients for the initial cost (million \$) and the annual cost equivalent (million \$1 yr) for pipeline construction are given in Table 3.3. A 5% interest rate and a 25-year period, as suggested by the Texas Department of Water Resources was used.

3.3 SINGLE-PERIOD MODEL APPLICATION

Application of the single period planning model is illustrated by three different scenarios for 1980, 2000 and 2030. In all three cases it was assumed that both proposed reservoirs, Applewhite and Cibolo, are available freshwater sources; and their economic impact is considered only by their high freshwater cost. The proposed Confluence plant is also considered to exist for all of the scenarios. Its construction cost is not considered to exist for all of the scenarios. Its construction cost is not considered, since the model does not include treatment plant capacity considerations. The construction cost and/or the capacity expansion cost of the proposed Confluence wastewater treatment plant and the dummy water treatment plant, are included in the objective function.

San Antonio Region in 1980 - Case 1

The first case considers the San Antonio area during 1980 for which the network is that shown in Figure 3.2. Table 3.4a lists the characteristics of the sources which include the water availability, the quality of the freshwater, and the maximum acceptable mass discharge of pollutant. Table 3.4b lists the users characteristics, including water demand and consumption, maximum concentration acceptable, impact on water quality and others. Information related to treatment plants, both water and wastewater, are listed in Table 3.4c, including the plant capacity, water losses, and removal efficiency.

Figure 3.2 shows the numbering system for flow variables and Table 3.5 shows the numbering system for the concentration variables.

The initial solution used in this case was obtained from the (OKA-LP) procedure detailed in (Ocanas and Mays 1980 and Ocanas 1980). The optimal flow distribution is shown in Figure 3.3, and Table 3.5 lists the optimal concentration values for each node. As shown in Figure 3.3, the City of San Antonio (node 10) only receives water from the Edwards Aquifer; therefore, importing water from Canyon Lake or Cibolo Reservoir is not required. Also note that node 4 (Applewhite Reservoir and Medina Reservoirs) is not required and therefore, variable X(5), which represents the flow from node 4 to node 11, is actually the flow from Medina Reservoir to the Medina Irrigation Area. The model solution indicates the convenience of using the treated effluents from Rilling Road wastewater treatment plant (node 20) to irrigate both irrigation zones, Alamo Valley (node 12) and Rilling Road Zone (node 15). Both power plants, Calaveras (node 13) and Braunig (node 14) do recycle large volumes of water and use the freshwater from Calaveras Creek (node 6) and San Antonio River (node 5) respectively, as make-up water to compensate water losses. Another interesting point is the fact that none of the proposed plants is to be constructed, even though no construction costs for treatment facilities are included in the single-period planning. This indicates that for 1980 the existing water and wastewater system in the San Antonio region is self-sufficient. A final observation is the fact that most of the treated effluents produced

TABLE 3.4a System Element Characteristics for 1980: Sources

Node	BOD Concentration, mg/l	BOD Maximum Discharge Acceptable, MGD-mg/l	Water Availability, MGD
1	5.0	500.0	27.0
2	3.0	500.0	18.0
3	2.0	500.0	255.0
4	5.0	1000.0	32.0
5	5.0	500.0	11.0
6	5.0	500.0	24.0
7	5.0	500.0	14.0
8		9000.0	0.0

TABLE 3.4b System Element Characteristics for 1980: Users

Node	BOD Maximum Concentration Acceptable, mg/l	BOD Concentration Increment, mg/l	Removal Efficiency, %	Water Demand, MGD	Consumption, MGD
10	5.0	260.0	0	214.0	73.0
11	10.0	100.0	0	18.0	18.0
12	10.0	100.0	0	21.0	21.0
13	15.0	0.0	0	1339.0	16.0
14	10.0	0.0	0	536.0	6.0
15	10.0	100.0	0	4.0	4.0
16	300.0	0.0	0	0.0	0.0
17	300.0	0.0	0	0.0	0.0
18	300.0	0.0	0	0.0	0.0

TABLE 3.4c System Element Characteristics for 1980: Treatment Plants

Node	BOD Removal Efficiency, %	Plant Capacity, MGD	Water Losses, MGD
9	15.0	100.0	0.0
19	97.2	24.0	0.0
20	98.1	94.0	0.0
21	97.2	24.0	0.0
22	98.1	154.0	0.0

TABLE 3.5 Optimal Concentrations Leaving Each Node: Cases 1, 2, and 3

Node	BOD, mg/l		
	Case 1	Case 2	Case 3
1	5.00	5.00	5.00
2	3.00	3.00	3.00
3	2.00	2.00	2.00
4	5.00	5.00	5.00
5	5.00	5.00	5.00
6	5.00	5.00	5.00
7	5.00	5.00	5.00
8			
9	3.68	4.25	4.14
10	262.00	262.69	263.08
11	105.00	105.00	105.00
12	105.00	105.00	105.00
13	5.00	5.00	5.00
14	5.00	5.00	5.00
15	105.00	105.00	105.00
16	7.34	7.18	7.37
17	4.98	5.00	4.77
18	6.40	7.36	7.37
19	7.34	7.36	7.37
20	4.98	4.99	5.00
21	7.34	7.36	7.37
22	1.33	4.99	5.00

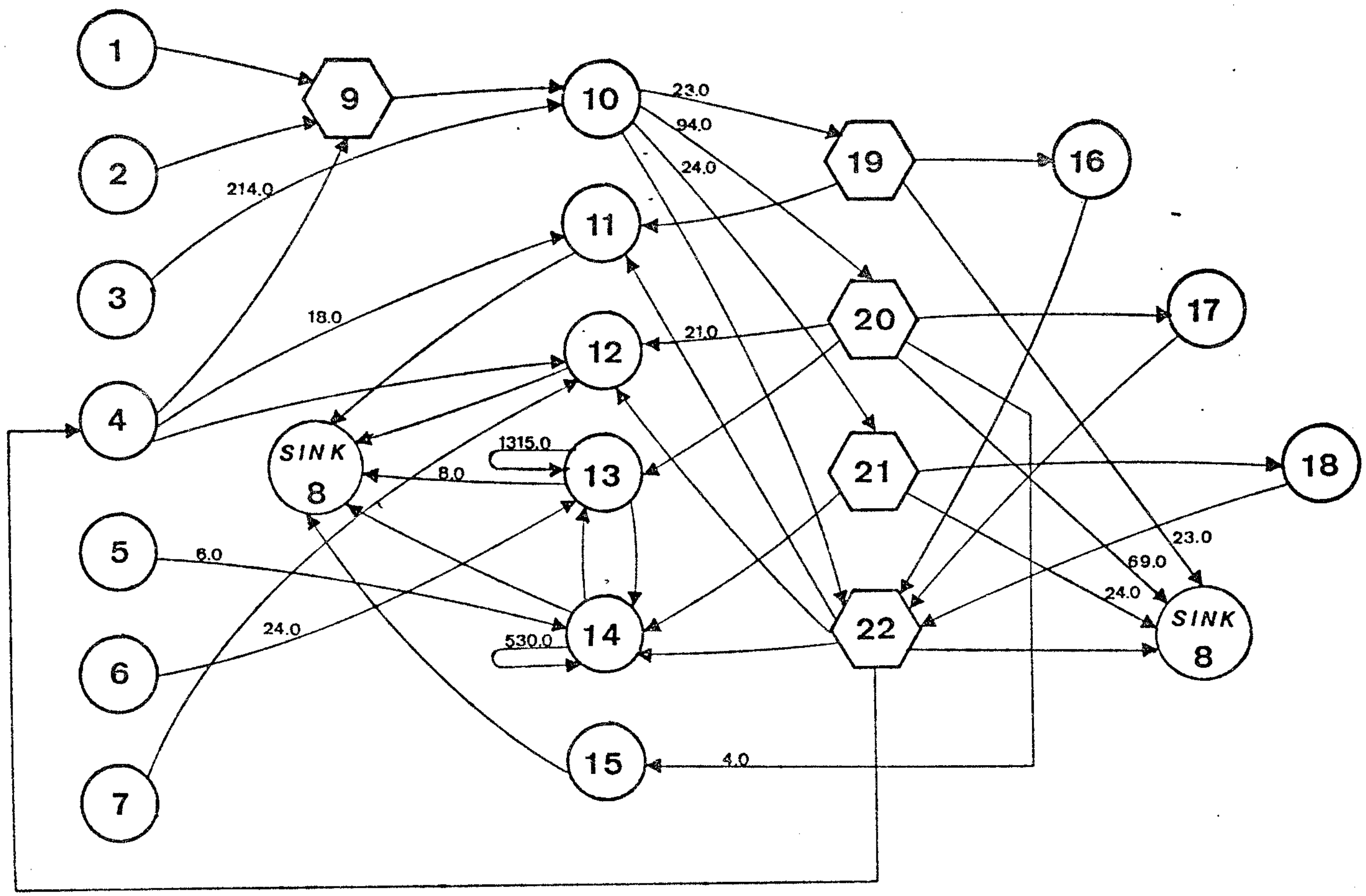


Figure 3.3 Optimal Flow Distribution for the San Antonio Region Case 1

by the existing wastewater treatment plants, are discharged to the dummy sink (node 8), indicating that for 1980, water reuse is not justified. The objective function value at optimality was 37.155 million dollars per year and other computational aspects such as the execution time are listed in Table 3.6.

San Antonio Region in Year 2000 - Case 2

The second case considers the San Antonio area during year 2000. The elements included are the same as in year 1980. Table 3.7 lists the projected water demands and consumptions as well as other related information for year 2000. The network structure for year 2000 is the same structure used for year 1980, as shown in Figure 3.2.

The optimal flow distribuiton is shown in Figure 3.4 and the resulting optimal concentration leaving each node is listed in Table 3.5. Figure 3.4 shows that for year 2000, Applewhite Reservoir (node 4) will be required and its surface water will compliment the Edwards Aquifer (node 3) to satisfy the water demands for the City of San Antonio (node 10). The Confluence Wastewater Treatment Plant (node 22) will be required and its treated effluent will be indirectly reused by sending it to the Applewhite Reservoir (node 4), which in turn will supply the City of San Antonio. The total annual cost for the flow distribution in the San Antonio region in year 2000 is 39.056 million dollars per year, which is slightly higher than that of Case 1, despite the fact that no construction costs other than piping are included. The main reason for the increased costs are the higher water demands for the region in year 2000.

San Antonio Region in Year 2030 - Case 3

The third case considers the San Antonio area during year 2030. The problem structure is the same as in Cases 1 and 2. The only difference was the water demands and consumptions for the users in year 2030. Table 3.7 lists the projected water demands and consumptions as well as other relevant information for year 2030 as provided by the Texas Department of Water Resources.

The optimal flow distribution is shown in Figure 3.5 and the associated optimal concentration variables are listed in Table 3.5. For year 2030 all proposed surface freshwater sources, i.e., Canyon Lake, Cibolo Reservoir and Applewhite Reservoir (nodes 1, 2, and 4, respectively), will be used to compliment the Edwards Aquifer (node 3) to satisfy the water demands for the City of San Antonio (node 10). Both the Calaveras (node 13) and Braunig (node 14) power plants will recycle large volumes of water. Make-up water for Calaveras will be obtained from the Calaveras Creek (node 6) while Braunig will use the San Antonio River (node 5) for make-up water. An interesting point is the fact that two of the existing wastewater treatment plants, Leon Creek (node 19) and Salado Creek (node 21) are not used. This behavior can be explained by: (1) no construction costs are included for the proposed Confluence plant (node 22); (2) economies of scale favor

TABLE 3.6 Computational Results

Case	Decision Variables				Number of Constraints	Nonlinear Functions	Execution Time,* s	Method	Objective Function, 10^6 \$/year
	Flow	Concentration	Capacity	Total					
1	42	22		64	68	32	51.042	LSGRG	37.155
2	42	22		64	68	32	120.798	LSGRG	39.056
3	42	22		64	68	32	99.244	LSGRG	67.042
4	126	66	15	207	216	92	30.852	SLPR	10,767.45

* University of Texas CYBER 175/750 system.

TABLE 3.7 Users Characteristics for 2000 and 2030

Node	BOD			Year 2000		Year 2030	
	Maximum Concentration, mg/l	Concentration, Increment, mg/l	Removal Efficiency, %	Water Demand, MGD	Consumption, MGD	Water Demand, MGD	Consumption, MGD
10	5.0	260.0	0	324.10	155.35	513.38	179.46
11	10.0	100.0	0	17.86	17.86	17.86	17.86
12	10.0	100.0	0	21.43	21.43	21.43	21.43
13	15.0	0.0	0	1607.10	20.54	1607.11	20.54
14	10.0	0.0	0	535.70	6.25	535.70	6.25
15	10.0	100.0	0	4.00	4.00	4.00	4.00
16	300.0	0.0	0	0.00	0.00	0.00	0.00
17	300.0	0.0	0	0.00	0.00	0.00	0.00
18	300.0	0.0	0	0.00	0.00	0.00	0.00

TABLE 3.8 Optimal Concentrations for Case 4

	Node Number			BOD Concentration, mg/l, SLPR			Capacity, MGD, SLPR		
	Period 1	Period 2	Period 3	Period 1	Period 2	Period 3	Period 1	Period 2	Period 3
Canyon Lake	1	23	45	5.00	5.00	5.00			
Cibolo Reservoir	2	24	46	3.00	3.00	3.00			
Edwards Aquifer	3	25	47	2.00	2.00	2.00			
Applewhite/Medina Reservoir	4	26	48	5.00	5.00	5.00			
San Antonio River	5	27	49	5.00	5.00	5.00			
Calaveras Creek	6	28	50	5.00	5.00	5.00			
Carrizo Aquifer	7	29	51	5.00	5.00	5.00			
Dumy sink	8	30	52						
Dummy WTP	9	31	53	3.45	4.25	4.13	66.9	193.3	258.4
San Antonio City	10	32	54	262.00	262.50	268.10			
Medina irrigation zone	11	33	55	105.00	107.30	107.30			
Alamo irrigation zone	12	34	56	105.00	105.00	105.00			
Calaveras power plant	13	35	57	5.00	5.09	5.00			
Braunig power plant	14	36	58	5.21	7.31	5.00			
Rilling Road irrigation zone	15	37	59	105.00	105.00	105.00			
Dummy user	16	38	60	7.34	7.35	7.37			
Dummy user	17	39	61	4.98	4.99	5.00			
Dummy user	18	40	62	7.34	7.35	7.37			
Leon Creek WWTP	19	41	63	7.34	7.35	7.37	23.5	25.9	25.9
Rilling Road WWTP	20	42	64	4.98	4.99	5.00	93.9	96.1	96.1
Salado Creek WWTP	21	43	65	7.34	7.35	7.37	23.5	23.5	23.5
Confluence WWTP	22	44	66	1.10	4.98	4.95	0.0	185.3	284.8

WTP is water treatment plant; WWTP is wastewater treatment plant.

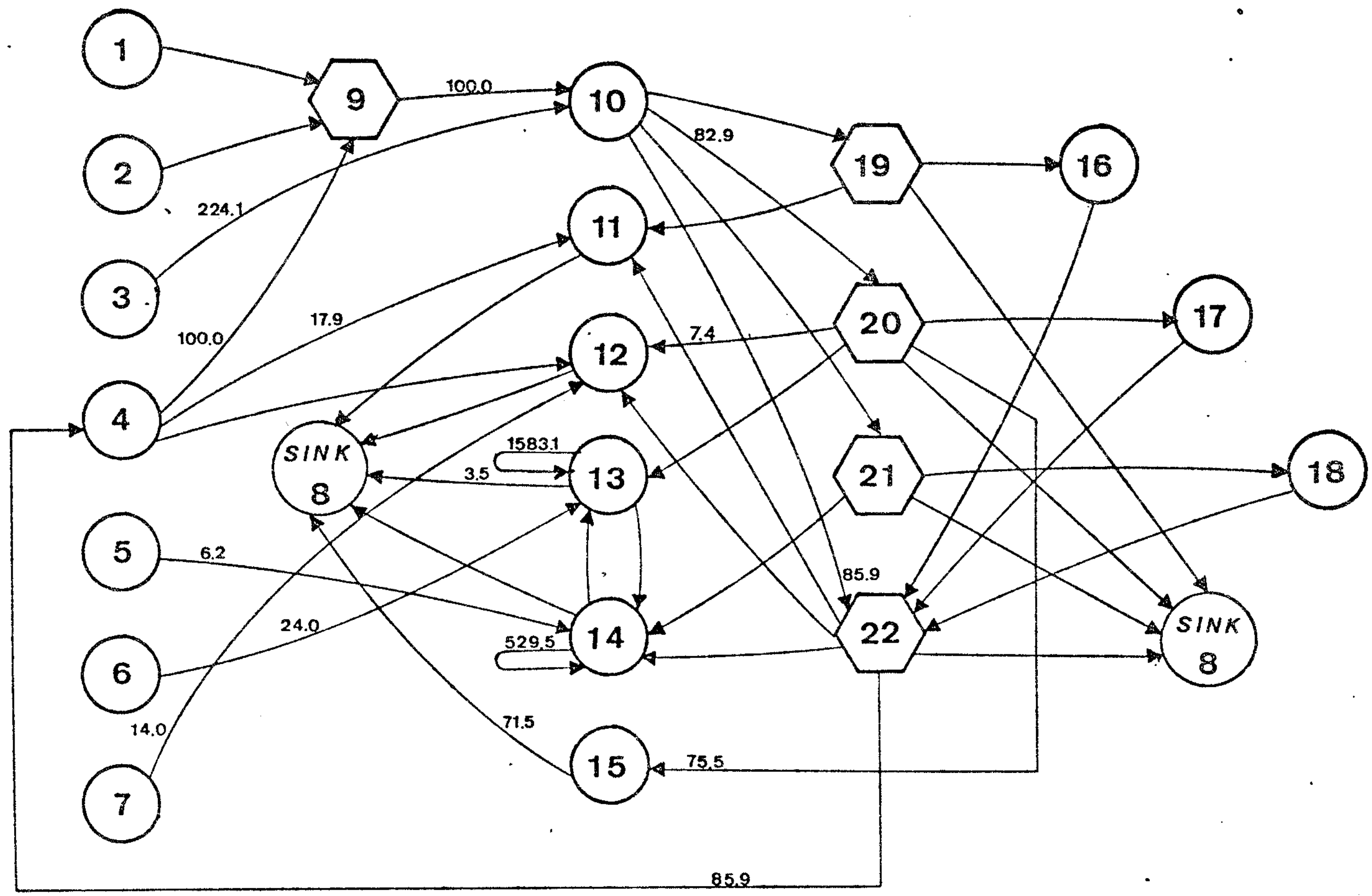


Figure 3.4 Optimal Flow Distribution for the San Antonio Region, Case 2

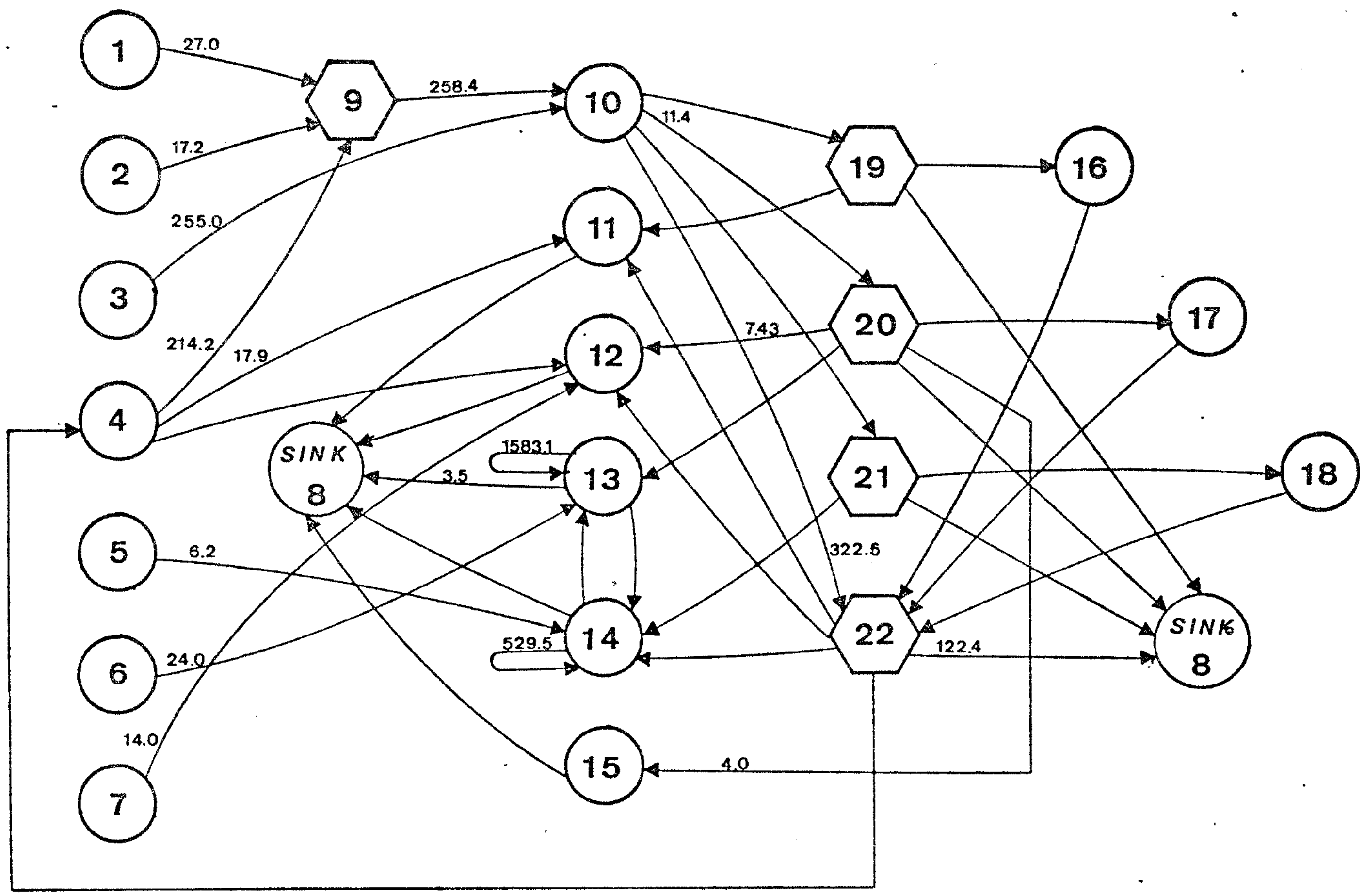


Figure 3.5 Optimal Flow Distribution for the San Antonio Region, Case 3

fewer and larger treatment plants over a larger number of smaller plants; (3) the relatively high water demands for the City of San Antonio (node 10) call for indirect reuse; and (4) only the Confluence plant allows for the reuse of treated effluents to supply the City of San Antonio.

As in Cases 1 and 2, no treated effluents from any of the existing plants (nodes 19, 20 and 21) are to be sent to the Confluence plant (node 22) for further treatment, indicating that, as far as BOD is concerned, one-stage treatment is enough. The total annual cost for the flow distribution in the San Antonio region in year 2030 is 67.042 million dollars per year, which is a significantly higher cost than for Cases 1 and 2, due mainly to the need for large quantities of water reuse which require the construction of large pipelines.

3.4 MULTI-PERIOD MODEL APPLICATION

Case 4 considers the San Antonio region for three planning periods: 1980-2000, 2000-2030, and 2030-2060. The characteristics for each planning period are the same as for Cases 1, 2 and 3. Most of the assumptions made during the first three cases are also applied for this case, but construction and capacity expansion of treatment facilities are included. The capacity variables representing the existing treatment plants, Leon Creek, Rilling Road and Salado Creek during period 1 are constrained to be less than or equal to their respective initial capacities and are only allowed to be expanded after the first period is over. Table 3.8 lists the node numbering system used for this multi-period application. The construction costs for new pipelines are included in this case as a single payment cost at the beginning of the planning period in which they are available.

A first attempt was made to solve this problem using the LSGRG method. The code used a conjugate gradient method (Polak-Ribiere method) for finding the search directions, as described in Ocanas and Mays (1980). The code failed to find an optimal solution after 1200 seconds of execution time. The problem was then solved using the successive linear programming with rejection (SLPR) method.

The optimal flow distributions obtained used SLPR for the three planning periods are shown in Figures 3.6a-c. Table 3.8 lists the resulting concentrations and the capacity for the treatment plants during each period.

A comparison of the single-period model results and the multi-period model results indicated the need for a multi-period type model. The optimal flow distribution for Cases 1, 2, and 3 (Figures 3.3, 3.4, and 3.5, respectively) can be compared with the optimal flow distributions for periods 1, 2, and 3 of Case 4 (Figures 3.6a, b and c, respectively).

Comparing the single-period and multi-period flow distributions for Case 1 (Figures 3.3 and 3.6a), indicate only minor differences in the flow distributions. However, comparing the flow distributions for Cases 2 and 3 with the flow distributions for periods 2 and 3 of Case form, indicates significant differences in the flow distributions. Comparing the BOD concen-

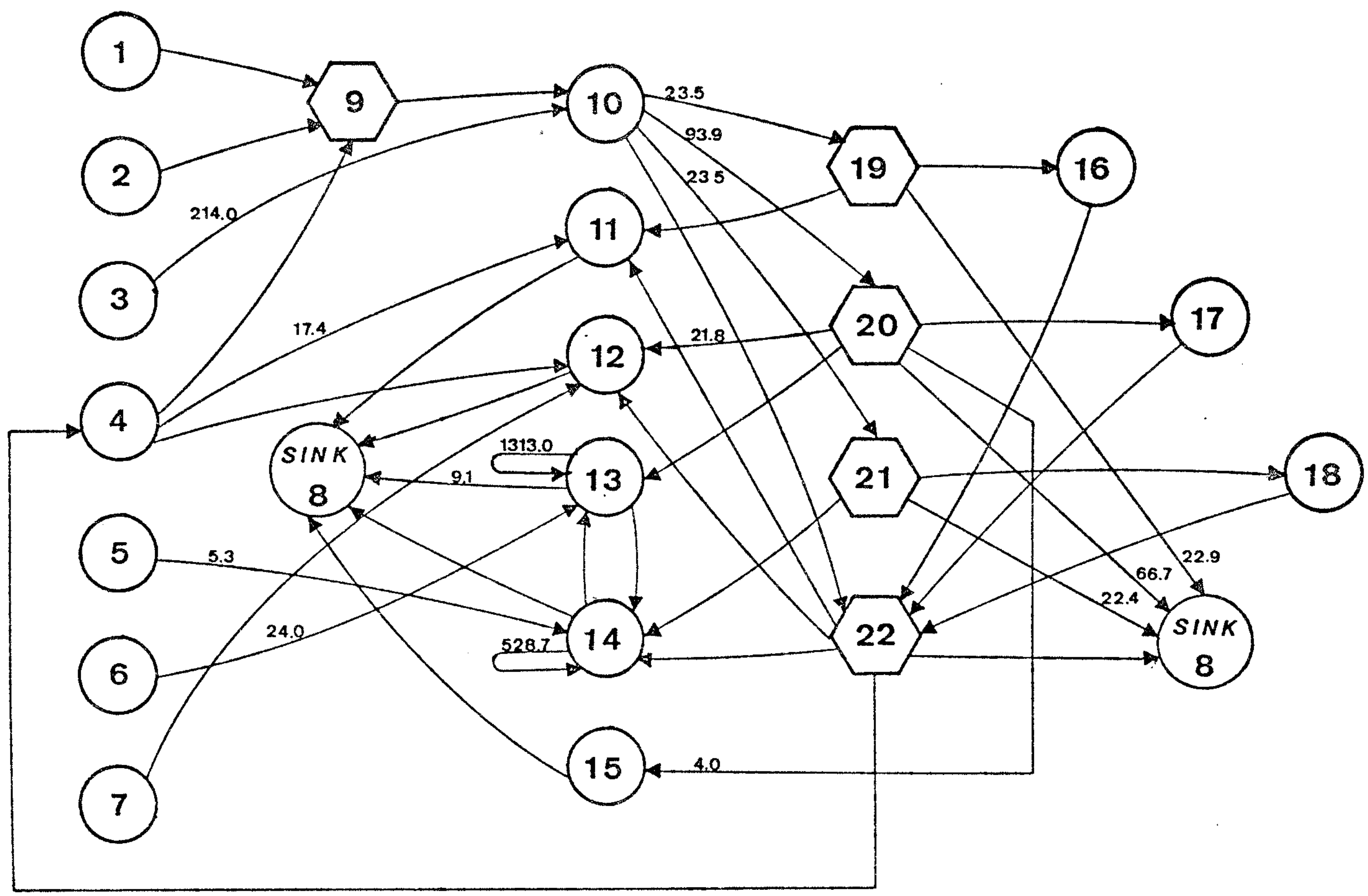


Figure 3.6a Optimal Flow Distribution for the San Antonio Region, Case 4, Period 1

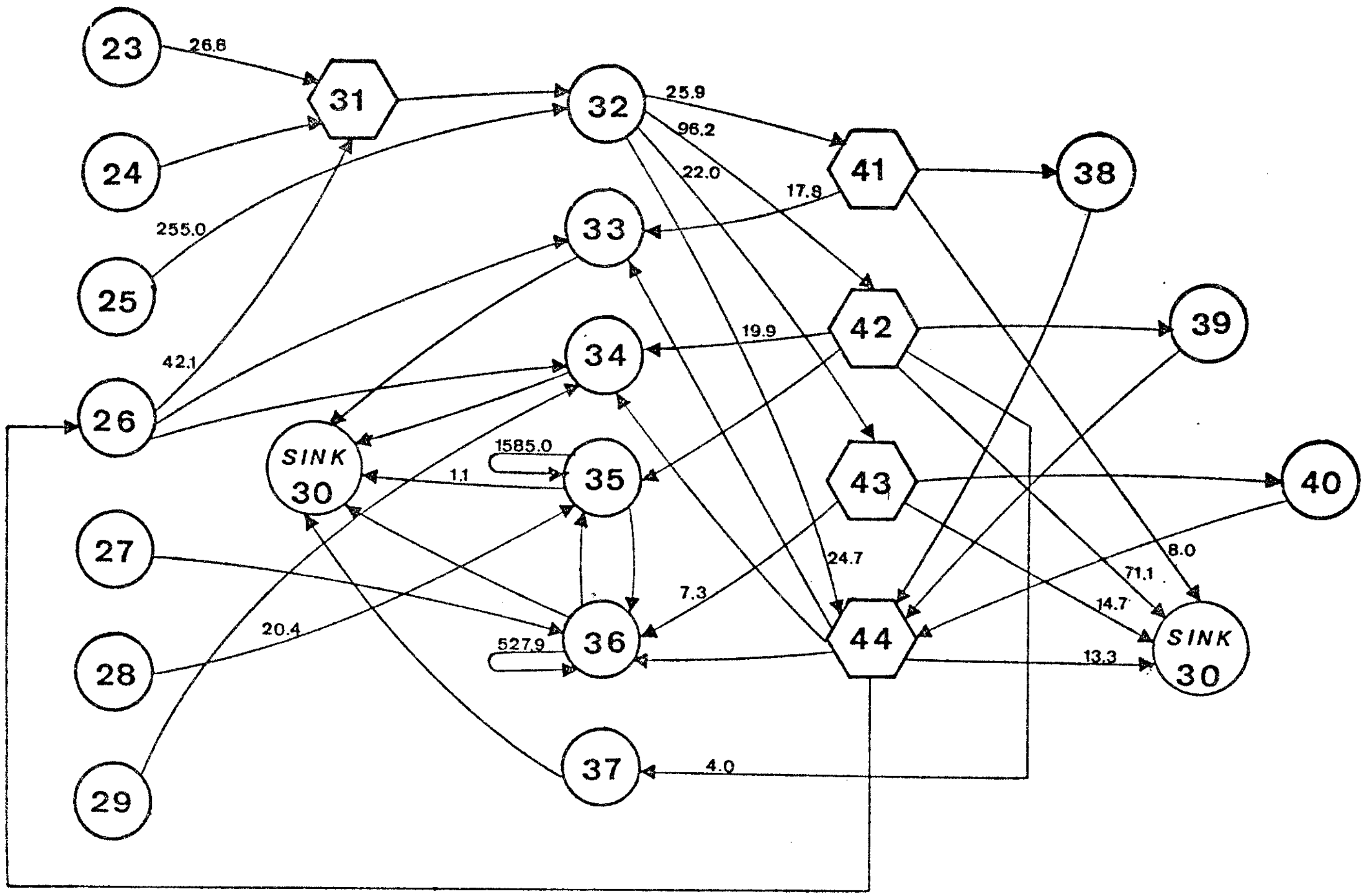


Figure 3.6b Optimal Flow Distribution for the San Antonio Region, Case 4, Period 2

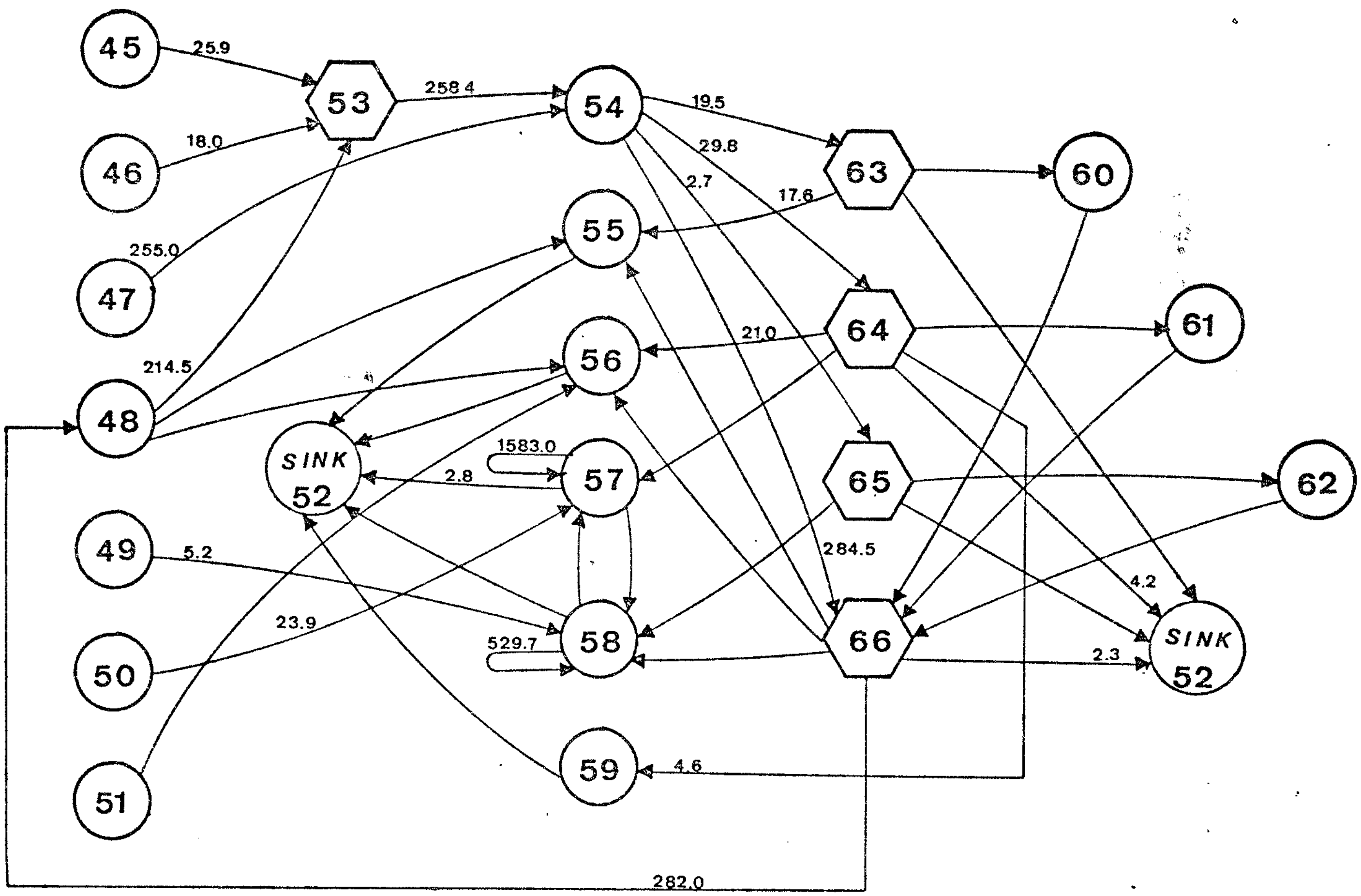


Figure 3.6c Optimal Flow Distribution for the San Antonio Region, Case 4 , Period 3

trations of the single-period results for Cases 1, 2, and 3 (Table 3.5) with the multi-period results for period 1, 2, and 3 of Case 4 (Table 3.8) indicates only minor differences in the concentration levels. However, the capacity for treatment plants is somewhat different.

The flow distribution during period 3 calls for large amounts of surface water to complement the Edwards Aquifer (node 48) in supplying the water demands for the City of San Antonio (node 54), differing significantly from Case 3 (Figure 3.5). All the available surface sources during period 3, -- Canyon Lake, Cibolo, and Medina and Applewhite Reservoirs (nodes 45, 46, and 49, respectively) -- are used to their full extent, with the Applewhite Reservoir indirectly reusing the treated effluents from Confluence Wastewater Treatment Plant (node 66). During period 3, it is also observed that most of the wastewater produced by the City of San Antonio is either directly or indirectly reused, indicating the necessity of water reuse practices to meet the growing demands of the region. Water reuse is highlighted by the use of treated sewage in all irrigation areas, Medina Valley, Alamo Valley and Rilling Road (nodes 55, 56 and 59) leaving the freshwater sources for municipal use.

This comparison of the single-period model results and the multi-period model results indicates that the solution of a sequence of single period problems does not correctly represent the dynamic nature of the system.

Figure 3.7 shows the capacity scheduling for each of the treatment plants in the region. One interesting feature is that the optimal solution calls for over-designed treatment facilities whenever the next period would require an expansion of the facility. This fact indicates the importance of solving the multi-period problem whenever a more accurate capacity construction scheduling is desired.

The total present cost equivalent for this case was 10,767.45 million dollars and some of the computational aspects are listed in Table 3.6. Other information related to the performance statistics of SLPR is given by Ocanas (1980) and Ocanas and Mays (1980).

3.5 SUMMARY, LIMITATIONS, AND CAPABILITIES OF LSNLP MODELS

A real world application of the single period and multi-period planning models is presented. The single period model provided insight as to how water should be allocated within a region; however, it neglected the capacity expansion of the system by disregarding its interaction with other planning periods. The multi-period model does consider capacity expansion for the system; and through its application to the study area, it showed the importance of the interaction among various planning periods when capacity construction scheduling is to be analyzed.

Application of the two models indicated their usefulness in selecting the "best" planning alternative. Thanks to the flexibility allowed by the models, various alternatives for the water and wastewater system can be simultaneously considered, improving the utility of the alternatives

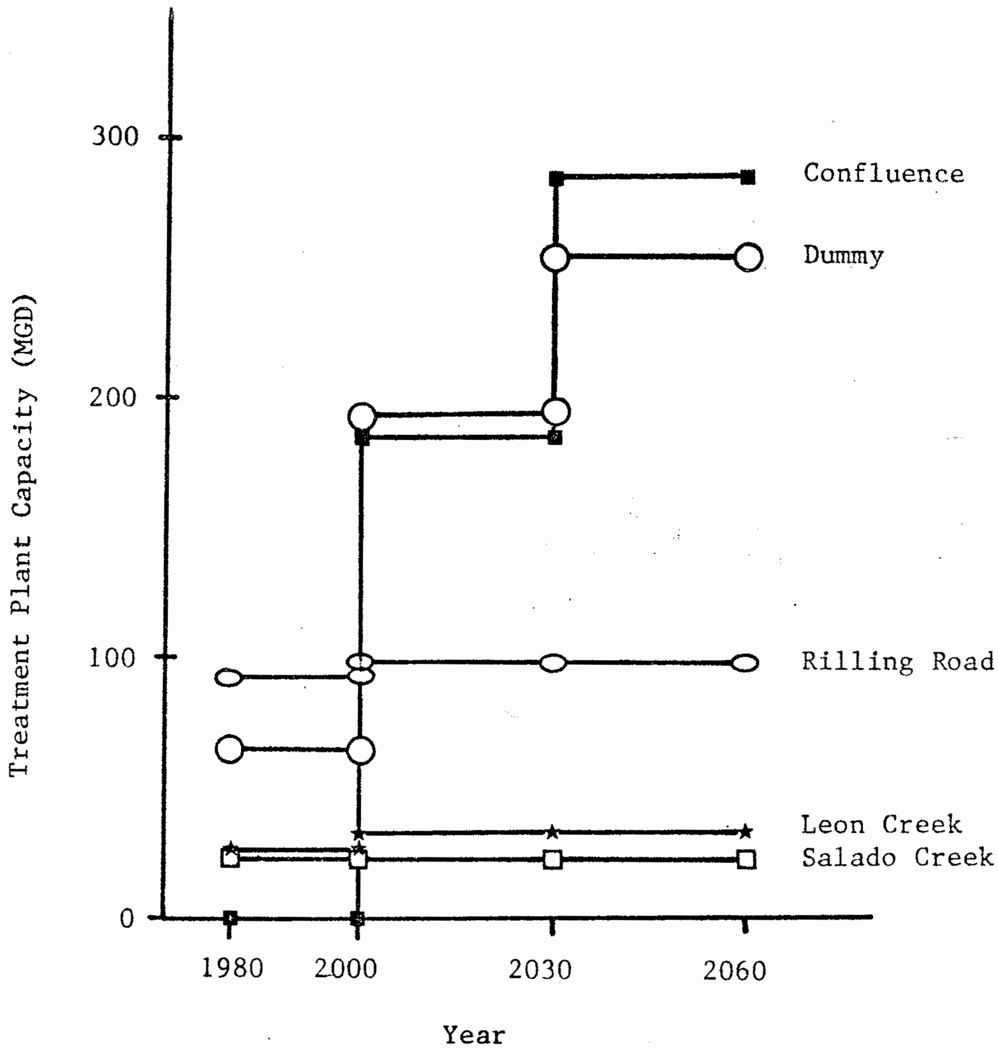


Figure 3.7 Capacity Expansion of Treatment Plants (Case 4)

generated by the model. Different alternatives regarding the growing nature of the system can also be incorporated in the multi-period model, thus providing capacity expansion scheduling.

As previously stated, to obtain a more accurate prediction for the capacity expansion of the system, it is necessary to incorporate several planning periods. This results in larger problems which are more difficult to solve, if indeed, they are solvable at all. However, rapid advancement of nonlinear optimization provides optimism for our ability to solve very large problems with many planning periods.

The limitations of the model include:

1. The model does not provide the ultimate answer to the problem of water allocation and capacity expansion within a region. The forecasting of the economic and system parameters suffers considerably in accuracy as the forecast horizon is extended. Furthermore, the exclusion of social and political implications from the model limit its role.

2. The model is deterministic; no stochastic or seasonal variations are included. The model assumes constant system parameters during the length of each planning period, and only allows changes in the parameters at the beginning of a new period.

3. The model does not guarantee global optimality, due to the non-convex structure of the constraint set and the concavity of the objective function when economies of scale are incorporated. However, the use of good initial points leads to quick convergence to good local optimum.

The capabilities of the model include:

1. The model includes capacity expansion considerations. It is general enough to incorporate both existing and non-existing treatment facilities, and to consider their capacity expansion once they are built.

2. The model explicitly considers growth in water use. A planning horizon can be divided into several planning periods. The shorter the periods, the more accurate the model represents the real world. There is, however, a trade-off between the number of planning periods analyzed and the size of the resulting problem, which is particularly important for nonlinear problems.

3. The model includes economies of scale in treatment and transportation costs. The objective function includes operation and maintenance costs for piping, pumping and treatment facilities, the construction costs for new piping and treatment facilities and the capacity expansion costs for existing treatment plants.

SECTION 4

DYNAMIC PROGRAMMING MODELS: MODEL STRUCTURE

4.1 GENERAL STRUCTURE

The previously developed nonlinear programming model (LSNLP) for determining the optimal allocation of wastewater for multi-period planning was designed to consider regional aspects over a long planning horizon with relatively large planning periods (e.g. 5, 10, 20 years). Shorter time periods (e.g. 1 year) could be used; however, the problem size could become too large to solve with existing nonlinear programming techniques. Also, the LSNLP model does not consider the selection of various treatment processes.

Models are developed in this section to determine optimal wastewater reuse allocation and treatment alternatives on a much smaller time scale. Two models, each based upon dynamic programming (DP), are developed: (1) the allocation DP model, which is used for the optimal allocation of water to secondary users and (2) the treatment DP model, which performs the optimal selection of treatment schemes (processes) over time.

In the allocation DP model the stages are represented as users, the decision variable is the amount of water allocated to a user from each possible treatment alternative considered, and the state variable is the amount of water left to allocate from the treatment alternative. The allocation DP model is solved for each possible treatment alternative and for each time period in the planning horizon. The objective function is to minimize costs. Once the allocation DP model has been solved for each possible treatment alternative and each time period in the planning horizon the treatment DP model is solved. The allocation DP model essentially is used to define the state space in the treatment DP model. The stages in the treatment DP model are represented by the time periods in the planning horizon. The state variable represents the various combinations of treatment alternatives defined by the allocation DP model and the decision variable is represented by the choice of treatment alternatives. The overall model then provides a minimum cost (present dollars) water allocation and wastewater treatment scheme, considering water reuse, over a planning horizon. The DP models are applied to the case study of San Antonio, Texas.

Physical Characteristics of the System

The major water related components considered in a water distribution and treatment system include: (1) freshwater source(s);(2) water treatment plant(s); (3) users-primary and secondary; (4) wastewater treatment plant(s); and (5) final sink(s)

The roles of the latter three components, that is, the users, the wastewater treatment plant(s), and the final sink(s) are fully investigated in this report. Assumptions are made concerning the first two components to allow this investigation.

(1) Water Sources in Sinks

In Figure 4.1, the interactions between sources and other components as well as water sinks and other components considerd by this model are presented. Because assumptions are made concerning the freshwater source and the water treatment plants, little attention is given to the interactions represented in Figure 4.1-a. Instead Figure 4.1-b will be investigated thoroughly.

Water sources in this model are expressed as an extension of the primary user. It is assumed that there exists a source of water, with a particular maximum yield, that can meet the demands of a user directly or via a water treatment plant. If it is ascertained that, even with water reuse, not enough water is present to meet primary user demands, a supplementary source is assumed to exist. There is no maximum yield limitation on this supplementary source.

All water routed to the sink is first treated in the wastewater treatment plant. The water sink only receives water that is not provided to a secondary user from the wastewater treatment plant. Entities that demand water from a wastewater treatment plant (WWTP) are consumptive secondary users. It is assumed that the fraction of water that would be wasted from the secondary users to the sink is very small, insignificant for the purposes of this model.

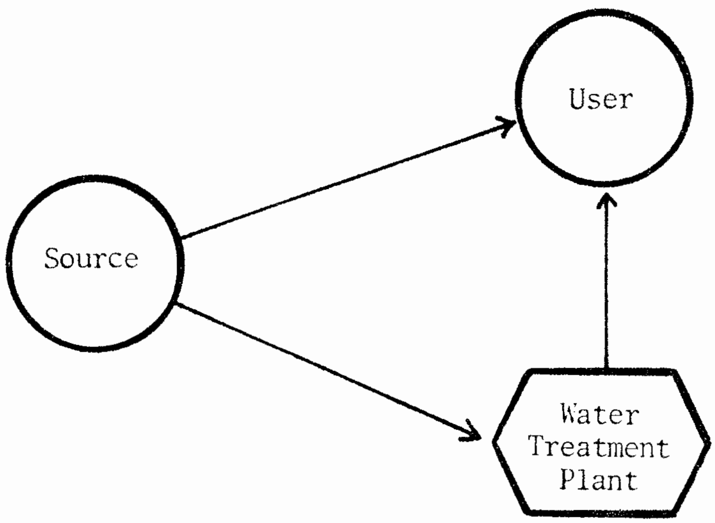
(2) Water Treatment Plants

As seen in Figure 4.2, the water treatment plant has little interaction with other components in the system. In the model presented here, a water treatment plant has no role and therefore is not defined. Yet, this does not mean that it could not be considered as part of the model.

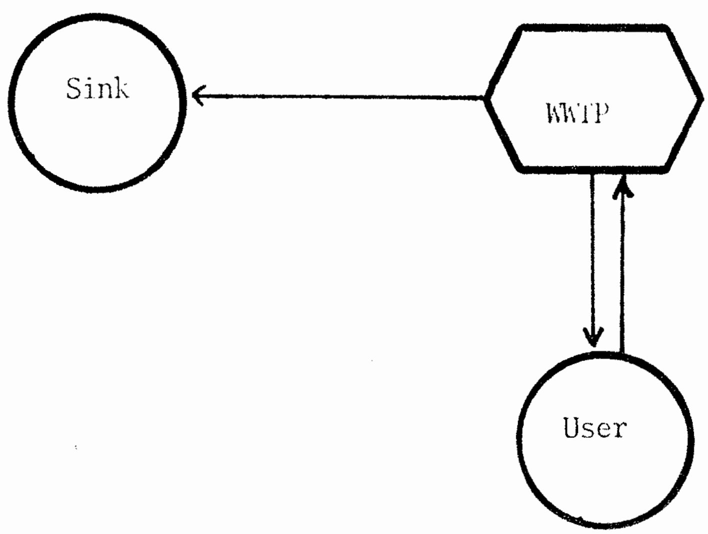
(3) Users

There are two types of users in this model, primary and secondary. Primary users are entities that demand water directly from the freshwater source and water treatment plant. Secondary users demand water directly from the wastewater treatment plant. Figure 4.3 (a) schematically depicts the overall role of a user of either type while Figure 4.3 (b) distinguishes between primary and secondary users.

The role of a primary user in this model is important for two reasons. According to the system, the primary user is defined by the amount and quality of the water routed to the WWTP. The amount and quality of this water is directly dependent upon the primary users' utilization of the water supply. Primary users are, in essence, the water supply sources for the secondary users.



(a) Source - User - Water Treatment Plant Interaction



(b) Sink - Wastewater Treatment Plant - User Interaction

Figure 4.1 Water Source and Sink Interactions with Other Components in the System

Figure 4.2 Water Treatment Plant Interaction with Other Components in the System

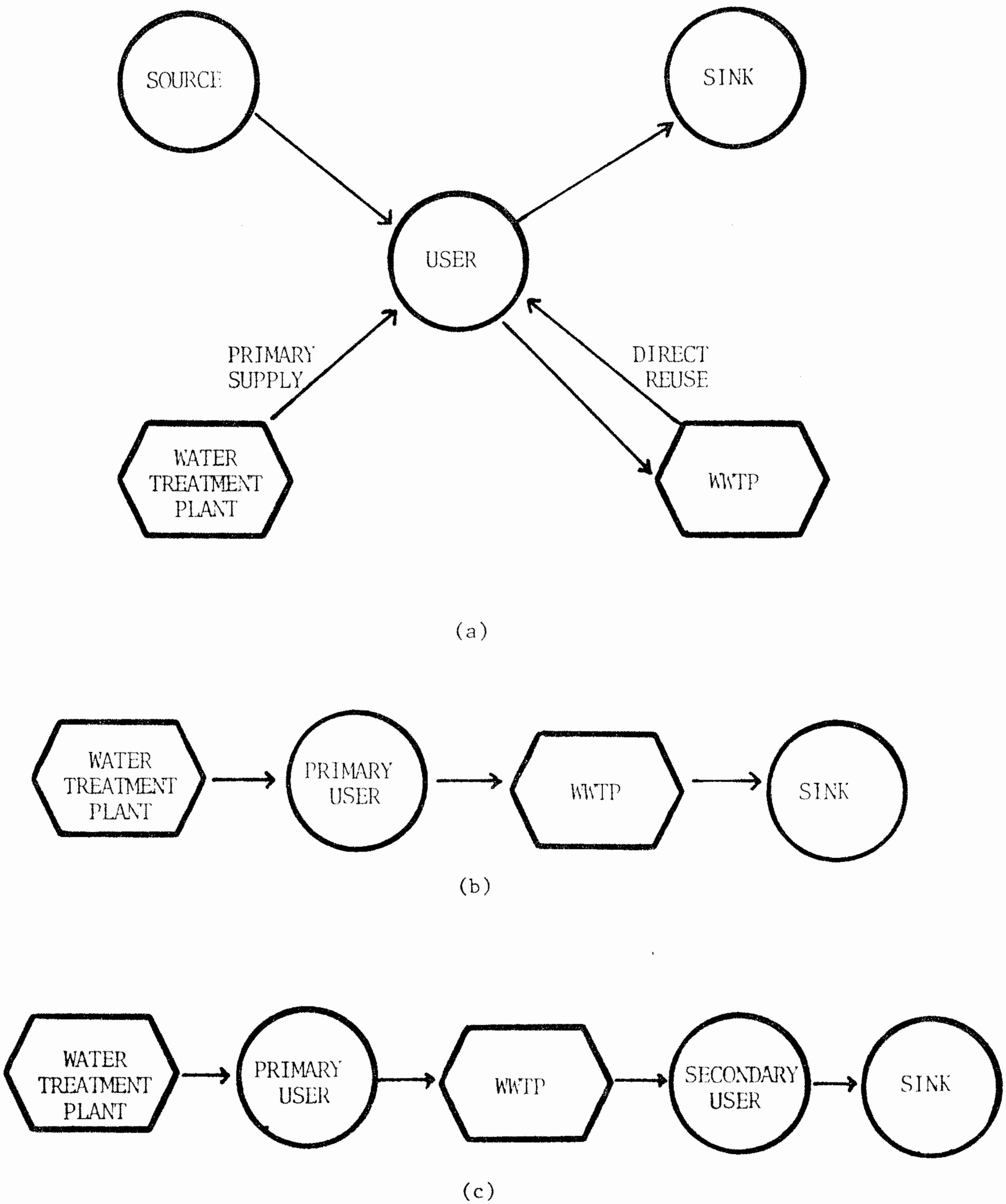


Figure 4.3 User Interactions with Other Components in the System

Users that have low quality demands, low pollutant addition rates, and can recycle their water many times are considered good candidates for direct water re-usage (user of effluent water from the WWTP). Entities that receive water yet do not return the water to the system are also attractive secondary users.

Secondary users, then, are defined by their,

1. Quantity demands--water received

2. Maximum acceptable pollutant concentrations

3. Distance from the WWTP

An irrigation area is one example of a good candidate for the role of a secondary user since its quality demands and pollutant addition rates are low and since it does not directly return water to the system.

(4) Wastewater Treatment Plants

The wastewater treatment plant is the most important component when considering the quality of the water to be provided for direct reuse. Each wastewater treatment plant is defined by its,

1. Location in the region considered by the model

2. Peak flow capacity

3. Treatment efficiency (dependent on plant type)

4. Water losses (consumption)

The role of a WWTP in this system can be more clearly seen in Figure 4.4.

An overview of the physical structure of the system (i.e., a summary of the roles of the components) is diagramatically depicted in Figure 4.5.

Assumptions

Overall assumptions were made to narrow the focus of the model described and to make the model usable on a general basis. There are two major assumptions made concerning freshwater sources and water treatment plants. It is assumed that there will always be 1) a large enough quantity of freshwater of 2) treatable quality to provide the user component with its demanded quantity and quality.

Water usage characteristics must be supplied by the model user. For the purpose of this presentation quantity usage projections are presented as total yearly withdrawals and consumption (water losses from the system as a direct result of water uses).

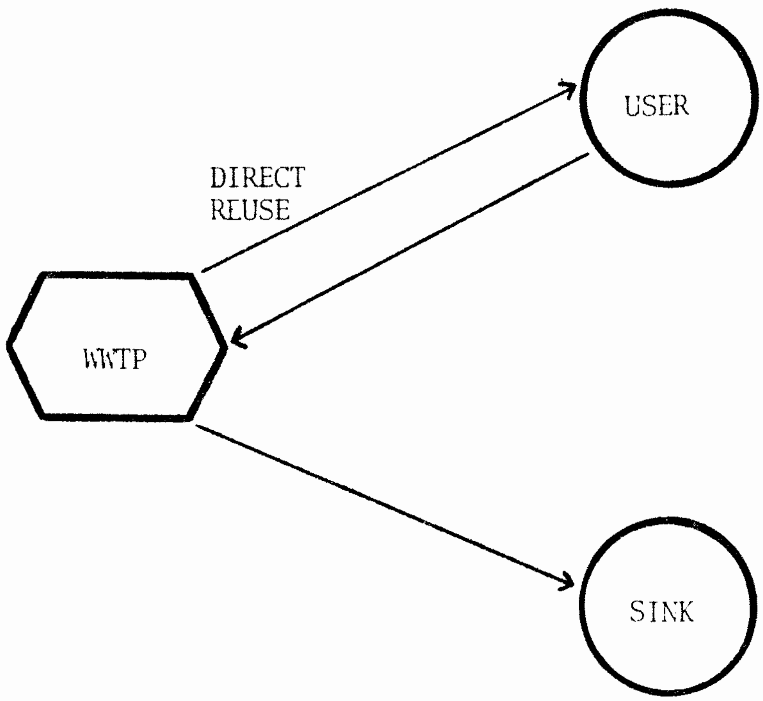


Figure 4.4 Wastewater Treatment Plant Interactions with Other Components in the System

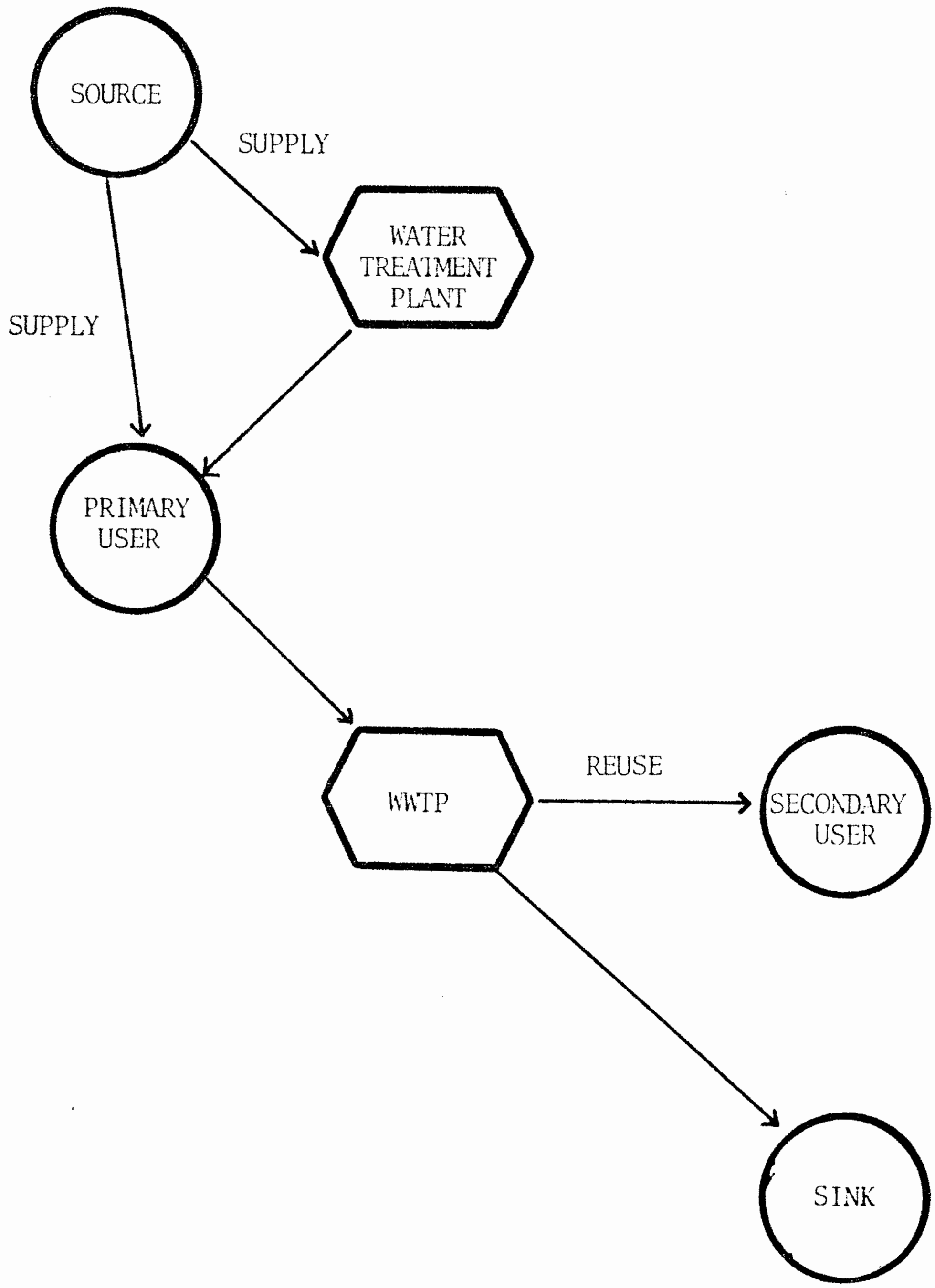


Figure 4.5 Overall Interaction Between Model Components

If there is insufficient water to meet the minimum demands of the secondary users a supplemental source is assumed available at an inflated cost. This cost can include many things but for the purposes of this model it includes the cost of treatment needed for this supplemental water, its transportation cost, and an arbitrarily weighted cost. The added arbitrary cost reflects the judgement that a water conservation policy should be initiated by the secondary users as if they were primary users (e.g., secondary users should recycle water where possible or, use less). If conservation is deemed impossible then the arbitrary weight can be lowered or removed.

After primary usage the water must be treated for future reuse. The amount of wastewater treatment is inherently dependent on which secondary users will receive an allocated quantity of water and at what quality level the secondary user requires the allocation. This is the crux of the water reuse model. Different treatment alternatives are considered based on the secondary user's quantity and quality demands.

A basic assumption is made concerning the treatment alternatives considered. Incorporated in the allocation and treatment models is the assumption that treatment levels (the quality of the effluent from the wastewater treatment plant) are dependent upon the influent quantity. Effluent quality is based on efficiency ratings taken from efficiency versus quantity information (e.g., Figure 4.6) estimated for each treatment alternative. These estimations are to be made when the decision maker has chosen which treatment alternatives should be considered in the model. Treatment alternative designs should be based on simple treatability studies which are assumed available for the wastewater in question.

Conceptual Structure of the Model

The water reuse model built around the physical components described in the previous section and presented here contains two major submodels: 1) a model allocating water to secondary users and 2) a model evaluating different treatment schemes over time. Before these models can be used the data needed for the model must be made available and in a usable form. Therefore this model also includes a submodel which collects, generates and organizes data into a usable form. The conceptual algorithm for the overall model is shown in Figure 4.7 and hereafter the submodels are referred to as the data presentation model, the allocation model, and the treatment model.

The data preparation model can include any type of data generation program. Deterministic data generation via linear regression or stochastic generation via Markov models are both viable generation models to use to provide data needed for the allocation and treatment models. Data such as freshwater availability over time or quality and quantity changes (e.g., due to population growth) in influent to the wastewater treatment plant for the planning period in question are needed and must be projected. Both linear regression and Markov models are commonly used to project this needed data. Figure A.3 in Appendix A presents a flow chart of the submodel DATFOR used to generate and organize the data needed for the model and case study presented here.

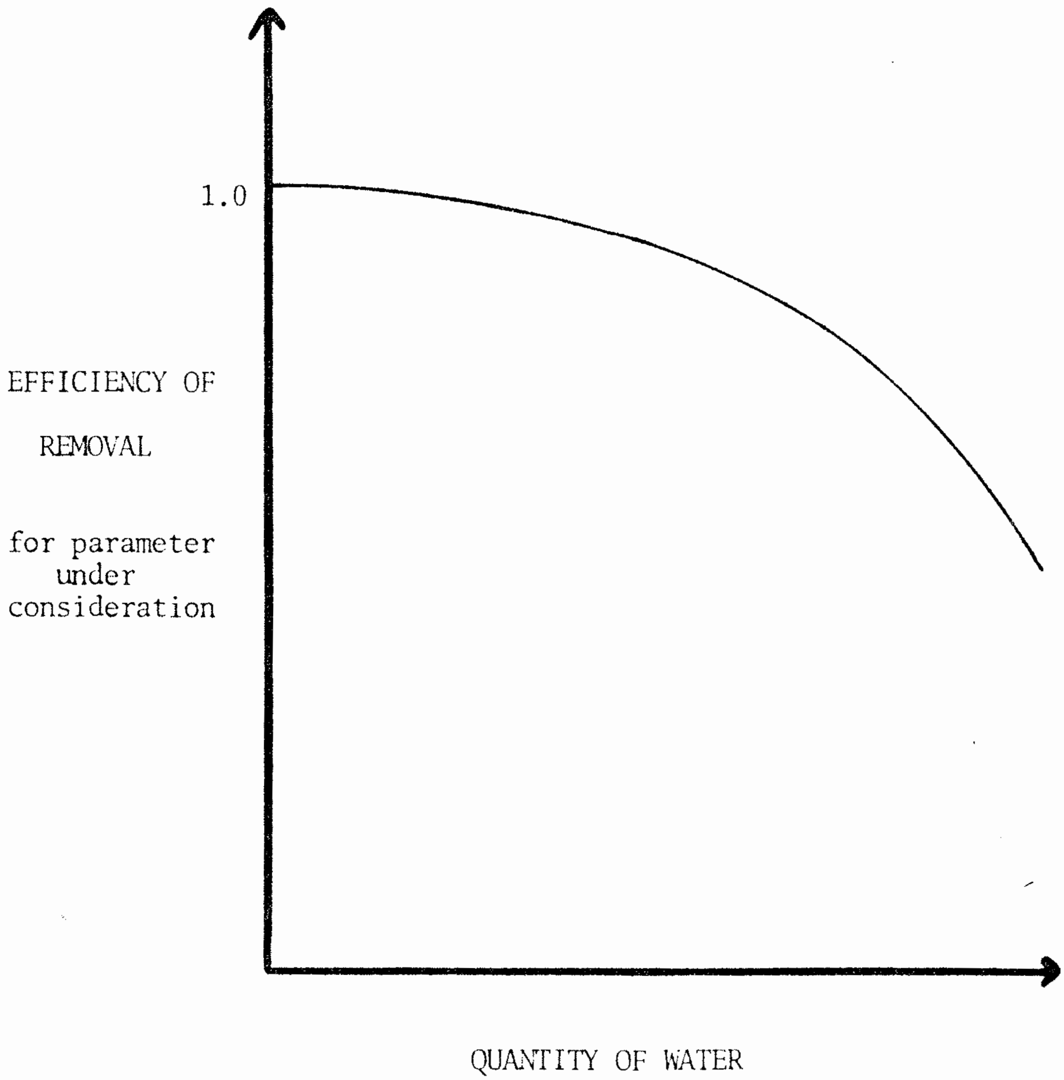


Figure 4.6 Removal Efficiency vs. Water Quantity Information

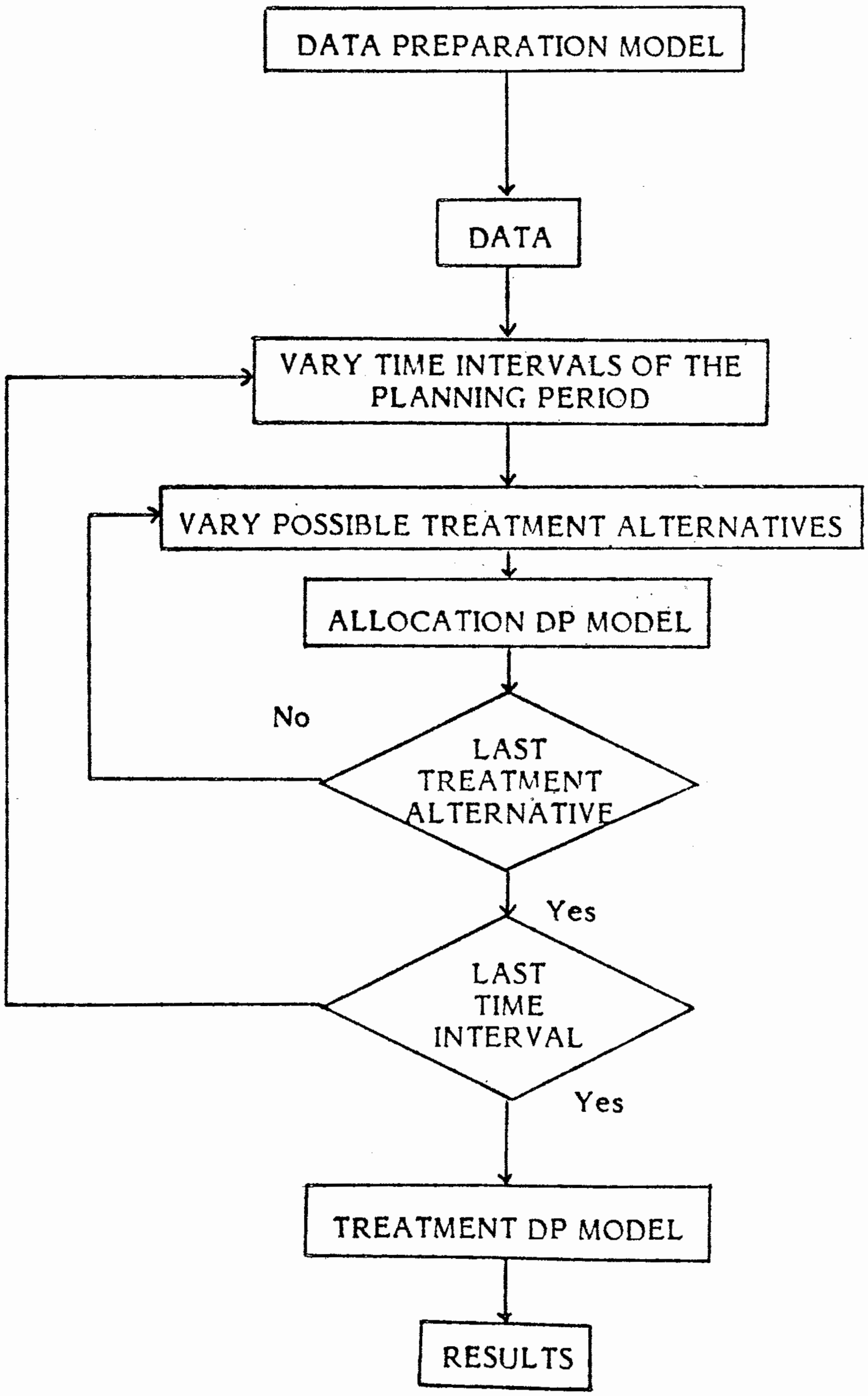


Figure 4 .7 Conceptual Algorithm for the Overall Model

The allocation model determines how the available water is to be allocated so as to reuse as much of it as possible. Water must be treated before it is allocated so the allocation model also evaluates the impact of a treatment alternative upon the allocation scheme. For example, if a wastewater treatment alternative cannot provide a secondary user with water at a quality level desired, then the treatment alternative is infeasible. The allocation scheme is not necessarily infeasible, though. It can be evaluated under another treatment alternative.

All the allocation schemes that have a feasible treatment alternative connected to the scheme are described in this model and then evaluated. The allocation scheme that promotes the most water reuse for each treatment alternative is singled out for use in the treatment model. The allocation model is solved for each time interval so an allocation which promotes the most water reuse for each treatment alternative and each time period is stored for use as the data set (state space) for the treatment model.

The treatment model evaluates the state space defined above for the minimum cost combination of treatment alternatives--a treatment scheme. Any type of treatment alternative can be considered. For example, a treatment scheme based on a system built from the primary, to secondary, to tertiary stages with capacity expansion can be considered over a planning period.

In this presentation a more common occurence is investigated. Oftentimes distribution and treatment systems are already in place and need to be modified to meet the quality requirements of the secondary users in the region. This is the problem considered in this presentation and a realistic solution is sought. The type of modification needed and when this modification must be placed on line are the important information needed.

Modification alternatives that can be evaluated by this model can also have graduated options, as does the former example. For example, immediate expansion of existing plants and then the later addition of a new plant might be considered in the application of this model. For this presentation, though, separate treatment alternatives were considered, as seen in the latter occurence.

The optimal treatment scheme--the minimum cost scheme--for the planning period in question is sought after a discount rate is applied and a net present value is assigned to each treatment scheme.

The overall model then provides a minimum cost scheme in present dollars and will define a treatment scheme and an allocation scheme over time. The decision maker(s) must then decide if this scheme meets political, social, and other environmental constraints. If not, the costs for the other alternatives have also been determined in the process of defining the minimum cost alternative and can be presented by the model for the decision-maker(s) perusal.

4.2 DECISIONS, DATA AND PROBLEM STATEMENT

User Decision and Data

In order to use the model, two major decisions are,

1. How many and which treatment alternatives should be considered?

2. What planning period and discretization thereof (e.g., yearly time intervals, seasonal time intervals, etc.) should be chosen?

Required data for the time period under consideration include:

Water Usage Data

1. Water availability from freshwater sources

2. Water usage estimates by users/water routed to wastewater plant

3. Consumption losses due to usage

4. Potential secondary users' minimum and maximum quantity demands

5. Sink quantity demand

Water Quality Data

1. Influent quality of wastewater to the plant for the parameters under consideration

2. Potential second user quality demands for parameters under consideration

Distance

Distance from wastewater plant to potential secondary user

Overall Problem Statement

A word statement of the problem is as follows:

Objective--To find the minimum cost solution which identifies water allocation, and the water treatment scheme that promotes water reuse in a potentially water short area. Real costs include the costs of wastewater treatment and the transportation costs include piping and pumping costs. The minimum cost solution includes these real costs and an aribtrary cost which promotes water reuse.

The results of the model must be systematic, understandable, and usable for the purposes of policy decisions on water bond timing and water rate structures.

Constraints

1. Demand Constraints

    a. User minimum demand

    b. User maximum demand

2. Capacity Constraints

3. Mass Balance Constraints

4. Water Availability Constraints

    a. From the freshwater source

    b. From a supplementary source

5. Water Quality Constraints

    a. User's quality requirements

    b. Quality changes produced by wastewater treatment plants

The final results of the overall model will include:

1. An allocation scheme which promotes water reuse over the planning period in question;

2. The treatment alternative and level needed for each time interval; and

3. The minimum cost treatment scheme for the planning period in question.

The more discrete the time interval, the more accurate are the results. Thus the decision maker will have a systematic basis on which to evaluate and make policy decisions concerning the water treatment system in a region and future water allocations and needs.

Nomenclature

$COST_{j,k}$	=	total operation and maintenance cost for treatment alternative
D	=	decision variable
$DQUAL_{i,j}$	=	quality level demand for secondary user i in time period j
$E_{j,k}$	=	efficiency of treatment alternative k in time interval j
i	=	secondary user index
j	=	time interval index
k	=	treatment alternative index
ℓ	=	treatment plant index
$OM_{j,k}$	=	total operation and maintenance costs for treatment alternative k in time period j
$OP_{j,k}$	=	piping operation and maintenance costs for treatment alternative k in time period j.
QA_j	=	water quantity available for allocation to secondary users in time period j
$QALOC_{j,i}$	=	water quantity allocation for secondary user i in time period j.

$QCAP_{j,k,\ell}$ = peak quantity that can be treated by plant ℓ for alternative k in time period j.

$QL_{j,k}$ = consumption losses due to primary usage and treatment alternative k in time period j

$QMAX_{j,i}$ = maximum quantity acceptable by secondary user i in time period j

$QMIN_{j,i}$ = minimum quantity demanded by secondary user i in time period j

QS_j = supplemental water supply assumed available to secondary users in time period j

QT_j - total treated water in time interval j

$QUAL_{j,i}$ = influent water quality to secondary user i in time period j.

S = state variable

TCOST = total cost over the planning period of allocation and treatment schemes

α, β = cost coefficients

4.3 ALLOCATION MODEL DESCRIPTION

Allocation Model Problem Statement

A word statement of the problem considered in this model which is solved for each treatment alternative for each year in the planning period is as follows:

Objective To find a minimum cost water reuse allocation scheme considering different wastewater treatment alternatives. Costs include wastewater treatment costs, water transportation costs and arbitrarily weighted costs which promote allocation of water for reuse.

Constraints

1. Water availability constraints

   a. From the primary user(s)

   b. From a supplementary source

2. Secondary user's water demand constraints

   a. Minimum demand

   b. Maximum demand

3. Mass Balance Constraints

   a. For wastewater treatment plants

   b. For the system

4. Water quality constraints

   a. Secondary users' quality requirements

   b. Quality changes produced by the wastewater treatment plant

5. Capacity constraints for each treatment alternative

Model Constraints

There are five general types of constraints to describe the model for a distribution system: 1) availability, 2) demand, 3) mass balance, 4) quality, and 5) capacity constraints. These are described in the following sections.

(1) Availability Constraints

Assumptions--These constraints insure that there is water of a treatable quality available to potential secondary users. Availability is dependent upon the amount of water used by the population of the region considered and the consumptive losses during the water usage and the wastewater treatment process.

Description--This constraint can be expressed as,

$$QT_j + QS_j - QL_{j,k} = QA_j \qquad \forall\ i,\ k \tag{4.1}$$

where QT_j = total water treated in time period j; QS_j = supplemental water available to secondary uses; $QL_{j,k}$ = consumptive losses due to primary usage in time and treatment alternative k; and QA_j = water quantity available for allocation to secondary users.

(2) Demand Constraints

Assumptions--Each secondary user is assigned a minimum quantity that must be provided during a particular time period. Each secondary user is also assigned a maximum demand. If it is possible to accomplish without depriving other secondary users of needed water, quantities greater than the minimum but less than the maximum quantity demanded will be allocated.

The investigation of the range between the minimum and maximum quantity demanded by each secondary user is advantageous for many reasons. For example, if more water than the minimum quantity demanded can be provided to a secondary user for irrigation purposes, crop output may benefit. Yet a maximum demand must be imposed so that, for example, the crops are not overwatered.

Description--The demand constraints are,

$$QMIN_{j,i} \leq QALOC_{j,i} \leq QMAX_{j,i} \tag{4.2}$$

where QMIN and QMAX are minimum and maximum demands, respectively.

(3) Mass Balance Constraints

Assumption--In any water allocation network the water allocated throughout the system should not exceed the water available to the system. This requirement forms a mass balance around the system.

Description--A mass balance constraint must also be written so that the total allocated to the secondary users does not exceed the total available, i.e.,

$$\sum_i QALOC_{j,i} \leq QA_j \qquad \forall\ j \tag{4.3}$$

(4) Quality Constraints

Assumptions--Each allocation scheme tested in the allocation model is evaluated for each treatment alternative. This approach is taken for several resons. If, instead, all the possible allocation schemes were determined according to this model structure and then the treatment alternative was considered, and if said treatment alternative was determined infeasible, this would automatically render the treatment alternative infeasible under any model conditions, (e.g., before it was tested under other allocation schemes).

It is possible that an alternative allocation scheme might provide the quality level requested by the secondary user. For example, if a particular treatment plant within a treatment alternative could not provide the quality level requested by the secondary user, then the treatment alternative for that allocation scheme, only, is deemed infeasible by the model. Other water allocation schemes can still be tested under this treatment alternative.

An option which can be added to the model might be to provide water to a secondary user whose quality demands aren't met by one treatment plant but can be met by another treatment plant within the same treatment alternative considered. In this presentation only one effluent quality was considered for each treatment alternative. This part of the model could be further refined so that more than one influent to each wastewater treatment plant (and therefore more than one quality level in the effluents to be allocated to a potential secondary user) can be considered. This option was not investigated by this report but can be easily included in the computer program based on this model.

If the treatment efficiency was found to be insufficient, the amount of added efficiency required to attain the quality desired would be determined and presented in the model. This was done so that the model user might know the extent of the modification needed if a particular treatment alternative was favored due to outside considerations. Yet a treatment alternative that produces a quality level below that of the demanded quality of the user under model constraints will cause the allocation scheme under consideration to be determined infeasible until modifications needed are included in the input data.

Description--Many different types of quality constraints can be used in this model, e.g., constant, linear or nonlinear. The model user may consider as many parameters as desired, i.e., more than one pollutant can be considered. The addition of quality constraints to the model would make it more accurate and, a solution would be reached faster by the computer program based on the model since the feasible set of solutions to be investigated would be smaller.

Quality constraints for various parameters can be considered in the model. Basically, the influent quality, $QUAL_{j,i}$, to a secondary user i in time period j msut be less than or equal to the demanded quality level, $DQUAL_{j,i}$ i.e.,

$$QUAL_{j,i} \leq DQUAL_{j,i} \qquad \forall\ j,\ i, \tag{4.4}$$

(5) Capacity Constraints

Assumptions--Another determinant in the consideration of a treatment alternative is the capacity of the plant(s). It is assumed that each treatment alternative has a fixed capacity--the sum of the plant capacities. If the influent water exceeds the capacity of the treatment alternative limits, the alternative is infeasible.

The amount of added capacity required to appropriately treat the influent quantity without a by-pass of polluted water around the treatment plant is determined and presented. This is done so that when capacity expansion of the treatment alternatives is considered (either via future sub-models added to the model structure or by changing the treatment alternative input data and re-evaluating the alternatives) the model user will know the extent of modification that is required to make the alternative feasible.

Description--Each treatment alternative k for a region has a fixed capacity for each time period j, which is the sum of the plant capacities, i.e., $QCAP_{j,k}$. For a treatment alternative to be feasible the sum of plant capacities for each time period must equal or exceed the water to be treated.

$$QT_j \leq \sum_{\ell} QCAP_{j,k,\ell} \tag{4.5}$$

Objective Function

In the allocation model the objective is to minimize the cost of the allocation and treatment of wastewater, while also promoting as much reuse as possible. The costs that are considered in this model are:

Operation and maintenance of wastewater treatment alternative

$$OM_{j,k,\ell} = \alpha_{k,\ell} \, [\sum_i QALOC_{j;i}]^{\beta_k} \tag{4.6}$$

Operation and maintenance of conveyance system

$$OP_{j,k,i} = \alpha_{k,i} \left[QALOC_{j,i} \right]^{\beta_k} \qquad \forall \; i, j, k \tag{4.7}$$

The objective function for the allocation model is

$$COST_{j,k} = \text{MIN} \sum_{i=1}^{I} OP_{j,k,i} + \sum_{\ell} OM_{j,k} \tag{4.8}$$

The above function would not vary if different allocation schemes were not considered. When the model is modified to consider different water routes (e.g., to meet quality specifications) the above objective function would have a greater number of solutions to search through in order to find the minimum cost solution.

4.4 TREATMENT MODEL DESCRIPTION

Problem Statement

A word statement of the problem considered by this model is as follows:

Objective--To find a minimum total present value cost treatment scheme which will allow wastewater reuse during the planning period in question. Costs include capital costs, operation and maintenance costs, water transportation costs, and arbitrarily weighted costs which promote allocation of water for reuse.

Constraints--Technological constraints.

Model Constraints

Assumptions--There is only one type of constraint in this model--technological. To make the problem realistic and to prevent the addition of shut down and start up costs, the model is constrained such that a higher technology treatment alternative or a higher capacity cannot be replaced by a lower technology treatment alternative or a lower capacity treatment alternative in the year $j + 1$.

For example, if a graduated option scheme was being considered, a tertiary process could not be shut down for a year and be replaced by an expanded secondary plant and then have the tertiary process returned on line the year after (at the previous capacity rating) in order to minimize cost.

Treatment alternatives for this example must be hierarchically ranked. The lowest rank would be equivalent to the lowest form of treatment and the lowest capacity e.g., $k = 1$. Higher technology/capacity treatment alternatives proceed upward from this initial k.

Using unrelated treatment alternatives in the evaluation process makes the technological constraint more detailed. Alternatives are grouped according to their compatibility. Groups are also hierarchically ranked. The model allows movement between two time intervals from a tertiary process being added to two old plants to adding a new plant to the system, but would not allow the closing of the new plant and capacity expansion of the tertiary plants in the next time interval.

Description--This constraint can be expressed as a limitation on the index k,

$$k_{j+1} \geq k_j \qquad \forall j, \tag{4.9}$$

Objective Function

In the treatment model the objective is to minimize the treatment and allocation costs defined in the allocation model over the planning period in question.

The capital costs that are considered in the model are for the construction of plant costs,

$$PLANT_{j,k,\ell} = \alpha_k \, [QCAP_{j,k,\ell}]^{\beta_k} \tag{4.10}$$

and for pipe construction cost,

$$PIPE_{j,k} = \alpha_k \, QALOC_{j,i}^{\beta_k} \tag{4.11}$$

The operation and maintenance costs were previously defined as $COST_{j,k}$.

Total costs for the treatment are then equal to

$$COST_j = (COST_{j,k} + PLANT_{j,k,\ell} + PIPE_{j,k}) \quad \forall j, \tag{4.12}$$

In order for these costs functions to be compared on a common basis over time, operation, maintenance, and capital costs were assigned values in present value terms via the following technique. The model considers capital and operation and maintenance costs included in the objective function of the treatment DP to be in present value dollars so that future costs are discounted. The $COST_j$ is converted to PVC_j. It is assumed that construction will take no longer than one time interval in this model, although the capital cost estimates will not vary drastically if two time intervals are needed for construction.

$$PVC_j = COST_j * \frac{1}{(1+i)^j} \tag{4.13}$$

The modified objective function which expresses the minimization of all the present value cost equivalents for the planning period is then,

$$MIN \quad TCOST = \sum_{j=1}^{J} PVC_j \tag{4.14}$$

where TCOST = the total cost for the planning period.

Capital costs are considered here (and not in the allocation DP) for two reasons. When the search is performed over time, a particular solution will determine at what time interval a particular alternative or plant should come on line. The capital cost is added in that time interval only in this model. Further refinement of the model though, would allow the cost to be spread out over time prior to the interval in which it is determined that a particular alternative must be on line. This would make the problem solution a representation of a typical pay-back scheme and would be useful for the development of a portfolio to issue a capital investment bond.

Capital costs are also considered on a time dependent basis. It is assumed that capital costs will increase over time due to inflation and resource scarcity. Therefore, a particular solution of the treatment model requires that the alternative be on line at a later time interval than another solution; optimality may be found in the other solution. This suggests a trade-off between capital costs and operation and maintenance costs. It might be better to begin operation of another treatment alternative (e.g., a new plant) before operating the previously used alternative at capacity.

4.5 SOLUTION TECHNIQUE

Dynamic Programming

The structural framework of both the allocation and treatment models is based on a systems analysis-operations research technique called dynamic programming (DP). The following general description of dynamic programming is summarized from Mays and Tung (1980).

Dynamic programming is an approach-oriented technique. It requires that the mathematical equations describing the model be developed to fit the particular application under consideration.

To apply dynamic programming to a water resource and treatment system, the system must have the following characteristics:

1. The problem must be divisible into stages. A stage represents a point in time, space, or a physical entity depending on the components of the system.

2. State variables describing the condition of the system at each stage must be finite in number. The state variable is discretized into several states defining possible conditions of the system.

3. A decision must be made at each stage to transform the current state of the system into a state associated with the next stage. Returns in the form of cost/benefits are connected with each set of decisions.

4. For any stage or state of the system, the optimal sequence of decisions must be independent of the decisions made in the previous stages.

The sequence of decisions made are mathematically described by a recursive equation. This equation might also be described as a partial objective function ie.,

$$f_x(\underline{S}_x) = \underset{\underline{D}_i}{\text{MIN or MAX}} \; [r_x(\underline{D}_x, \underline{S}_x) \cdot f_{x-1}(\underline{S}_{x-1,i})] x=1,2,\ldots X \tag{4.15}$$

where $f_x(\underline{S}_x)$ represents the minimum or maximum of the objective through stage x for state vector $\underline{S}_x$; $f_{x-1}(\underline{S}_{x-1,i})$ represents the minimum or maximum of the objective through stage x for state n and is known; and $r_x(\underline{D}_x, \underline{S}_x)$ is the return function objective value for stage and is dependent on the current state, $\underline{S}_{x-1,i}$ and the decision, $D_{x,i}$ made. The operator "o" specifies how $r_n(\underline{D}_x, \underline{S}_x)$ and $f_{x-1}(\underline{S}_{x-1,i})$ are combined to yield $f(\underline{S}_x)$. For example, in most water resource applications "o" delineates a summation operation. The overall objective can be found by applying the recursive equation to each stage in the sequence beginning with stage 1 and terminating at x and by tracing through all the solutions for the solution(s) that meet the constraints of this problem (eg. a minimum or maximum solution).

Allocation Dynamic Program

The allocation model (solved for each treatment alternative k, k = 1,...K, and each time interval j, j = 1,...J) uses the following recursive equation,

$$f_i(S_{i,n}) = \underset{D_{i,n}}{\text{MIN}} [r_i(S_{i,n}, D_{i,n}) + f_{i-1}(S_{i-1,n})] \quad \begin{matrix} i=1,\ldots I \\ n=1,\ldots N \end{matrix} \tag{4.16}$$

where $f_i(S_{i,n})$ represents the minimum cost at stage n for state n, i is the set of potential users (stages), and n represents the state for the state variable. $r_i(S_{i,n}, D_{i,n})$ is the return function and represents the return determined by the arbitrary weighting scheme devised to promote water reuse and the operation and maintenance costs of piping and treatment for the treatment alternative k and the year j under consideration. $f_i(S_{i,n})$ is equivalent to $\text{Cost}_{j,k}$ (Eq. 4.8) in the Allocation DP. The basic structure of this model is seen in Figure 4.8.

The state variable $S_{i,n}$ for each treatment alternative k, k=1,...K and in each year j, j=1,...J, is the water quantity left over after a water quantity has been allocated to user i. All possible leftover quantities from which another allocation can be withdrawn at the next stage i are represented by,

$$\underline{S}_{i,n} = \begin{bmatrix} S_{i,1} \\ S_{i,2} \\ \vdots \\ S_{i,N} \end{bmatrix} \tag{4.17}$$

The transformation function defines the manner in which a state variable from the previous stage (i-1) is transformed into the state variable at stage i;

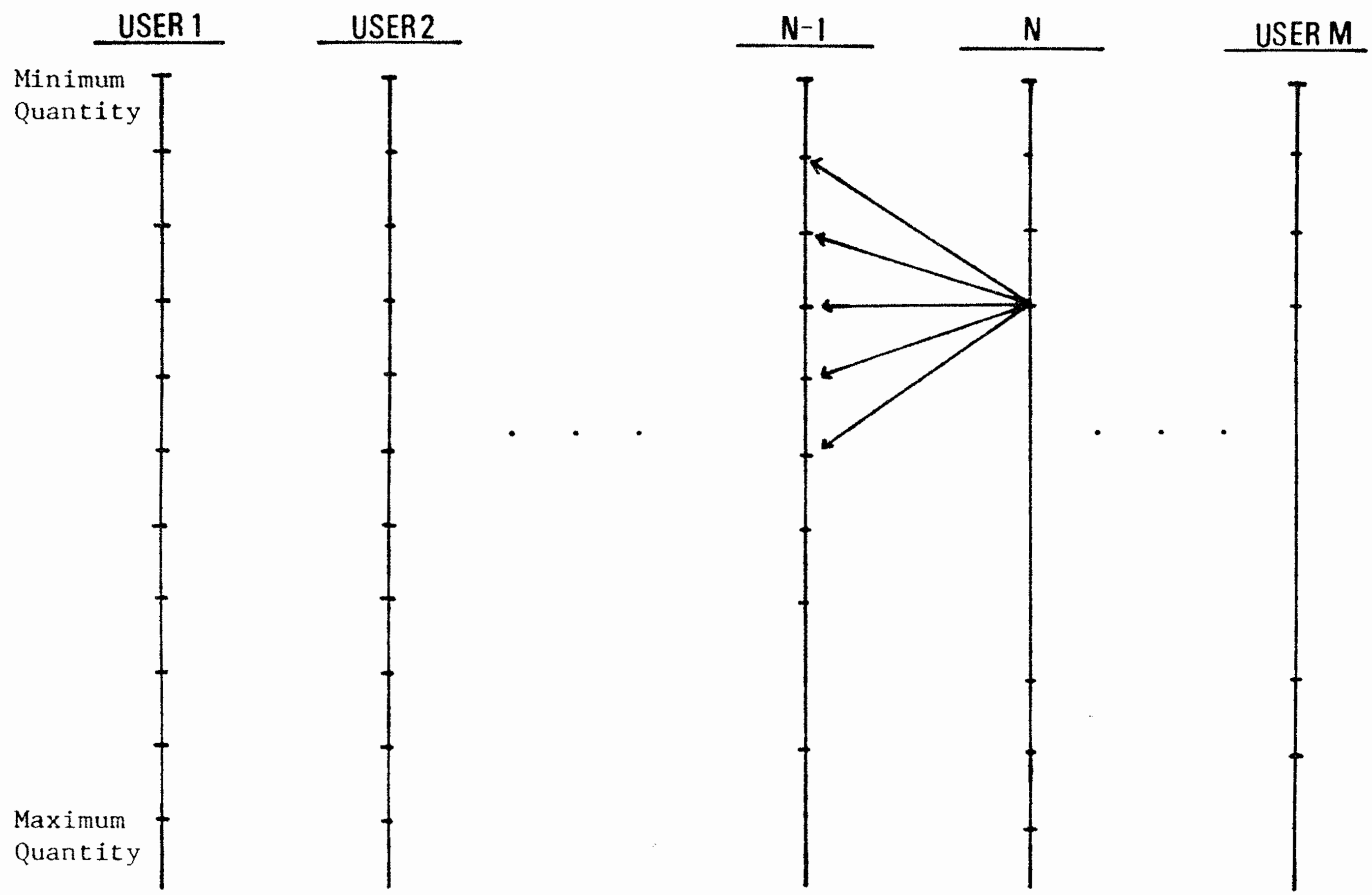


Figure 4.8 State Space for Allocation DP Model

$$D_{i,n} = S_{i-1,n} - S_{i,n} \tag{4.18}$$

where $D_{i,n}$, the decision variable, represents the actual amount of water allocated to user i under treatment alternative k in time interval j. All these allocation quantities are expressed as,

$$\underline{D}_{i,n} = \begin{bmatrix} D_{i,1} \\ D_{i,2} \\ \vdots \\ D_{i,N} \end{bmatrix} \tag{4.19}$$

The discretization of the water quantity increments is influenced by the range of demands of the user.

Algorithm--The dynamic programming computations start with the first secondary user and proceed stage by stage to the sink. On any stage i the DP computations consider different quantity allocations between the minimum and maximum acceptable demands of the secondary user. The costs, $f_{i-1}(S_{i-1,n})$ for each possible allocations, n=1,...N are determined for all stages i, for each treatment k in each year j.

The recursive equation equation for a feasible allocation alternative for each treatment k in each year j is

$$f_i(S_{i,n}) = \underset{D_{i,n}}{\text{Min}} \left[r_i(S_{i,n}, D_{i,n}) + f_{i-1}(S_{i-1,n}) \right] \begin{matrix} i=1,\ldots I \\ n=1,\ldots N \end{matrix} \tag{4.20}$$

in which $r_i(S_{i,n}, D_{i,n})$ represents the return for the particular allocation and treatment piping operation and maintenance costs, and $f_{i-1}(S_{i-1},n)$ represents the minimum cost of the allocation alternative that is connected to the unallocated quantity of treated wastewater for n_{i-1}. $f(S_{i,n})$ represents the cost (both the arbitrary and operation and maintenance costs) of the allocations that are connected to allocation n_i. Using the recursive equation (4.20) the allocation at n_{i-1} that represents the minimum cost, $f_n(S_{i,n})$, to allocation n_i is selected and stored for DP computations at stage n + 1. This procedure is possible through Bellman's Principle of Optimality (Nemhauser, 1966). The DP computations continue until each allocation n_i is considered at each i. The computational procedure continues until it applies the recursive equation to all i_n, i.e., the procedure is completed when stage I (the sink) and state N are reached.

Once the DP computations are complete a traceback is performed. The traceback is performed for each treatment alternative k and each year j to determine the minimum cost allocation alternative for the region. First the least cost allocation alternative is identified by comparing the sum of the weighted and operation and maintenance costs determined for each allocation alternative $-f_i(S_{i,n})$. The allocation to each secondary user is then identified from the allocation scheme identification index which defines the optimal stage to stage transitions. The traceback is complete when all states of the first secondary user have been considered. This traceback is performed for each treatment k, k = 1,2...K, in each time interval j, and for all years j, j = 1, 2...J.

Algorithm for Treatment Dynamic Program

The state space for the treatment DP model is illustrated in Figure 4.9. The applicable recursive equation is,

$$f_j(S_{j,k}) = \underset{D_{j,k}}{\text{Min}} \left[r_j(S_{j,k}, D_{j,k}) + f_{j-1}(S_{j-1,k}) \right] \quad \begin{matrix} j=1,2...J \\ k=1,2...K \end{matrix} \tag{4.21}$$

where j is the time interval used--the stages--and k represents the treatment alternative (which has an optimal allocation alternative associated with it) under consideration--the states. $f_j(S_{j,k})$ represents the total cost after the cost of the treatment chosen in time interval j is added. All possible costs (treatment and allocation alternative costs) to which the next stage's costs can be added are represented by

$$\underline{S}_{j,k} = \begin{bmatrix} S_{j,1} \\ S_{j,2} \\ \vdots \\ S_{j,k} \end{bmatrix} \tag{4.22}$$

The transformation function, then, is

$$D_{j,k} = S_{j-1,k} - S_{j,k} \tag{4.23}$$

where $D_{j,k}$, the decision variable, represents the cost of treatment alternative k in year j. These costs are expressed as

$$\underline{D}_{j-1,k} = \begin{bmatrix} D_{j,1} \\ D_{j,2} \\ \vdots \\ D_{j,k} \end{bmatrix} \tag{4.24}$$

Algorithm--The dynamic programming computations start with the first time interval and proceed stage by stage to the end of the planning period. On any stage j the DP computations consider different treatment alternatives. The costs $f_{i-1}(S_{i-1,n})$ for each treatment alternative are determined on the basis of the actual operation and maintenance costs and the capital costs (if need be) for each time interval j.

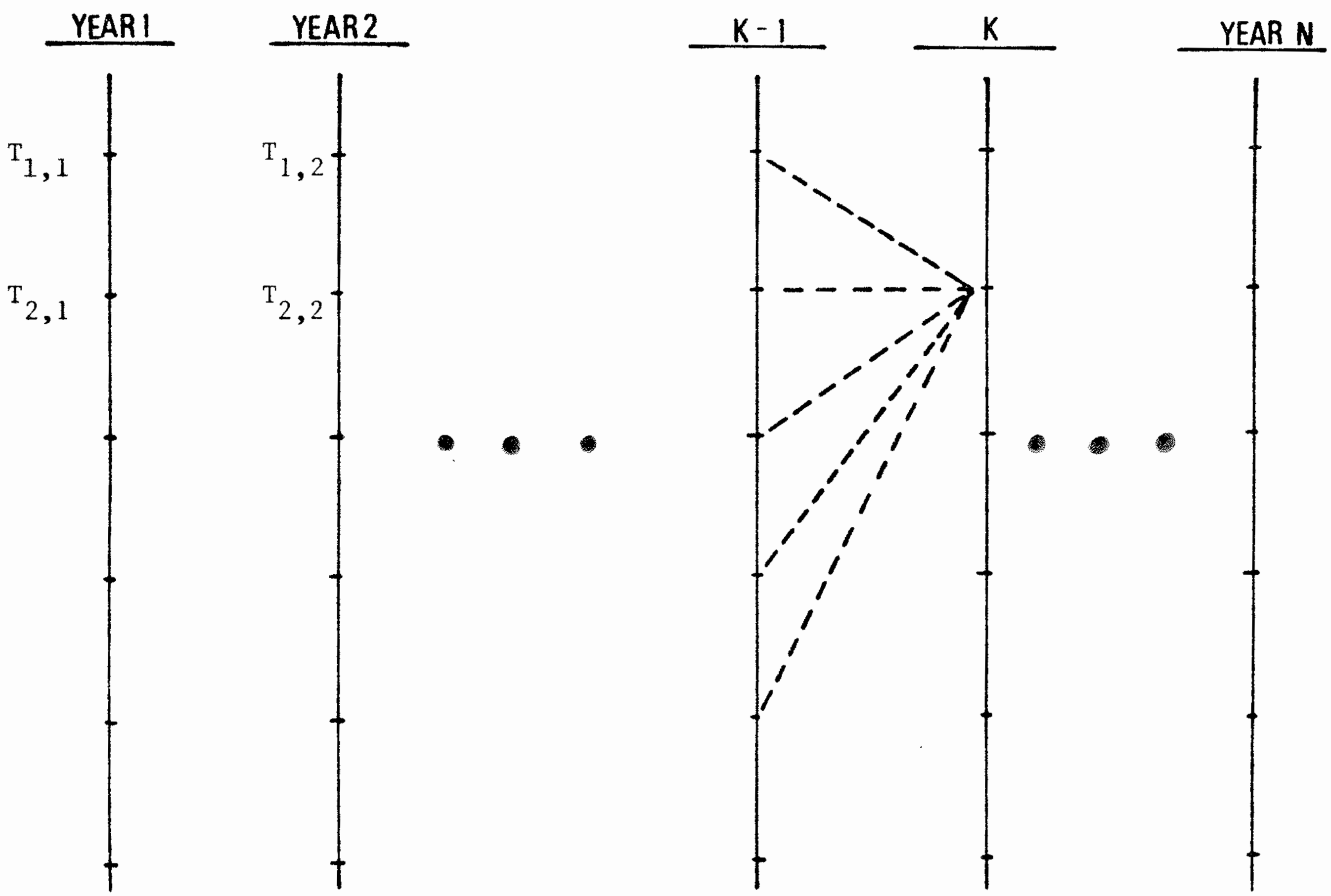


Figure 4.9 State Space for Treatment DP Model

The recursive equation for a feasible treatment scheme for the planning period is,

$$f_j\,(S_{j,k}) = \underset{D_{j,k}}{\mathrm{Min}}\ [\,r_j(S_{j,k},\,D_{j,k}) + f_{j-1}\ \ (S_{j-1,k})\,]\ \begin{matrix} j=1,\ldots J \\ k=1,\ldots K \end{matrix} \tag{4.25}$$

in which $r_j\,(S_{j,k},\,D_{j,k})$ represents the present value cost for the particular alternative considered and $f_{j-1}\,(S_{j-1,k})$ represents the minimum cost of the treatment and allocation alternative that is connected to the treatment and allocation scheme denoted at k_{j-1}. $f(S_{j,k})$ represents the present value cost (capital and operation and maintenance costs) of the treatment and allocation scheme that is connected to the treatment and allocation alternative at k_j. Using this recursive equation (4.25) the treatment and allocation alternative at k_{j-1} that represents the minimum cost, $f_j(S_{j,k})$, to treatment alternative k_j is selected and stored for DP computations at stage j + 1. The DP computations continue until each treatment and allocation alternative k_j is considered at each j. The computational procedure continues until it applies the recursive equation to all j_k, i.e., the procedure is complete when stage I (the sink) and state N are reached.

Once the DP computations are complete, a traceback is performed and the least cost treatment and allocation alternative is identified by comparing the costs determined at treatment and allocation alternative $f_j(S_{j,k})$. The allocation alternative and treatment alternative are then identified from the treatment scheme indices when defines the optimal stage to stage transitions. The traceback is complete when all states (treatment and allocation alternatives) of the last time interval have been considered. This traceback is performed for each planning period.

SECTION 5

APPLICATION OF THE DP MODELS: THE SAN ANTONIO CASE STUDY

5.1 INTRODUCTION

This section presents the real world application of the model outlined in the previous chapter to the city of San Antonio, Texas, and surrounding areas. The use of this model is shown for a planning period covering 20 years (1980-2000) with discrete time intervals represented yearly. Different treatment alternatives were considered including upgrading plants, abolishing old plants and building new ones. Different reuse schemes were also considered; the different schemes depending on secondary user needs and limitations.

The model was used to find solutions to the reuse allocation problem for each year and to find the least expensive treatment alternative or combination of alternatives. A description of the region, water demands in the region, the existing treatment facilities, proposed treatment facilities, existing and proposed allocation, and treatment schemes for the planning period are presented in this chapter.

5.2 REGION DESCRIPTION

The study area selected for application of this model includes the watersheds of Leon Creek, Salado Creek, and the upper San Antonio River. A detailed investigation is made of the southern third of Bexar Country and especially the southern third of the Section 201 Planning Area (see Figure 5.1).

The system elements considered in the planning process are:

1. City of San Antonio
2. Braunig Power Plant
3. Calaveras Power Plant
4. Alamo Irrigation Area
5. Rilling Road Irrigation Area
6. San Antonio River

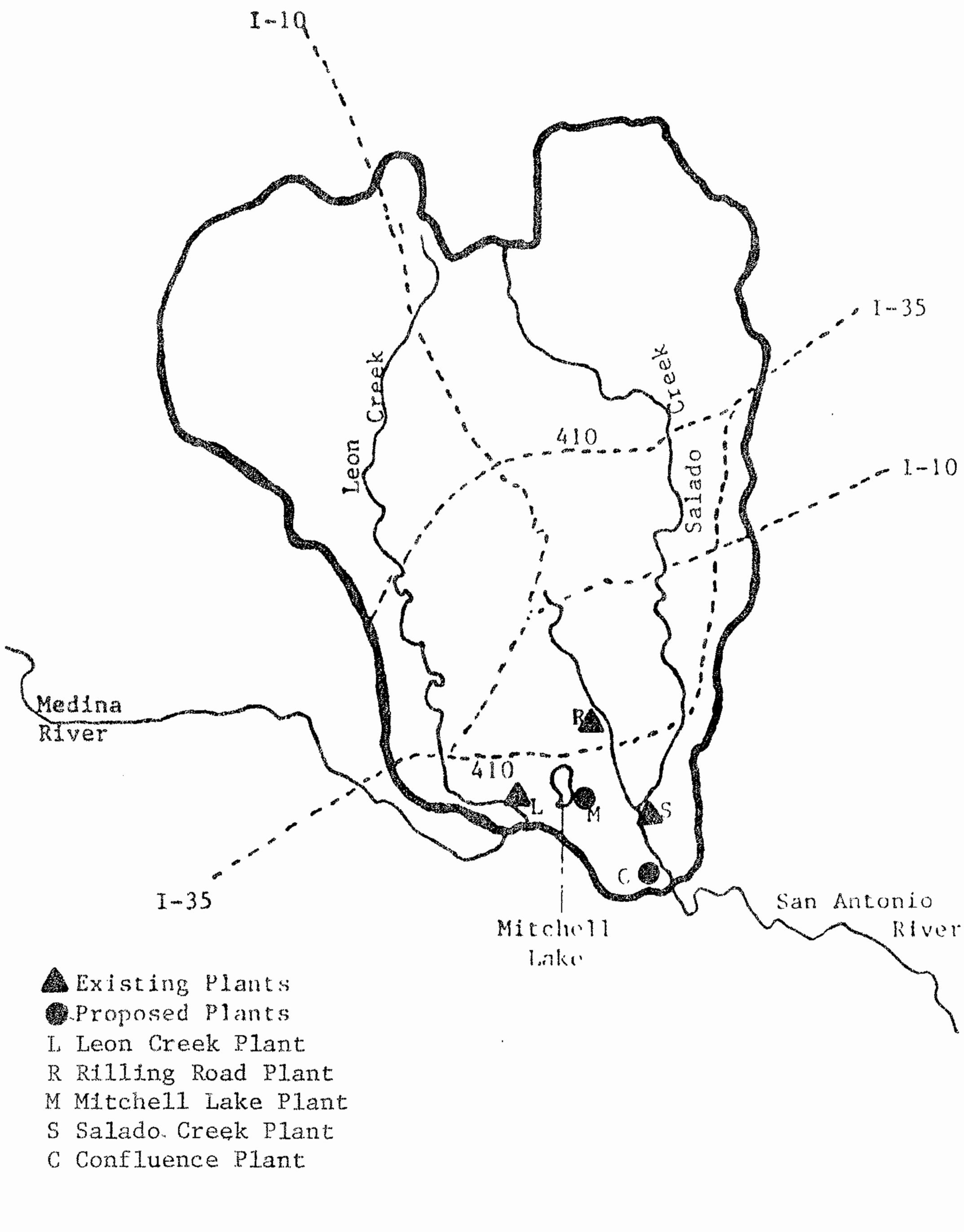


Figure 5.1 San Antonio 201 Planning Area

7. Rilling Road Sewage Treatment Plant

8. Salado Creek Sewage Treatment Plant

9. Leon Creek Sewage Treatment Plant

10. Confluence Sewage Treatment Plant

11. Mitchell Lake Sewage Treatment Plant

(1) Physiography

The physiography of Bexar County is dominated by the Balcones escarpment. The topography north of the escarpment is rugged and the slopes are steep. Soft, mixed, and hard limestone are predominant on the north side of the escarpment. In the south (i.e., where the study area is located) the topography is gently rolling. On the south side of the scarp, alluvium and clay dominate the gelology. The soil types, although varied, follow this north/south division. The soil north of the scarp is thin and rocky whereas south of the scarp it is deep and clayey or sandy in texture.

(2) Water Quality and Quantity

The water quality of the San Antonio River (whose headwaters are located at Olmos Creek) is poor. Often experiencing seven day - two year low flows of 11 cfs, the river receives the municipal wastewater discharge of the Rilling Road Sewage Treatment Plant. Although the Medina River empties into San Antonio River several miles south of San Antonio raising the volume of seven day - two year low flows to 53 cfs, the DO sag below the 83.4 MGD Rilling Road facility outfall does not meet the 5.0 mg/l DO standard and often does not recover for as far as 20 miles downstream.

The water quality of Leon Creek is also poor, experiencing seven day - two year low flows of 0.0 cfs and receiving municipal wastewater discharge from the Leon WWTP and industrial waste discharge from Kelly Air Force Base. During low flow the wastewater discharge comprises the entire flow of Leon Creek. The average discharge rate from these plants and thus the flow during low flow conditions is 17.5 MGD. Leon Creek's base flow is greatly affected by recharge in the Edwards Aquifer zone. The DO concentration falls below 5 mg/l daily during low flow periods.

The water quality of Salado Creek on the other hand, is generally good. The 16.6 MGD discharge into the creek is dominated by small package plants. Surface runoff also adds to the flow. The aquatic habitat of the southern reach of this creek is of good enough quality to provide a locally heavy fishing pressure load.

(3) San Antonio

The population of San Antonio in 1975 was over 770,000 and, the city is projected to have nearly 1.1 million people by the year 2000. The inner areas of the city are more densely populated than the suburban fringes. Rapid growth is occurring to the north of the city whereas the inner city population seems to have stabilized. Residential land use also dominates the southern part of the city with some industrial development and a military installation. Growth in the southern area is expected to be minimal.

The Edwards Aquifer is the only source of drinking water for the city of San Antonio and for the nearby military installations, industrial users, irrigation areas and for domestic and livestock purposes in Bexar County. As the population increases withdrawal from the aquifer will increase greatly. Depletion of the aquifer has already occurred in the drought period of 1947-1956. Depletion, where discharge exceeds the recharge rate over the long term, if not regulated, could occur especially if withdrawal by the City of San Antonio is not limited to 285,000 ac-ft/year.

5.3 SYSTEM ELEMENTS DESCRIPTION

Water Users

(1) City of San Antonio

The City of San Antonio was the only primary user considered in this case study. The BOD concentration of the wastewater effluent from the City of San Antonio was considered constant throughout the planning period at 262 mg/l. The wastewater flows produced by San Antonio were considered the influent flows to the treatment plants under consideration.

Projected wastewater flows and losses for the 20 year planning period were obtained from the the Texas Deparment of Water Resources (see Table 5.1). A linear extrapolation was made to convert this data into projected yearly demands (See Table 5.1).

(2) Calaveras Power Plant

The Calaveras Power Plant is considered a secondary user and demands large amounts of water for cooling purposes. Located southeast of San Antonio, this power plant can recycle its own water through the use of a cooling pond. Water from Calaveras Creek diverted to the plant supplements the power plant's water demands. Therefore, quantity demands placed on the water considered for reuse are not very high. These demands, obtained from TDWR are listed in Table 5.1. Again a linear extrapolation was made from this data to provide yearly demands for modeling uses.

It is assumed that the BOD level of the treated wastewater allocated to the plant for reuse is not affected by the plants usage--therefore treatment prior to discharge from the Calaveras Plant is not needed. It is also assumed that the quantity discharged to the San Antonio River is small since much of the power plant's water is recycled or consumed.

Table 5.1 Quantity and Quality Demands

User	Needs (MGD)	Quantity Losses (MGD)	Recycles (MGD)	Maximum BOD Acceptable (mg/1)
City of San Antonio	10685.9			
Calaveras Power Plant	25193.0	425.5	25514.0	15.
Braunig Power Plant	565.7	6.25	559.7	10.
Rilling Road Irrigation Area	4.0	4.0	-	10.
Alamo Irrigation Area	21.43	21.43	-	10.
Sink (San Antonic River)	-	-	-	32.

Wastewater Treatment Plants	Capacity MGD	Average BOD Removal Efficiency Percent
Leon Creek	24	92.1
Rilling Road	93.5	93.7
Salado Creek	24	88.5
Confluence (proposed)	154	98.1 (proposed)
Mitchell Lake	118	

(3) Braunig Power Plant

Also located southeast of San Antonio and requiring large amounts of water is the Braunig Power Plant. This secondary user has a cooling pond which allows recycling of its water, demands water having a maximum allowable BOD concentration of 10 mg/1, and does not produce a significant increment to the BOD concentration.

Projected water demands and losses for this plant are listed in Table 5.1 and have been linearly extrapolated to project yearly demands for model usage. This power plant's demand can be supplemented through a direct diversion of water from the San Antonio River if there is not enough treated water for reuse to meet the plants demands.

(4) Rilling Road Irrigation Area

This area is actually a partition from the Alamo Valley Irrigation zone. This area has a maximum acceptable BOD concentration level of 10 mg/1. The projected demands as provided by TDWR are listed in Table 5.1 and are been linearly extrapolated to yearly values. It is assumed that water provided to this irrigation area is entirely lost from the system.

(5) Alamo Irrigation Area

This secondary user makes up the rest of the Alamo Valley Irrigation zone. Its quantity demands are listed in Table 5.1. The area has a maximum acceptable BOD concentration level of 10 mg/1. Again it is assumed that water provided to this irrigation area is lost from the system.

Existing Wastewater Treatment Plants

Effluent water from any of the wastewater treatment plants (WWTP) considered by a particular treatment alternative can be allotted to any user or to the sink (the San Antonio River). Capacity expansion is not considered within any of the alternatives. Each alternative is represented at its maximum capacity and efficiencies although one alternative might be a capacity/efficiency modification of another alternative.

(1) Rilling Road Treatment Plant

Located south of San Antonio, the Rilling Road WWTP is very old. It has undergone many capacity expansions since its construction in 1930. Its current capacity is 93.5 MGD. This treatment plant provides preliminary treatment, primary settling, modified activated sludge treatment (reaeration of activated sludge before returned for recycle), secondary clarification and chlorination.

In 1976, the average influent BOD was 201 mg/1 and the average effluent was 12.6 mg/1 for an average removal efficiency of 93.7%. The overall efficiency for 1976 was estimated at 90.3% and is assumed to be declining as a greater quantity of water passes through the plant and no alternations in the treatment processes are made.

(2) Leon Creek Treatment Plant

Also located south of San Antonio, the Leon Creek WWTP currently has a capacity of 24 MGD. The Leon Creek WWTP provides preliminary treatment, primary settling, modified activated sludge treatment, secondary clarification and chlorination. In 1976, its overall BOD removal efficiency was rated at 92.1%. The influent wastewater to this plant is from San Antonio and effluent not allocated for reuse is discharged into Leon Creek.

(3) Salado Creek Treatment Plant

A relatively new treatment plant, the Salado WWTP was constructed southeast of San Antonio with a capacity of 24 MGD and a BOD removal efficiency of 88.5%. This facility provides preliminary treatment, primary settling, modified activated sludge, secondary clarification, and chlorine contact. Water that is not allocated for reuse is discharged into the San Antonio River.

Proposed Wastewater Treatment Plants

(1) Confluence Treatment Plant

In one of the alternatives explained in the next section a new element addition to the existing treatment and distribution system is a proposed wastewater treatment plant to be located at the confluence of the Medina and San Antonio Rivers. This plant would provide treatment to the tertiary level including: preliminary treatment, primary treatment, two stage biological activated sludge treatment, filtration and disinfection.

Wastewater will enter the plant in two stages. At design capacity, 83 MGD of raw wastewater from the City of San Antonio can be treated through the first stage of the activated sludge facilities. Following the intermediate clarifiers, this partially treated wastewater will be combined with secondary treated water from the Leon and Salado Creek plants for treatment to a tertiary level. Operation design capacity for the latter process is 154 MGD.

The BODs removal efficiency expected for this treatment plant is 98.1%. All water that is not allocated for reuse from this plant will be discharged into the San Antonio River, 1,200 linear feet upstream from the confluence.

(2) Mitchell Lake Treatment Plant

Although detailed information concerning this proposed plant was not readily available, this new facility near Mitchell Lake would discharge tertiary treated effluent to the Medina River, (EIS, pg. 3-33). It would receive wastewater from the City of San Antonio and secondary treated wastewater would be transported for tertiary treatment from the Leon Creek Plant. It was assumed that 95 MGD would be treated through the secondary process at the Mitchell Plant (that water no longer treated by the Rilling Road Plant) and the total capacity of the plant with tertiary treatment would be 118 MGD.

5.4 TREATMENT ALTERNATIVES CONSIDERED

Five treatment alternatives were chosen for consideration in this model. These alternatives were chosen from a total of 26 possible treatment alternatives presented in the EIS, Environmental Impact Statement, and from the seven alternatives that were selected after screening of the original 26 by TDWR and EPA. A summary of the capacity and treatment level changes proposed can be found in Table 5.2.

(1) Treatment Alternative 1 (Equivalent to "No Action" in the EIS)

This alternative suggests that no improvements on the existing system should be made. Current violations of their discharge permits would continue and future growth in the area would be accommodated by septic tanks. The schematic of this alternative is depicted in Figure 5.2-a.

(2) Treatment Alternative 2 (matches 2A in the EIS)

This alternative also supports the idea of keeping the existing facilities in place, but improvements on the system are also suggested. They are:

a. Maintain the Rilling Road WWTP at its current efficiency and capacity but transport its secondary effluent to the Salado Creek Plant.

b. The Salado Creek Plant would be modified to have an expanded capacity and its treatment process would be improved to the tertiary level. Discharge from this plant would occur at the existing outfall.

c. The Leon Creek Plant would undergo capacity expansion and improvement of its treatment process to the tertiary level would also be required. Discharge would be from the existing outfall.

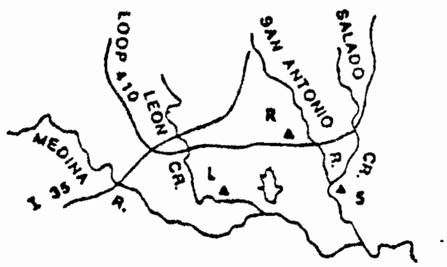
This alternative involves much rehabilitation and replacement work. Rilling Road WWTP, built in the 1930's, would need new headworks, a new electrical system, new primary and final clarifier mechanisms, a new aeration system for part of the plant, better odor control facilities, new digestor covers and controls, rehabilitation of the return sludge pumping system, a new in-plant water distribution system, and new equalization facilities to handle peak loads. A new storage and maintenance building would also be needed.

Nitrification facilities using synthetic media in the trickling filters are required for the Leon and Salado Creek Plants so discharges will meet

Table 5.2 Summary of Capacity and Treatment Level Changes Considered by the Model

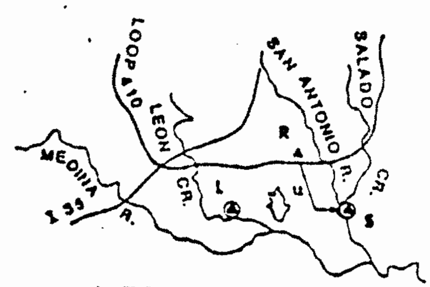
Wastewater Treatment Plant	Type of Action Taken	Alternatives 1	2	3	4	5
Rilling Road Treatment Plant	Capacity Expansion (MGD)	None	Secondary effluent to Salado Plant	Abandon Treatment Plant	Abandon Treatment Plant	Abandon Treatment Plant
	Process Improvement	None	Complete Renovation			
Leon Creek Treatment Plant	Capacity Expansion (MGD)	None	24 to 35	24 to 35	24 to 35	24 to 35
	Process Improvement	None	Tertiary treatment added	Tertiary treatment added	Secondary effluent to Confluence Plant	Secondary effluent to Mitchell Lake Plant
Salado Creek Treatment Plant	Capacity Expansion (MGD)	None	24 to 36 Secondary	24 to 119[a]	24 to 36	24 to 36
	Process Improvement	None	Tertiary treatment added to 119 MGD	Tertiary treatment added	Secondary effluent to Confluence Plant	Tertiary treatment added
Confluence Plant	Capacity Expansion (MGD)				Raw Treatment (83) and Secondary (154) Treatment to tertiary level	
	Process Improvement					
Mitchell Lake Treatment Plant	Capacity Expansion (MGD)					Raw treatment (83) and Secondary (154) Treatment to tertiary level
	Process Improvement					

[a]Raw water from Rilling Road plant would be rerouted to Salado plus existing Salado raw water equals 119 MGD.



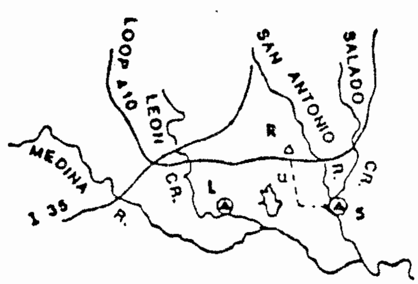
NO ACTION ALTERNATIVE

ALTERNATIVE 1
a



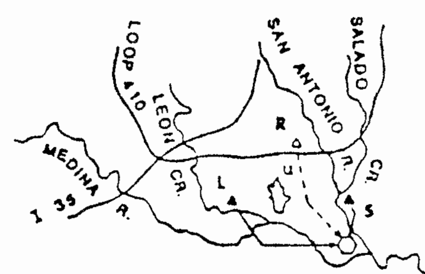
ALTERNATIVE 2A

ALTERNATIVE 2
b



ALTERNATIVE 2B

ALTERNATIVE 3
c

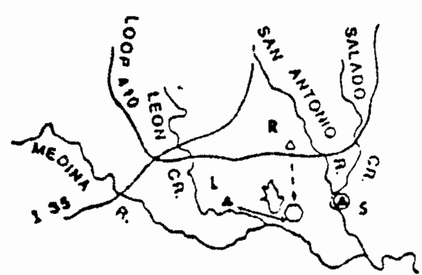


ALTERNATIVE 3B

ALTERNATIVE 4
d

LEGEND

- ▲ EXISTING TREATMENT PLANT
- △ ABANDONED TREATMENT PLANT
- O PROPOSED TERTIARY TREATMENT PLANT
- O PROPOSED NEW TREATMENT PLANT
- ——→ SECONDARY EFFLUENT PIPELINE
- - - -→ RAW SEWAGE PIPELINE
- L LEON CREEK PLANT
- R RILLING ROAD PLANT
- S SALADO CREEK PLANT
- b PIPELINES IN BOTTOMLANDS
- u PIPELINES IN UPLANDS



ALTERNATIVE 4B

ALTERNATIVE 5
e

Figure 5.2 Treatment Alternatives for the San Antonio Region permit standards. The schematic diagram of this alternative is depicted in Figure 5.2-b

(3) Treatment Alternative 3 (Matches 2B in the EIS)

Alternative 3 requires that the Rilling Road WWTP be abandoned and that the Salado and Leon Creek WWTP be expanded and their treatment processes be improved to a tertiary level to handle the increasing amounts of wastewater flow. Discharges would occur at the existing outfalls. All water previously routed to the Rilling Road Plant would be rerouted through transfer lines to the Salado Creek Plant. The schematic diagram of this alternative is depicted in Figure 5.2-c.

(4) Treatment Alternative 4 (3B in the EIS)

This treatment alternative also mandates that the Rilling Road WWTP be abandoned and that a new WWTP be built at the confluence of the Medina and San Antonio Rivers. This new plant would be equipped with facilities to provide treatment up to the tertiary level, including preliminary treatment, primary treatment, two-stage biological activated sludge, multi-media filtration and disinfection. This new facility would discharge into the San Antonio River, have a raw wastewater treatment capacity of 83 MGD, and be able to treat 154 MGD from a secondary to tertiary level.

The Leon and Salado plants would undergo capacity expansion for treatment of all wastewater to a secondary level and then this water would be transfered to the confluence facility for tertiary treatment. The raw wastewater previously routed to the Rilling Road Plant would be rerouted to the confluence plant and treated. A schematic of this alternative is shown in Figure 5.2-d.

(5) Treatment Alternative 5 (matches 4B in the EIS)

Abandonment of the Rilling Road Plant and addition of a new plant facility located near Mitchell Lake is suggested by this alternative. The Leon Creek Plant would be capacity expanded and its secondary effluent would be transferred to the Lake Mitchell WWTP for tertiary treatment.

The Salado Creek Plant would not only be expanded, but facilities to treat wastewater to the tertiary level would be added. Wastewater from this plant would be discharged at the existing outfall. A schematic of this alternative is shown in 5.2-e.

5.5 COST FUNCTIONS FOR CASE STUDY

Cost Functions

This planning model minimizes the total cost of the planning period through finding the minimum cost for each year and then summing the costs over all of the years. The costs are discounted so as to be represented in present value dollars. A single payment factor was used to discount capital, operation and maintenance costs.

Included in the operation and maintenance costs are treatment and piping operation and maintenance. Included in the capital costs are new construction costs (if any) and capacity expansion costs. New pipelines and pumping apparatus are required to provide for the changes in the system. It was therefore assumed that pipe construction costs would be uniform for alternatives 2-5 so these costs were not considered in this model application.

The following cost equations were selected from Ocanas and Mays (1980).

(1) Capital Costs

Treatment plant construction cost equations for both new construction and capacity expansion of plants were needed for the evaluation of alternatives. The EPA has gathered significant amounts of cost data for construction and expansion of plants in the U.S., from which Ocanas and Mays developed equations for the San Antonio area.

For new plants to be constructed to the secondary level of treatment the cost equation used is,

$$\text{Construction Cost (Million \$)} = 2.88Q_j^{.99} \qquad (5.1)$$

The cost equation for capacity expansion of a secondary plant is

$$\text{Capacity Expansion Cost (Million \$)} = 2.25 \quad Q_j^{.890} \qquad (5.2)$$

If a plant's treatment efficiency had to be improved to a tertiary treatment level, the capital cost equation for the construction of pressure diatomite filters was used, ie.

$$\text{Construction Cost (Million \$)} = 1.5\ Q_j^{.81} \qquad (5.3)$$

These capital costs were added directly into the objective function the year the plant was placed on line. The capital cost was not divided through the life of the plant because this model depicts actual yearly costs before the development of a pay back scheme for capital costs, i.e., before the issuance of a municipal bond. Based on this information a pay back scheme can be devised.

(2) Operation and Maintenance

Piping operation and maintenance costs as well as treatment operation and maintenance costs were considered here. Pumping operation and maintenance costs were not considered here due to lack of information concerning total design head for the pipelines. Piping costs were estimated on the basis of approximate distances between treatment plants and users only. Costs for piping between treatment plants were not estimated due to lack of information on the proposed placement and sizing of the pipelines.

Treatment operation and maintenance costs for secondary treatment-activated sludge facility were evaluated via the following equation,

$$\text{Wastewater Treatment Cost (Million \$/Year)} = .825\ Q_j^{.96} \tag{5.4}$$

This cost equation probably overestimates the real cost of the treatment alternative's operation and maintenance costs since it estimates them on the basis of total flow in year j for a treatment alternative versus estimation based on the flow through each plant considered in the alternative.

For a treatment alternative that treats water to the tertiary level the following operation and maintenance cost was added

$$\text{Water Treatment Cost (Million \$/Year)} = .015\ Q_j^{.798} \tag{5.5}$$

Operation and maintenance on piping was estimated for piping between the treatment plants and the users only. These cost functions varied with the secondary user distance from the WWTP providing the water for reuse. The generalized equation is,

$$\text{Piping Cost (Million \$/year)} = 4.56 \times 10^{-3} \times \text{Distance (mi)}\ \ QALOC_{j,i,k}^{.495} \tag{5.6}$$

In most cases the treatment plant closest to the user was chosen to represent the distance (except in the case where quality constraints would not allow the wastewater to be transfered) and therefore the cost function representing this distance described the operation and maintenance costs. These cost functions are specified for each treatment alternative since different plants are used in each alternative.

Cost Functions for Alternatives

(1) Alternative 1 Costs

Since Alternative 1 suggests that no improvements be made on the current system, no costs were computed for this alternative.

(2) Alternative 2 Costs

As explained in the previous section, Alternative 2 requires a major renovation of the Rilling Road Plant and capacity expansion and treatment improvement of the Leon and Salado plants. Since such major renovation was required for the Rilling Road plant the capital cost for this renovation was determined on the basis of the new construction cost equation. The capital construction costs for treatment Alternative 2 are listed in Table 5.3.

The estimates presented in this example problem are not construed as the final cost estimates for the construction/operation of a particular alternative. For this example case they are used as a valid basis from

TABLES 5.3 COSTS COMPUTED FOR ALTERNATIVE 2

CAPITAL CONSTRUCTION

1.	Rilling Road renovation	$2.88(94)^{.99}$	=	258.70
2.	Salado Creek			
	Capacity expansion	$2.25(12)^{.890}$	=	20.54
	Treatment improvement	$1.53(130)^{.810}$	=	77.34
3.	Leon Creek			
	Capacity expansion	$2.25(11)^{.810}$	=	15.69
	Treatment improvement	$1.5(35)^{.81}$	=	26.72
	Total capital cost			$398.99

PIPING OPERATION AND MAINTENANCE

Symbol	From	To	Distance	Cost Function
SC	Salado	Calaveras P.P.	3	$.0137\ (QALOC_{j,2,1})^{.495}$
SB	Salado	Braunig P.P.	8	$.0365\ (QALOC_{j,2,2})^{.495}$
SR	Salado	Rilling Road Irr.	1.5	$.0068\ (QALOC_{j,2,3})^{.495}$
SA	Salado	Alamo Valley Irr.	6	$.0274\ (QALOC_{j,2,4})^{.495}$

TREATMENT OPERATION AND MAINTENANCE

SL	Secondary Level	$.825Q_j^{.960}$
ST	Tertiary Level	$.015Q_j^{.798}$

TABLE 5.4 CAPITAL COSTS FOR ALTERNATIVE 3

Salado Creek			
Capacity expansion	$2.25(95)^{.890}$	=	129.53
Treatment Improvement	$1.5(119)^{.810}$	=	71.99
Leon Creek			
Capacity expansion	$2.25(11)^{.890}$	=	19.01
Treatment improvement	$1.5(35)^{.810}$	=	26.72
Total Capital Costs			$247.25

which a comparison of costs among the alternatives can be made and a financing scheme can be developed.

Operation and maintenance costs for Alternative 2 have been estimated and are listed in Table 5.3. Operation and maintenance costs are also listed in Table 5.3. Total annual operation and maintenance costs are

$$OM_{j,k} = SL + ST + SC + SB + SR + SA \tag{5.7}$$

This cost will also be estimated at a higher rate than the EIS estimates. As mentioned previously the operation and maintenance treatment costs are estimated on the basis of total flow, not individual flows through the plants. Also, the EIS does not include water reuse possibilities in its cost estimation. Piping costs from plants to secondary users were also not considered in the EIS estimations of O & M costs.

(3) Alternative 3 Costs

The capital costs for this alternative represent the capacity expansion and treatment improvement of both the Leon and Salado Creek plants but do not consider the piping construction costs for the transfer line from the abandoned plant to the Salado Creek Plant. Capital costs for this alternative are listed in Table 5.4. Again, this is a much higher estimate than that provided by the EIS of 1976.

Operation and maintenance costs for this alternative are the same as in Alternative 2, Eq. (5.7). This is assumed for two reasons. First, the Leon Creek Plant; not as close to the secondary users as the Salado Creek Plant is and second, operation and maintenance costs are considered on a total flow basis, not on an individual basis. It must also be mentioned that this ensures a discharge and therefore a flow in Leon Creek during periods of low flow.

(4) Alternative 4 Costs

The capital costs in this treatment alternative include construction of a treatment plant at the confluence of the Medina and San Antonio Rivers. This plant would treat wastewater through a tertiary level process. The capital costs also include capacity expansion of the Leon and Salado Creek Plants but no treatment improvements are made as this water is transferred to the confluence plant. Capital costs are listed in Table 5.5.

The operation and maintenance costs for this alternative are different from the other alternatives considered so far. The assumption that operation and maintenance costs are the same since they are based on total flow is probably not an accurate assumption but the piping and maintenance costs are more realistic.

Treatment operation and maintenance costs are probably an overestimation of the actual costs as listed in Table 5.5. Total operation and maintenance costs for this alternative are,

TABLE 5.5 COSTS FOR ALTERNATIVE 4

CONFLUENCE PLANT (NEW CONSTRUCTION)

Secondary Level	$2.88\ (93)^{.99}$	=	288.71
Diatomite Filters	$1.5(154)^{.81}$	=	88.71
Leon Creek Plant Capacity expansion	$2.25(11)^{.89}$	=	19.01
Salado Creek Plant Capacity expansion	$2.25(12)^{.89}$	=	20.54
Total Capital Cost			\$356.97

PIPING OPERATION AND MAINTENANCE

Symbol	From	To	Distance	Cost Function
CC	Confluence	Calaveras P.P.	1.5*	$.0068\ (QALOC_{j,4,1})^{.495}$
CB	Confluence	Braunig P.P.	2.	$.0091\ (QALOC_{j,4,2})^{.495}$
SR	Salado	Rilling Road Irr+	1.5	$.0068\ (QALOC_{j,4,3})^{.495}$
CAI	Confluence	Alamo Valley Irr	3.	$.0137\ (QALOC_{j,4,4})^{.495}$

TREATMENT OPERATION AND MAINTENANCE COSTS

SL	Secondary Level	$.825\ Q_j^{.960}$
ST	Tertiary Level	$.015Q_j^{.795}$

*The actual distance is approximately 7 miles but the San Antonio River is used to convey the water.

+Although the BOD may not meet the Rilling Road Irrigation area's water quality demands when heavy pollutant loads are found in the San Antonio wastewater (e.g., due to seasonal variation), it is assumed that secondarily treated wastewater from the Salado WWTP is of high enough quality to be provided for reuse by the Rilling Road Irrigation area.

$$OM_{j,k} = SL + ST + CC + CB + SR + CAI \quad (5.8)$$

(5) Alternative 5 Costs

Alternative 5 also requires the abandonment of the Rilling Road WWTP and the addition of a new tertiary level wastewater treatement plant located at Mitchell Lake. The secondary effluent from a capacity expanded Leon Creek WWTP would be transferred to this new Mitchell Lake plant. The Salado Creek WWTP would be expanded and its treatment level raised. The capital costs for this treatment alternative are listed in Table 5.6.

The operation and maintenance cost equations are derived from rough estimates of the distance between the secondary users and the wastewater treatment plant at Mitchell Lake. The maps used to obtain these distances were (Figure 4.1 EIS, pg. 3-5) and from (Ocanas and Mays, Figure 6.1, pg. 205). The costs are listed in Table 5.6. So that the total annual operation and maintenance costs for this treatment alternative are

$$OM_{j,k} = SL + ST + MB + MC + MR + MA \quad (5.9)$$

5.6 MODEL STRUCTURE AND APPLICATION

Model Structure

In order to model this region the secondary users are numbered continuously starting with the sink; the receiving waters at the waste water treatment plant outfall are considered the final user. The potential to install a minimum quantity and quality demand for the final user is built into the model, but only the minimum quality demand is used in this example. If users downstream have Riparian rights to water that might be interfered with by a water reuse scheme or a particular water treatment alternative the potential to consider a minimum quantity demand has been built into the model structure and can be used.

Figure 5.3 represents the case study considered as a number of networks. Treatment alternatives 1-3 represent a simple "black box" treatment and allocation scheme whereas alternatives 4-5 are slightly more complex treatment alternatives and allocation schemes.

Solution Strategy

The solution strategy used for the application of this dynamic programming model consists of the following steps:

1. Numbering of the regional secondary users (see Table 5.7)

2. Numbering of the treatment alternatives

TABLE 5.6 COSTS FOR ALTERNATIVE 5

MITCHELL LAKE PLANT (NEW CONSTRUCTION)

Secondary treatment	$2.88(83)^{.99}$	=	228.71
Diatomite filters	$1.5(118)^{.81}$	=	71.50
Leon Creek			
Capacity expansion	$2.25(11)^{.89}$	=	19.01
Salado Creek			
Capacity expansion	$2.25(12)^{.89}$	=	20.54
Diatomite filter	$1.5(36)^{.81}$	=	27.33
Total Capital Cost			$367.10

PIPING OPERATION AND MANITENANCE

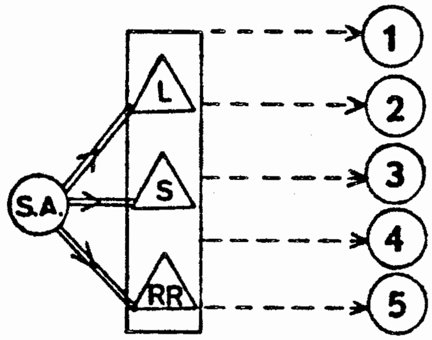
Symbol	From	To	Distance	Cost Function
MB	Mitchell	Calaveras P.P.	12	$.0548\ (QALOC_{j,5,1})^{.495}$
MC	Mitchell	Braunig P.P.	7	$.0319\ (QALOC_{j,5,2})^{.495}$
MR	Mitchell	Rilling Road Irr.	3	$.0137\ (QALOC_{j,5,3})^{.495}$
MA	Mitchell	Alamo Valley Irr.	3	$.0137\ (QALOC_{j,5,4})^{.495}$

TREATMENT AND MAINTENANCE COSTS

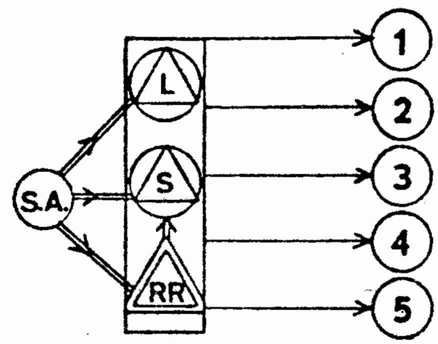
SL	Secondary Level	$.825\ Q_j^{.96}$
ST	Tertiary Level	$.015\ Q_j^{.798}$

TABLE 5.7 SECONDARY USER NUMBERING FOR THE SAN ANTONIO REGION

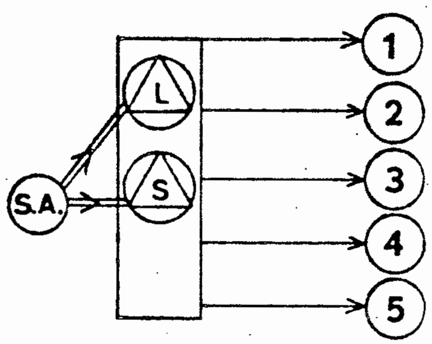
1. Calaveras Power Plant
2. Braunig Power Plant
3. Rilling Road Irrigation Area
4. Alamo Valley Irrigation Area
5. San Antonio River (Sink)



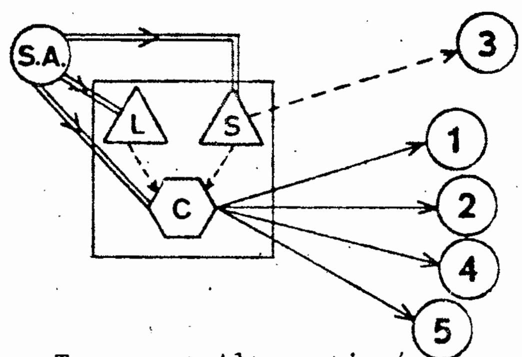
Treatment Alternative 1



Treatment Alternative 2

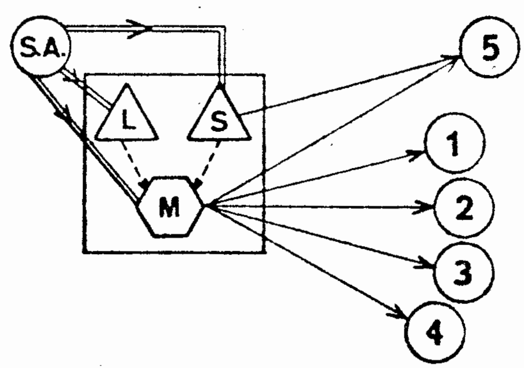


Treatment Alternative 3



Treatment Alternative4

Existing Treatment Plant
Abandoned Treatment Plant
Proposed Tertiary Treatment Plant
Proposed New Treatment Plant
C Confluence Plant
L Leon Creek Plant
M Mitchell Lake Plant
R Rilling Road Plant
S Saledo Creek Plant
Secondary Effluent Pipeline
Raw Sewage Pipeline



Treatment Alternative 5

Secondary Users

1. Calaveras Power Plant
2. Braunig Power Plant
3. Rilling Road Irrigation Area
4. Alamo Valley Irrigation Area
5. San Antonio River (sink)

Figure 5.3 Treatment Alternatives for Case Study

3. Application of Data Preparation Model

This involves the use of linear regression equations to generate quantity and quality data representing source (the City of San Antonio) inputs and user demands. Tables 5.1 shows the actual source inputs, user demands and where needed. Treatment efficiencies also had to be determined via piecewise linear regression techniques.

4. Application of the Allocation DP

The allocation DP as outlined in Chapter 4 is applied using the input file described above. Figure 4.7 outlines the application of the allocation DP model.

5. Application of Treatment DP Model

From the solutions to the allocation DP a final solution to the problem is found through the application of the Treatment DP model outlined in Chapter 4. The data preparation model, the allocation model, and the treatment model are all programmed as one computer code so that steps 3, 4, and 5 are performed essentially as one step, the application of the computer model.

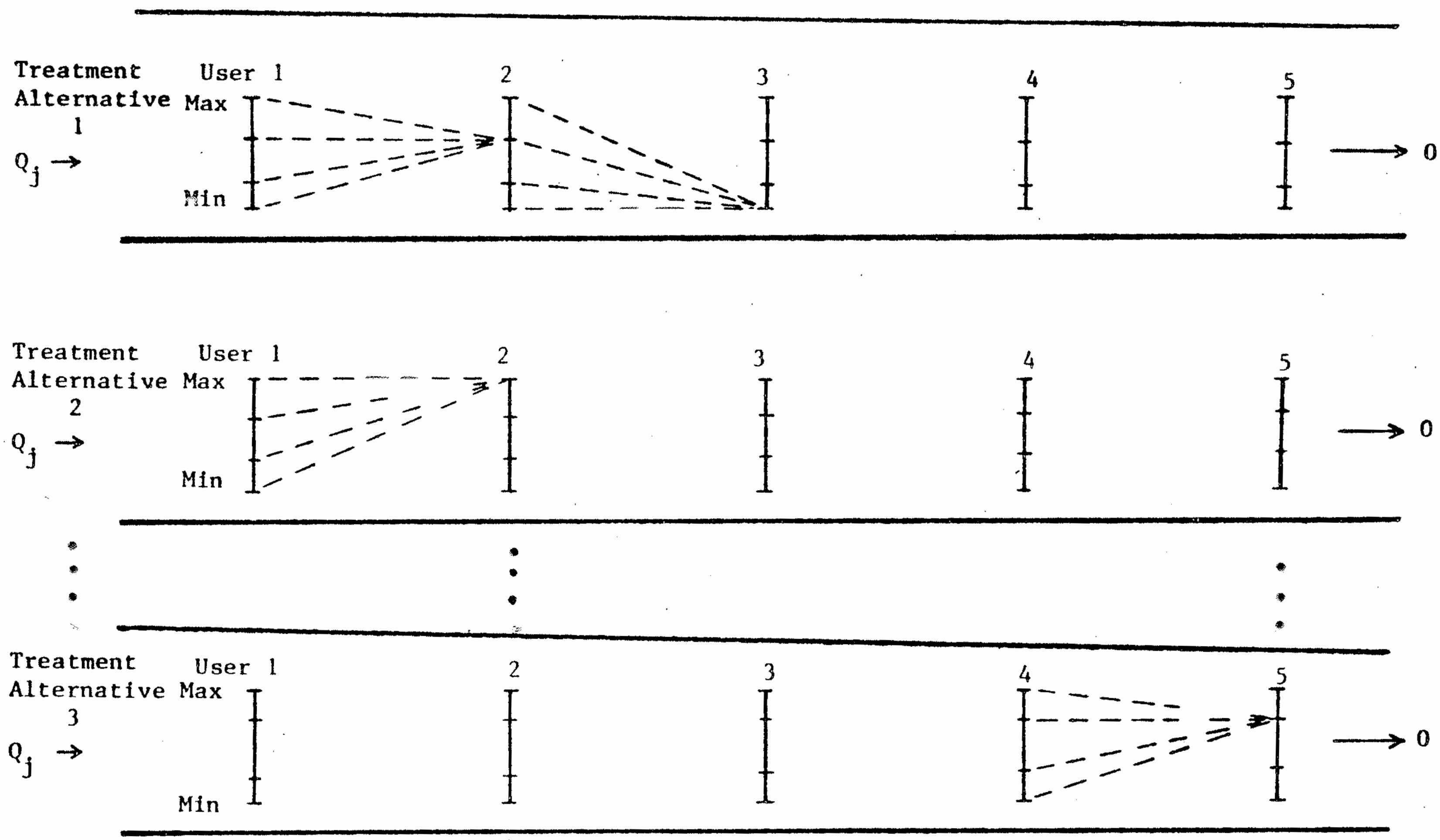
6. Analysis of Policy Implications

An analysis of the public policy implications can be performed for the final solution obtained.

Case Study

This study considers the San Antonio area from the years 1980-2000. It consists of one source, five treatment alternatives with 2-3 treatment plants, four secondary users, and one sink. The characteristics of the source, the users and the sink have been explained in the previous section. The feasible allocation schemes and treatment characteristics are delineated in the following section.

Application of Allocation DP--Figure 4.4 presents a schematic of the allocation DP problem solved for an arbitary year j. Treatment alternatives 1 through 5 (Figure 5.2) are considered. For each treatment alternative the state space defines the amount of water remaining to be allocated to other users. The state space is defined for each of the five users listed in Table 5.7. The stages are defined by the users. The Allocation DP model consists of solving the DP for each treatment alternative, k=1,...5 for each year, j=1,...20. The application of the DP model consists of solving the DP problem k x j times or 100 times for the San Antonio application. The optimal k x j solutions from the allocation DP model are then used to define the state space for the Treatment DP model.



User 1 = Calaveras Power Plant
2 = Braunig Power Plant
3 = Rilling Road Irrigation Area

User 4 = Alamo Valley Irrigation Area
5 = San Antonio River (Sink)

Figure 5.4 Allocation Dynamic Program

The following assumptions were made for application of the DP model for the San Antonio Case,

1. Only one pollutant, BOD_5, is included for water quality considerations.

2. Complete information on the treatment efficiencies of each alternative was not available (from the EIS) so assumptions were made concerning the equations which would characterize the efficiency versus quantity relationships. These assumptions followed efficiency guidelines listed in Table 5.8 (Klemetson and Grenney, 1975). Peak efficiencies were assumed to occur at the design operation flow for the various treatment alternatives. Normally this information would be obtained from treatability studies and treatment models such as those presented in Metcalf and Eddy (1979).

3. It was assumed that water allocated to users 1-4 (Table 5.7) was lost from the system, therefore did not need to be transported to the final sink. Disposal costs are therefore not considered but can be added into the model. In other words, recycling for the secondary users is possible; however, the cost for recycling was not considered in the model solution.

The solution from application of the allocation DP is listed in Table 5.9. The optimal quantity allocated to each secondary user for treatment alternatives 3-5 was the same. This implies that treatment alternatives 3-5 provided enough treatment to meet the BOD standard of each secondary user including the sink and was able to transport enough water to meet at least the minimum demands. In this case more than the minimum demands could be met as evidenced by the large quantity of water that was left over and directed to the sink.

Only treatment Alternative 1 was not able to provide water of sufficient quality for reuse in the beginning of the planning period. This was determined the first year where the program found an insufficient treatment level, the efficiency needed to be improved to be able to provide adequate treatment.

Treatment Altenative 2, on the other hand, was successful in treating the water and providing it for reuse until year 19 when the flow from the primary user exceeded the peak capacity of the wastewater treatment system. If the planning period was shortened this alternative would have been considered totally feasible on its own. Another alternative could have been added to Alternative 2 during the planning period. This possibility was tested in Treatment DP.

At no point in the problem were the allocation schemes infeasible due to lack of water quantity for allocation purposes. Minimum demands were always met and maximum demands could have been exceeded but weren't. If a minimum demand would have been placed on the sink the resulting alloca-

TABLE 5.8 TREATMENT CAPABILITIES FOR VARIOUS TYPES OF WASTEWATER TREATMENT UNITS

Treatment Type	BOD mg/l	BOD % Removal
Primary & Secondary		
Waste Stabilization Lagoon	10-60	70
Extended Aeration	20-20	80-90
	20	-
Primary Sedimentation	120	45
		25-40
High Rate Trickling Filters	40	80
Single Stage	-	60-85
Two Stage	-	80-95
Standard Rate Trickling Filters	20-30	85
	-	80-95
High Rate Activated Sludge	30-50	75
Standard Rate Activated Sludge	15-20	90
	-	85-95
Physical-Chemical	10-15	93
Tertiary		
Intermittent Sand Filtration	3-5	-
	-	90-95
Chemical Precipitation	-	50-75
Chemical Treatment (Solids Contact)	2.9-5	-
Granular or Mixed Media Filtration w/Chem.	3.1-5.8	-
Sand Filtration - Deep Bed	4-12	94-98
Chemical Coagulation and	4-12	94-98
Sand Filtration	2-3	
Microbial Denitrification	-	-
Ammonia Stripping/B.P. Chlorination, (10 mg Cl per 1.0 mg NH_3)	1-3	98
Carbon Adsorption	2-10	95-99
	1.0	-
Microscreening	4-10	94-98
	3	-
Ion Exchange	-	-
	-	-
Reverse Osmosis	1-2	99
	-	-
Electrodialysis	1-2	99
	-	-
Dissolved Air Flotation	-	-
Ultrafiltration	< 1	-
Land Dispersal/Ground Drains	1-2	99

Source: Klemetson, L. and Grenny, W.J., Development of a Dynamic Programming Model for the Regionalization and Staging of Wastewater Treatment Plants, Utah Water Research Laboratory Report PRWA20-2, Logan, Utah State Univ., 1975.

Table 5.9 Minimym Cost Allocation Schemes

Treatment Alternative	Year j	Allocation to Secondary Users (MGD) Calaveras Power Plant	Braunis Power Plant	Rilling Road Irrigation Area	Alamo Valley Irrigation Area	San Antonio River (sink)
K = 1	1-20	Treatment Level Insufficient				
K = 2-5	1	40.28	12.25	4.0	21.20	63.27
	2	40.35	12.25	4.0	21.20	64.58
	3	40.43	12.25	4.0	21.20	65.90
	4	40.50	12.25	4.0	21.20	67.21
	5	40.57	12.25	4.0	21.20	68.53
	6	40.64	12.25	4.0	21.20	69.84
	7	40.72	12.25	4.0	21.20	71.16
	8	40.79	12.25	4.0	21.20	72.47
	9	40.86	12.25	4.0	21.20	73.78
	10	40.94	12.25	4.0	21.20	75.10
	11	41.01	12.25	4.0	21.20	76.41
	12	41.08	12.25	4.0	21.20	77.73
	13	41.16	12.25	4.0	21.20	79.04
	14	41.23	12.25	4.0	21.20	80.36
	15	41.30	12.25	4.0	21.20	81.67
	16	41.37	12.25	4.0	21.20	82.99
	17	41.45	12.25	4.0	21.20	84.30
	18	41.42	12.25	4.0	21.20	85.71
	19	41.39	12.25	4.0	21.20	87.13
	20	41.47	12.25	4.0	21.20	88.44

tion alternatives would have been different. This was not seen as a necessity since a large quantity of water was routed to the sink.

The costs ($Cost_{j,k}$) defined by Equation 4.8 for each feasible allocation alternative and treatment alternative ranged between 400 and 700. Table 5.10 lists the minimum returns for the optimal allocation alternatives (Table 5.9) found for each treatment alternative in each year. Table 5.11 lists the actual O and M costs before the weighted returns were added. The implications of these results will be discussed in Section 5.7

Application of Treatment DP--Figure 5.5 presents a schematic of the treatment DP problem solved over the entire planning period j=1,...20. Each year defines a stage the state space defines the treatment alternative's cost (capital and operation and maintenance costs) for each year. This treatment DP consists of solving a DP for one planning period

The following assumptions were made for the solution of the Treatment DP;

1. Each treatment alternative was considered a separate but complete treatment alternative. That is, this case is not a graduated option treatment design problem where one process could be added to another to make a new alternative. The evaluation of different processes and the possible combinations of these processes had been done previously for the San Antonio area (EIS, 1976). It was from these combinations that treatment alternatives were chosen to be tested in the model.

2. If Node 3 was to receive water for reuse while Treatment 4 was on line, the water would be routed from the Salado plant lessening the wastewater load (and therefore the operation and maintenance costs) to the confluence plant.

3. The interest rate used was 6.375% (EIS, 1976) for the first solution and 5% (Ocanas and Mays 1980) for the second solution.

Based on the results from the allocation DP, the search for the minimum cost treatment alternative was performed. The results are shown in Table 5.12.

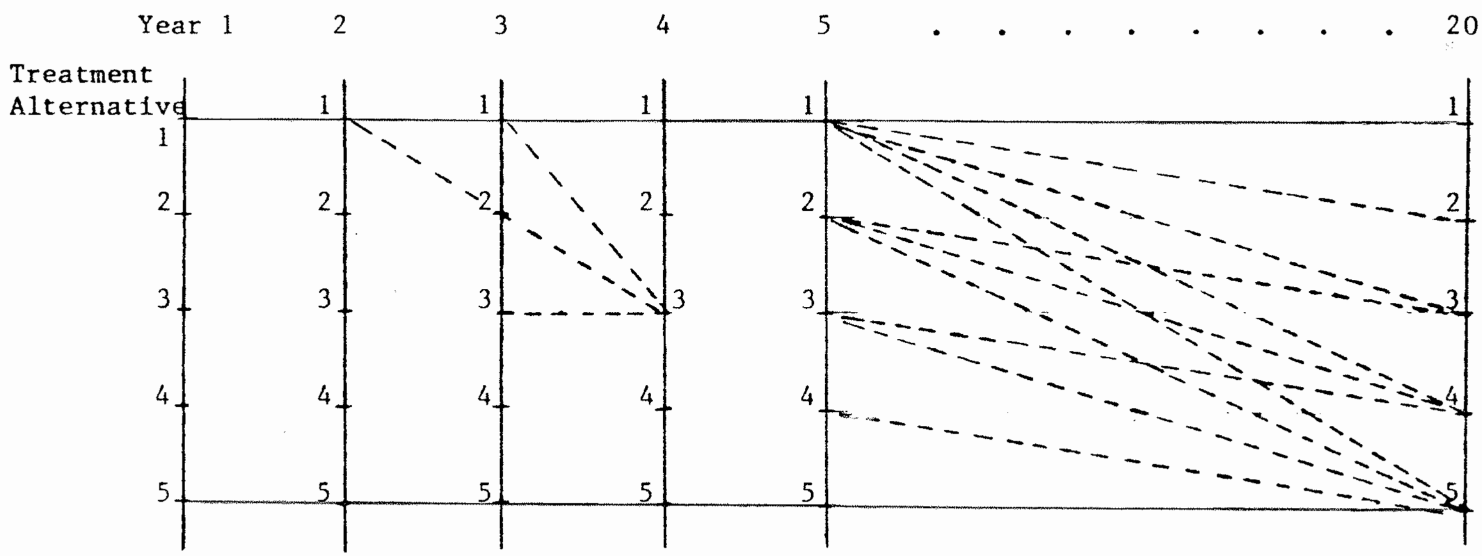
Treatment Alternative 3 (Figure 5.2C) was chosen as the optimal treatment alternative for the entire planning period to provide water for reuse according to the allocation scheme presented in Table 5.9. The capital cost found for this alternative is not comparable to the capital cost quoted from the study from which the alternatives were chosen, i.e., this model quoted a capital cost of $232.43 million whereas the EIS quoted a costs of $71.1 million. The operation and maintenance costs for this alternative for the entire planning period are $67.31 million. These costs are also not comparable to the EIS operation and maintenance cost estimates because different factors were considered.

Table 5.10 Weighted Returns for Treatment Alternatives; Results from Allocation DP

Year j	Treatment Alternative 1	2	3	4	5
1	*	441.95	441.95	441.72	442.08
2	*	454.83	454.83	454.60	454.96
3	*	467.70	467.70	467.48	467.83
4	*	480.58	480.58	480.35	480.71
5	*	493.46	493.46	493.23	493.59
6	*	506.33	506.33	506.10	506.46
7	*	519.21	519.21	518.98	519.46
8	*	532.08	532.08	531.86	532.22
9	*	544.96	544.96	544.73	545.09
10	*	557.84	557.84	557.61	557.97
11	*	570.71	570.71	570.48	570.84
12	*	583.59	583.59	583.36	583.72
13	*	596.46	596.46	596.23	596.60
14	*	609.34	609.34	609.11	609.47
15	*	622.22	622.22	621.99	622.35
16	*	635.09	635.09	634.86	635.22
17	*	647.97	647.97	647.74	648.10
18	*	662.34	662.34	662.11	662.48
19	*	*	676.18	676.49	676.85
20	*	*	689.59	689.36	689.73

Table 5.11 Actual O & M Costs for Treatment Alternatives: Costs Before Discount (million $)

Year j	Treatment Alternative 1	2	3	4	5
1	*	10.67	10.67	10.44	10.80
2	*	10.77	10.77	10.54	10.90
3	*	10.86	10.86	10.63	10.99
4	*	10.96	10.96	10.73	11.09
5	*	11.06	11.06	10.83	11.19
6	*	11.15	11.15	10.92	11.28
7	*	11.25	11.25	11.02	11.38
8	*	11.34	11.34	11.11	11.48
9	*	11.44	11.44	11.21	11.57
10	*	11.54	11.54	11.31	11.67
11	*	11.63	11.63	11.40	11.76
12	*	11.73	11.73	11.50	11.86
13	*	11.82	11.82	11.59	11.96
14	*	11.92	11.92	11.69	12.05
15	*	12.02	12.02	11.79	12.15
16	*	12.11	12.11	11.88	12.24
17	*	12.21	12.21	11.98	12.34
18	*	12.30	12.30	12.07	12.44
19	*	*	12.40	12.17	12.53
20	*	*	12.49	12.26	12.63



--- Example Transitions
___ State Space Limitations

Figure 5.5 Treatment Scheme DP

Table 5.12 Solution to Treatment DP: Minimum Present Value Costs for 20 Years (Alternative 3)

Year j	Waste Flow (MGD)	Minimum Cost (million $)
1	141.00	242.46 *
2	142.39	9.52
3	143.77	9.03
4	145.16	8.56
5	146.55	8.12
6	147.94	7.70
7	149.32	7.30
8	150.71	6.92
9	152.10	6.56
10	153.49	6.22
11	154.87	5.89
12	156.26	5.59
13	157.65	5.29
14	159.04	5.02
15	160.42	4.75
16	161.81	4.51
17	163.20	4.27
18	164.59	4.04
19	165.97	3.83
20	167.36	3.63

Minimum Cost for the Planning Period is $\$359.202 \times 10^6$

*Capital + Operation and Maintenance Costs

This model considered piping operation and maintenance costs to secondary users but did not consider piping to the wastewater treatment plant from the primary user; nor did it consider any pumping operation and maintenance or capital costs.

Because of this particular case study scenario, the full flexibility of this model was not tested. The large difference between capital costs for each alternative and the fact that the allocation weighted returns were the same (due to similar quantity allocations to secondary users) were the controlling factors in the solution of the DP. If pumping costs had been considered, a different solution might have been found because of the topography through which each pipe to a secondary user must be placed.

5.7 ANALYSIS OF RESULTS

The design of treatment alternatives developed for the improvement of San Antonio's wastewater treatment system (EIS, 1976) and used in this model were developed without the goal or forethought of reuse of the wastewater treated. From a technological standpoint, one implication of the initial solution to the model (the solution of the Allocation DP) is that the goal of water reuse can be easily incorporated into a public works plan, i.e., no special allowances or costs need be added to a wastewater treatment plant process design to accommodate wastewater reuse. The only added cost was the piping and pumping costs for transport of treated wastewater to the secondary users.

The optimal returns obtained from the Allocation DP were very close because the allocation schemes were the same. If a particular treatment alternative had been broken down into separate treatment plants and different quantities of water were transported, the solution to the Allocation DP might have been very different. Piping O & M costs would not necessarily have been based on the closest plant to the secondary user. Actual operation and maintenance costs added to the allocation weighted returns might have varied and influenced the solution if the difference was great.

Actual O & M cost varied little between treatment alternatives. They were all very close estimates and even when discounted stayed within a 1-2% range of each other.

The final solution determined that Alternative 3 was the optimal treatment alternative for the entire twenty-year planning period. If the planning period were extended this might not have been so. It must be noted that peak capacity loading of this alternative was 168.75 mgd and the final flow tested in year 20 was 167.36 mgd.

A major limitation of this model is presented when the planning period is extended beyond 20 years. The optimal alternative chosen is dependent up on the length of the planning period examined. If the planning period 25 years, the result is that Alternative 3, then Alternative 4 are needed so that the wastewater flow from the primary user might be treated without a by-pass. This model does not investigate the possibility of putting Alternative 4 on the line initially and evaluating the cost effectiveness.

This problem will only occur when separate treatment alternatives are considered and not enough constraints have been entered. If a graduated option scheme (e.g. building a system treatment level by level or plant by plant) were investigated with the model as it exists, minimizing at each stage would minimize cost for the entire planning period.

5.8 SUMMARY, LIMITATIONS AND CAPABILITIES OF DP MODELS

This section presented a real world application of a planning model which is designed to aid the decision maker in the analysis of various planning alternatives and in finding the best solution to wastewater treatment and allocation within a region.

The San Antonio region was selected as the study area. Information gathered from the Texas Department of Water Resources, EPA documents and Ocanas & Mays (1980) was used to calibrate the model for application to the San Antonio region.

Application of this model indicates that it is useful in selecting the "best" planning alternative. The full flexibility of the model was not tested through this particular application. A different set of quantity and quality demands, ones that had a greater minimum and maximum range would have shown the full flexibility. Consideration of each treatment plant separately, and then as part of a particular alternative could have been added to the model and probably shown different results.

Two major limitations of the models were also found through the application to the San Antonio region. If the planning period was extended a different "best treatment solution" might be found. Also, separate treatment alternatives are more difficult to consider in this model without more constraints in the second DP--the Treatment DP.

The solution technique presented proved highly successful. Using a series of DP solutions to provide the data space for a larger DP provided an excellent path which the solution to a multi-faceted problem such as this one became easy and required minimal computer time. The allocation DP promoted as much water reuse as possible and the treatment DP chose the minimum cost treatment to provide as much wastewater as possible for water reuse.

The flexibility and limitations that exist for the model can be seen in its application. The model is adaptable to regional characteristics such as varying quality or quantity demands for the reusers. Since the model considers demands on the basis of time intervals that are user designated, varying quantity allocations and quality demands are considered. Quality demands for different pollutants and by different reusers can also be considered since the model can consider more than one pollutant constituent in the water and more than one user.

This model also considers varying influent characteristics. If the quality of the influent varies seasonally for example, the effluent quality will also vary. If the influent quantity varies, the effluent quantity will also vary.

The model is flexible in that a number of alternatives and different types of alternatives can be considered. It can consider the development of a wastewater treatment and allocation system from where no coordinated system existed previously. Plants can be designed, in this case by first considering. First primary treatment processes, then secondary, and so on. The optimal process chosen will be the one that meets user quality demands. Different potential reusers can be added to the system or taken from the system for particular years. The model is constrained, though, so that an unreasonable treatment scheme will not be chosen (e.g., progressing from a secondary treatment back to a primary treatment process) as an optimal alternative.

The model can also consider the case where a treatment plant already exists and needs to be modified to provide reusable wastewater. Different processes can be considered and compared to the quality needed. If capacity expansion is needed this alternative may also be considered. This application would require the refining of constraints in the program, since it is not necessarily the effectiveness of a particular progression of processes that needs to be considered, but the efficiency of alternative processes. Alternatives must be carefully constrained and designed in this case.

The model can also consider the case where many WWTP's already exist which need different modifications to be able to provide wastewater for reuse. Again constraints must be added to this program which would be user specific and related to the already existing system (e.g., in the San Antonio case presented). Alternatives may be separately added to the system or they may be progressional in character as in the first type of model presented. Each alternative considered must be clearly explained by the model user so that the decision-maker can determine its impact.

There are four major limitations of this model. Two limitations that become evident when first using the model lie in the structuring of the data set, its characteristics, and the computer space it occupies. In order to vary the set of reusers or alternatives considered for a particular region, additions to or subtractions from the data set must be made and the computer model run again. In other words, if only users 1, 2 and 3 were chosen by the engineer as potential reusers in the San Antonio case, user 4's quantity and quality demands would have to be deleted from the data set and the model run again. Future refinement of the model includes the ability to run the model interactively so potential reusers may be added onto or taken off of the system without a restructuring of the data set. This limitation is not serious because the data set is simply designed.

Although the San Antonio case in no way tested the limits of data space availability (with four reusers and five alternatives), if many potential reusers, many alternatives, or many time intervals were used with the current data packaging method, the program might not be usable on a common computer system. No attempt was made to utilize many of the efficient data packaging methods, as this was not the purpose of this thesis. It is believed that this would greatly expand the capacity of this program. The fact remains that this dynamic program (as in the case of most dynamic programs) uses a large amount of data space. The actual cost of running the program is very small.

The next two limitations of the model do not become evident until the model has been run under varying assumptions. The following assumptions are not limitations as much as they are warnings to the model user. The planning horizon over which the model should be applied must be carefully chosen. For example, the planning horizon chosen in the case study was 20 years. If a 25-year planning horizon had been chosen, the optimal alternative would be Alternative 4 as versus Alternative 3. This is because the wastewater treatment capacity for Alternative 3 (according to the linearly regressed population projections and influent flow projections) would be exceeded in year 23 and become a higher cost solution, for more capacity would have to be added to the plants.

Planning horizons should not be chosen haphazardly. It might be necessary to run the model for different planning horizons and compare the yearly costs and optimal alternatives chosen. Reliability of projections and an extra effort to collect accurate data for the model would greatly reduce the risk of producing results that may not fully depict all the optimal choices a decision-maker might have.

Constraints must be carefully placed in the program. If one alternative should not be paired with another or if costs should be added to the total cost estimates as the program searches through the time intervals, then this should be clearly denoted. Improper placement or statement of constraints that regulate alternative interrelationships might produce an impractical optimal solution.

Future refinements might include the development of subroutines for the model that would designate constraints for the three system situations in a region that might exist before reuser is instituted. These situations are: no wastewater treatment whatsoever, one treatment plant that needs to be modified, or a system of treatment plants that might have to be modified to accommodate wastewater reuse. This might make the program less flexible in that it considers the three scenarios listed only, but it makes the program easier to use and available to a wider range of users.

SECTION 6

SUMMARY AND CONCLUSIONS

6.1 SUMMARY OF MODELS

The economic growth of a region is intimately related to reliable sources of water to support the various use sectors and to satisfy the quality and quantity requirements. The concepts of both water conservation and reuse are important to obtain the maximum utility of water resources. This research developed mathematical optimization models which include the LSNLP models for the regional planning and DP models for the allocation and treatment alternatives. These models, used in an integrated framework could aid the decision maker in the selection of the best alternative to operate a conjunctive water and wastewater system considering water reuse.

A single period planning model (LSNLP) was developed to determine the optimal water reuse alternative within the framework of water resources allocation in a region with various freshwater sources, users, treatment plants and disposal sites during one time period. The single period model was then extended to incorporate the dynamic nature of the system, where capacity expansion considerations are included. Both the single period model and the multi-period models were deterministic, assumed constant system parameters for each planning period, and were solved using the LSNLP techniques.

The application of the models was illustrated first through a series of hypothetical examples (Ocanas and Mays 1980, 1981) to provide information upon the selection of the solution strategy. Two nonlinear optimization methods were used, one is the Large Scale Generalized Reduced Gradient (LSGRG) algorithm developed by Lasdon, et al. (1979). The other is the Successive Linear Programming with Rejection (SLPR) algorithm developed by Palacios and Lasdon (1980). Both techniques proved the capability to solve fairly large nonlinear problems. No formal comparison of the two methods was made, however, a first indication seems to indicate that SLPR might be a faster solution technique for larger problems.

A real world application of the LSNLP models are presented. The Texas Department of Water Resources provided a study area consisting of the City of San Antonio, Texas, and surrounding areas. The area included surface water and groundwater sources, water and wastewater treatment facilities and various use sectors, including municipal, industrial and agricultural users.

A model was also developed to consider the planning aspect on a sub-regional basis which could determine optimal allocation and treatment alternatives on a much smaller time scale. These two models both are based upon dynamic programming (DP). The models are 1) the allocation DP model, which is used for the optimal allocation of water to secondary users and 2) the treatment DP model, which performs the optimal selection of treatment schemes (processes) over time.

The allocation DP model determines how the available water is to be optimally (minimum cost) allocated considering water reuse. In this model the stages are represented as users, the decision variable is the amount of water allocated to a user from each possible treatment alternative considered, and the state variable is the amount of water left to allocate from the treatment alternative. The allocation DP model is solved for each possible treatment alternative and for each time period in the planning horizon. The objective function is to minimize costs. Once the allocation DP model has been solved for each possible treatment alternative and each time period in the planning horizon the treatment DP model is solved. The allocation DP model essentially is used to define the state space in the treatment DP model.

The stages in the treatment DP model are represented by the time periods in the planning horizon, the state variable is the various combinations of treatment alternatives defined by the allocation DP model and the decisions variable is represented by the choice of treatment alternatives. The overall model then provides a minimum cost (present dollars) water allocation and wastewater treatment scheme, considering water reuse, over a planning horizon. The DP models are also applied to the San Antonio case study.

The research also provided a set of computer software programs to aid the models users in the solution of various problem situations. These programs can be used for any region which can be modeled by various sources, water and wastewater treatment plants, users and disposal sites.

6.2 INTEGRATED USAGE OF THE MODELS IN A PLANNING PERSPECTIVE

The LSNLP and DP models are basically developed for two different levels of usage. The LSNLP models are more applicable to long range regional planning using relatively large planning periods. Decisions made by the LSNLP models are (1) the optimal flow distributions among the various users and sources, (2) the optimal pollutant concentrations of the flows leaving users and (3) the capacity expansion of the treatment facilities (treatment plant capacities) in terms of flows. The number and length of the planning periods are dependent upon the size of problem that can be handled.

The allocation and treatment DP models are for determing the optimal allocation of water to users and reusers and the optimal selection of treatment schemes. This model can consider many time periods but can only look at a portion of the region. The model can be used to consider the detailed treatment alternative (processes) whereas the LSNLP models can not.

Because of the nature of each of these models, they can be used in differing aspects of the planning process; however, they also can be used in an integrated fashion for the overall regional planning process. One possible context of using these in an integrated fashion would be to use them interactively. Figure 6.1 illustrates one possible framework. This integrated implementation is for the generation of various alternatives by the decision maker. Usage of the models include interactive modification to planning scenarios by the decision maker (model user) in an effort to generate various planning alternatives. A detailed explanation of an integrated implementation of these models is beyond the scope of this report. Various users and decision makers could use these in several integrated frameworks to consider various types of problems.

6.3 CONCLUSIONS

Conclusions for the LSNLP Models and Their Applications

During the development of this research, different phases have been performed, including development of the models, evaluation of solution techniques, development of computer software and application of the models. Based on all these phases, the following conclusions can be stated:

1. Optimal water allocation planning should incorporate both water and wastewater systems conjunctively to better assess the impact of water reuse.

2. Economies of scale are significant in the outcome of the optimal solution to the planning problems.

3. The solution of a sequence of independent single period problems does not provide an accurate representation of the growing nature of the system. Therefore, single period planning is best suited to assess the impacts of different planning scenarios under current conditions.

4. The capacity expansion of a system is strongly dependent upon the number of planning periods incorporated in the multi-period problem, since the solution for a given period is highly influenced by the characteristics of the remaining periods.

5. The performance of the solution algorithm is greatly influenced by the selection of the initial point, the characteristics of the objective function and the values of the testing parameters in the optimization algorithm.

6. Using a good initial solution significantly decreases the execution time. Furthermore, in some instances, the lack of an initial solution results in the solution technique failing to converge at an optimal point.

7. The final solution can be very dependent upon the specified initial solution, indicating the importance of selecting one initial

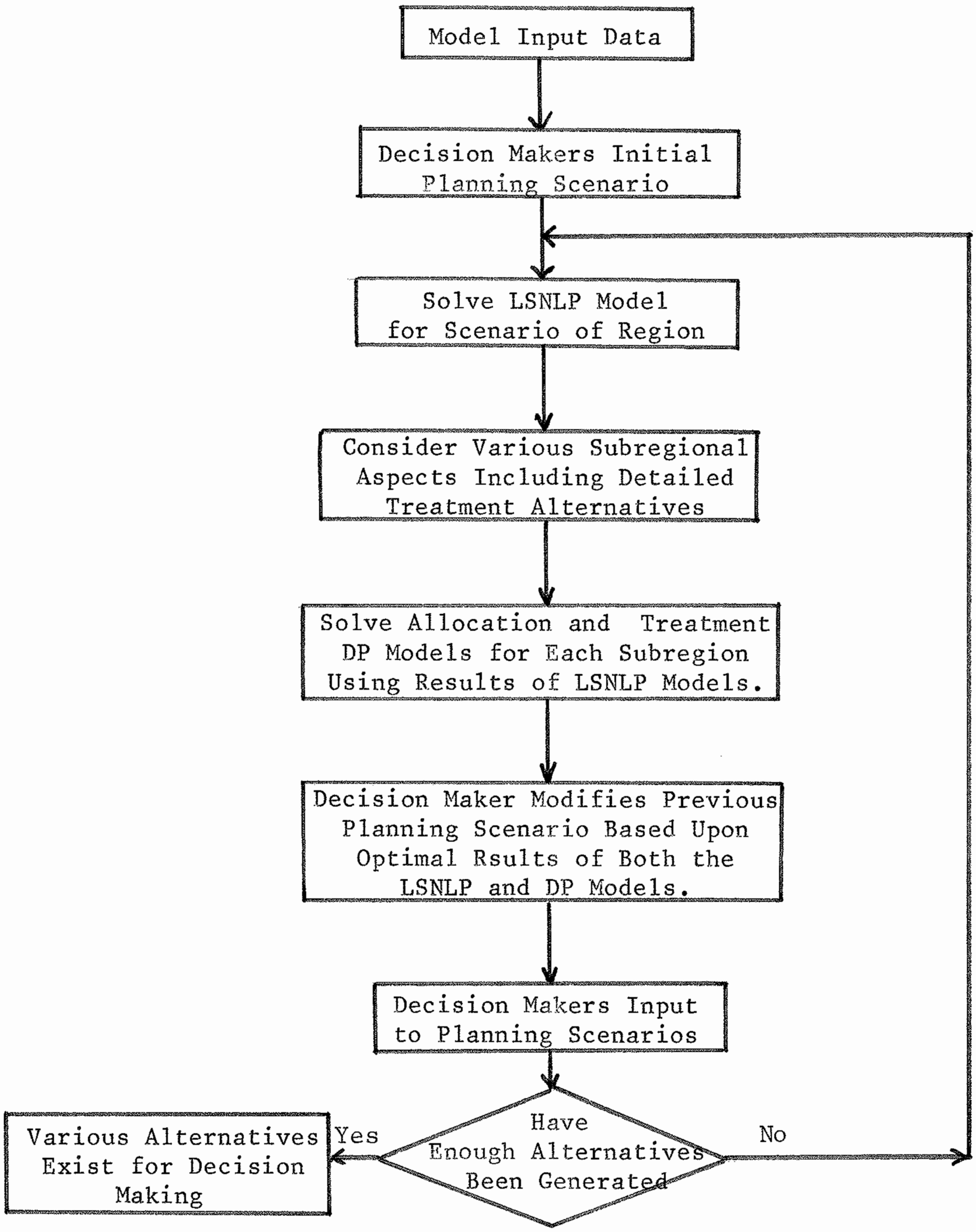


Figure 6.1 Integrated Implementation of LSNLP and DP Models to Generate Planning Alternatives

point. This dependency is due to the concave nature of the objective function which tends to keep a non-basic variable at its bound, because of its relatively large gradient value.

8. Due to the nature of the constraint set, which includes many mass balance equations, and the concavity of the objective function, the optimal solutions found by the LSGRG method were usually extreme points, with all the variables either basics or non-basics at their lower bounds. Since LSGRG automatically defines any variable specified in the initial solution as superbasic, the code required a significant amount of time to eliminate the superbasics to find an extreme point (or an "almost" extreme point).

9. Although most of the work during this research was focused on the use of LSGRG as the solution technique, due to the fact mentioned in number eight, the possibility of having better performance statistics from a method of such as the Successive Linear Programming with Rejection (SLPR) algorithm was considered. Deeper and more formal comparative studies are required before any final recommendations on the solution techniques can be made.

Conclusions for the DP Models and Their Applications

The following conclusions are stated concerning the DP models developed and applied herein:

The objectives of this study were met in the following ways:

1. An investigation of reuse treatment and allocation methods and approaches were reviewed revealing:

a). That treatment technology currently exists to reclaim wastewater to an acceptable level, especially in the case of non-potable reuse;

b). the need for a more integrative and regional approach to the development and evaluation of wastewater reuse alternatives; and

c). policy issues that might effect the outcome or implementation of any wastewater reuse planning or cost-effectiveness evaluation.

2. A model was developed using different treatment and allocation alternatives for the reuse of wastewater. The model considered as important factors the regional characteristics and potential reuser quantity and quality demands on a descrete time interval-basis.

3. The model was successfully applied to a case study in the San Antonio, Texas region and an optimal cost-effective alternative was chosen. Information needed on treatment level and quantity

allocations was provided in order to meet the quality demands of potential reusers.

4. Flexibilities and limitations found during the application of the DP model were presented providing guidelines for future research.

5. Wastewater reclamation and reuse systems are technologically feasible and can be chosen on an environmental and economic basis. With the DP model presented, using a common integrative approach to wastewater reuse and an understandable and systematic presentation of results, policy decisions based on technological, environmental and economic issues should be easier to reach by the decision-maker.

6.4 SUGGESTIONS FOR FURTHER STUDY OF THE LSNLP MODELS

Further extensions for this research can be divided into model-efforts and solution methodology.

Suggestions for Modeling Effort

1. In an effort to better represent the actual situation, the objective function should incorporate the capacity expansion of piping and pumping facilities. This would result in a more complex and larger problem, since new variables associated with the capacity of the piping and pumping facilities would have to be introduced.

2. Incorporation of the treatment plant removal efficiencies as decision variables. The objective function would then include an additional treatment cost function in terms of the removal efficiency. Although this would allow the model more flexibility, the size of the problem would increase by introducing more decision variables, and treating the removal efficiency as a continuous variable may end up with an efficiency value which, from a physical, chemical or biological viewpoint, is not possible to achieve.

3. Incorporation of seasonal variations in the model. This could be easily achieved with the multi-period model by dividing the planning period into subintervals which represent the various seasons. This, however, would result in much larger optimization problems which could not be solved with current technology.

4. In an attempt to reduce the effective size of the problems, it would be desirable to have a matrix generator which could delete those constraints which are known "a priori" to be non-restrictive, as might be the case of a user with a quality criteria which involves only one limiting pollutant. In such a case, the quality

constraints of such users associated with the remaining pollutants could be deleted. This would allow the model to include many pollutants for the quality constraints enforcement without seriously increasing the size of the problem.

Suggestions on Solution Methodology

1. The current LSGRG version should be modified to accept an initial basis consisting of nonlinear variables. This would allow the user to specify the initial solution obtained from the procedure (OKA-LP) described in Chapter 2 as an initial basis rather than as superbasics, which would benefit from the fact that the optimal solutions include very few superbasics, if any. It would also allow specifying the optimal basis of a smaller problem (two planning periods, for instance) as the initial basis of a larger problem (three planning periods). This procedure could then be tested to evaluate the possibility of a hierarchical solution procedure which would allow the solution of larger problems with a relatively small execution time.

2. A formal comparison of the two proposed solution techniques, LSGRG and SLPR, should be made. The comparison would require very careful inspection of the effect of initial solutions for both methods, the effect on algorithm parameters, and the effect of initial ranges for all the variables as required by SLPR.

REFERENCES

1. Abadie, J. and J. Carpentier, "Generalization of the Wolfe Reduced Gradient Method to the Case of Non-Linear Constraints," Optimization, R. Fletcher, Ed., Academic Press, 1969, pp. 37-47.

2. Abadie, J., "Application of the GRG Algorithm to Optimal Control Problems," Non-Linear and Integer Programming, J. Abadie, Ed., North Holland Publishing Co., 1972, pp. 191-211.

3. American Water Works Association Research Foundation, "Abstracts of Technical Papers and Poster Presentations," Water Reuse Symposium, Washington, D.C., March 1979.

4. Bauer, W. J., "Economics of Urban Drainage Design," Proc. Am. Soc. Civil Eng., Vol. 88, 1962, pp. 93-98.

5. Becker, L. and Yeh, William W. G., "Optimal Timing, Sequencing and Sizing of Multiple Reservoir Surface Water Facilities," Water Resources Research, Vol. 10, No. 1, February 1974, pp. 57-62.

6. Beightler, C. S. and D. T. Phillips, Applied Geometric Programming, John Wiley and Sons, Inc., New York, 1976.

7. Benjes, H. H., "Design of Sewage Pumping Stations," Public Works Magazine, Vol. 91, 1960, pp. 89-90.

8. Bhalla, H. S. and R. F. Rikkers, "Multi-Time Period, Facilities Location Problems," Publication No. 21, Water Resources Research Center, University of Massachusetts, Amherst, Massachusetts, 1971.

9. Bishop, A. B., and D. W. Hendricks, "Water Reuse Systems Analysis." Journal of the Sanitary Engineering Division, ASCE, Vol. 97, No. SA1. pp.41-57 1971.

10. Bishop, A. B., D. W. Hendrick, and J. H. Mulligan, "Assessment Analysis for Water Supply Alternatives," Water Resources Bulletin, AWRA, Vol. 7, No. 3., 1977, pp.542-553.

11. Bishop, A. B., and R. Narayanan, "Seasonal and Stochastic Factors in Water Planning," Journal of the Hydraulics Division, ASCE, Vol. 2, No. HY10. pp.1159-1172, 1977.

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

13. Cohen, C., "Generalized Reduced Gradient Technique for Non-Linear Programming -- User Writeup," Vogelback Computing Center, Northeastern University, February 1974.

14. Converse, A. D., "Optimum Number and Location of Treatment Plants," Journal of the Water Pollution Control Federation, Vol. 44, No. 8, August 1972.

15. Curry, G. L. and R. W. Skeith, "A Dynamic Programming Algorithm for Facility Location and Allocation," AIEE Transactions, Vol. 1, No. 2, February 1969, pp. 133-138.

16. Dakin, R. J., "A Tree Search Algorithm for Mixed Integer Programming Problems," Computer Journal, Vol. 8, 1975.

17. Deininger, R. A., "Water Quality Mangement - The Planning of Economically Optimal Pollution Control Systems," Annual Meeting of AWRA, 1st, University of Chicago, Chicago, Illinois, 1965.

18. Ford, L. R. Jr. and D. R. Fulkerson, Flows in Networks, Princeton University Press, Princeton, New Jersey, 1978.

19. Fletcher, R. and C. M. Reeves, "Functional Minimization by Conjugate Gradients," British Computer Journal, Vol. 7, 1964, pp. 149-154.

20. Fletcher, R., "A New Approach to Variable Metric Algorithms," The Computer Journal, Vol. 13, 1970, pp. 317-322.

21. Garrison, W. E. and R. P. Miele, "Current Trends in Waer Reclamation Technology," Journal of the AWWA, Vol. , No. July 1977, pp. 364-369.

22. Graves, G. W., A. B. Whinston, and G. B. Hatfield, "Mathematical Programming for Regional Water Quality Management," FWQA, Department of Interior, 1970.

23. Haimes, Y. Y. and W. S. Nainis, "Coordination of Regional Water Resource Supply and Demand Planning Models," Water Resources Research, AGU, Vol. 10, No. 6, December 1974, pp. 1051-1059.

24. Heltne, D. R. and J. M. Liitschwager, "Users Guide for GRG 73," and "Technical Appendices to GRG 73," College of Engineering, University of Iowa, September 1973.

25. Hillier, F. S. and G. J. Lieberman, Operations Research, Holden-Day, Inc., San Francisco, 1974.

26. James, L. D., and R. R. Lee, Economics of Water Resources Planning. McGraw Hill, New York, N. Y., 1971.

27. Kaplan, M. A. and Y. Y. Haimes, "Dynamic Programming for Optimal Capacity Expansion of Wastewater Treatment Plants," Water Resources Bulletin, American Water Resources Association, Vol. 11, No. 2, April 1975, pp. 279-293.

28. Klemetson, L. and W. J. Grenney, "Development of a Dynamic Programming Model for the Regionalization and Staging of Wastewater Treatment Plants. Utah Water Research Laboratory Report PRWA20-2. Utah State University, Logan, Utah, 1975.

29. Lasdon, L. S. and A. D. Waren, "Generalized Reduced Gradient Software for Linearly and Non-Linearly Constrained Problems," Working Paper 77-85, Graduate School of Business, University of Texas at Austin, October 1977.

30. Lauria, D. T., "Regional Sewerage Planning by Mixed Integer Programming," Research Report, University of North Carolina, Chapel Hill, N. C., 1975.

31. Leondes, C. T. and R. D. Nandi, "Capacity Expansion in Convex Networks with Uncertain Demand," Operations Research, Vol. 23, No. 6, November-December 1975, pp. 1172-1178.

32. Lesso, W. G., et al., "A Method for Determining the Selection and Scheduling of Waste Treatment Plants in a River Basin." Report, University of Texas, 1977.

33. Luenberger, D. G., Introduction to Linear and Nonlinear Programming, Addison-Wesley, Reading, Massachusetts, 1973.

34. Martin, Q. W., "Optimal Capacity Expansion of a Regional Water Supply System," Report, Texas Water Development Board, Austin, Texas, November 1975.

35. Metcalf and Eddy, Inc., Wastewater Engineering: Collection, Treatment, Disposal, McGraw-Hill, Inc., 1972.

36. Metcaff and Eddy, Inc. Wastewater Engineering: Treatment, Disposal, Reuse. McGraw Hill, New York, N. Y., 1979.

37. Milliken, J. G. and A. S. Trumbly, "Municipal Recycling of Wastewater," Journal of the American Water Works Association, Vol. 79, No. 10, October 1979.

38. Mulivihill, M. E., and J. A. Dracup, "Optimal Timing and Sizing of a Conjunctive Urban Water Supply and Wastewater System with Non-Linear Programming." Water Resources Research, AGU, Vol. 10, No. 2, April 1974, pp. 170-175.

39. Murtagh, B. and M. Saunders, "Non-linear Programming for Large Sparse Systems," Technical Report SOL 76-15, Department of Operations Research, Stanford University, August 1976.

40. Nemhauser, G. L. Dynamic Programming. New York: John Wiley and Sons, 1966.

41. Ocanas, G. and Mays, L. W. A Model for Water Reuse Planning. Water Resource Research, AGU, Vol. 17, No. 1. pp.25-32, February 1981a.

42. Ocanas, G. and L. W. Mays, Water Reuse Planning Models: Extensions and Applications, Water Resource Reserch, AGU, Vol. 17, No. 5, pp. 1311-1327, October 1981b.

43. Ocanas, G. and L. W. Mays, Models for Water Reuse Planning. Technical Report No. 173. The University of Texas Center for Research in Water Resources, Austin, Texas, 1980.

44. Palacios-Gomez, F., L. S. Lasdon and M. Engquist, "Non-Linear Optimization by Successive Linear Programming," Submitted for Publication to Management Science, July 1980.

45. Perry, A., "An Improved Conjugate Gradient Algorithm," Technical Note, March 1976, Department of Decision Sciences, Graduate School of Management, Northwestern University, Evanston, Illinois.

46. Pingry, D. E., and T. L. Shaftel, "Integrated Water Management with Reuse: A Programming Approach." Water Resources Research, AGU, 1979, pp. 8-14.

47. Polak, E., Computational Methods in Optimization: A Unified Approach, Academic Press, 1971.

48. Rios, R. A., J. S. Sherman, and J. F. Malina, "A Non-Linear Programming Model for Evaluating Water Supply Policies in the Texas Coastal Zone." Technical Report EHE-75-07, CRWR-130. The University of Texas Center for Research in Water Resources, Austin, Texas 1975.

49. Rossman, L. A. and J. C. Liebman, "Optimal Regionalization of Wastewater Treatment for Water Quality Management," WRC Research Report No. 89, Water Resources Center, University of Illinois, Urbana, Illinois, 1974.

50. Sa, G. "Branch-Bound and Approximate Solution to the Capacitated Plant Location Problem," Operations Research, Vol. 17, No. 6, November-December 1969, pp. 1005-1016.

51. Salkin, H. M., Integer Programming, Addison-Wesley, Reading, Massachusetts, 1975.

52. San Antonio City Water Board, Preliminary Engineering Report Applewhite Dam and Reservoir, 1974.

53. Saunders, M. A., "A Fast, Stable Implementation of the Simplex Method Using Bartels-Golub Updating," in J. R. Bunch and D. J. Rose, Eds., Sparse Matrix Computations, Academic Press, New York, 1976, pp. 213-226.

54. Scarato, R. F., "Time-Capacity Expansion of Urban Water Systems," Water Resources Research, AGU, Vol. 5, No. 5, October 1969, pp. 929-936.

55. Texas Department of Water Resources, Texas Water Facts, Austin, Texas, January 1979.

56. Thomann, R. V., Systems Analysis and Water Quality Management, Environmental Research and Applications, Inc., McGraw Hill, New York, 1972.

57. Tuesdale, G. A. and R. G. Taylor, "Sewage Treatment to Meet Tomorrow's Needs," Journal of the Water Pollution Control Federation, Vol. 74, 1975, pp. 455.

58. Tsou, A., L. G. Mitten and S. O. Russell, "Search Technique for Project Sequencing," Journal of the Hydraulics Division, Vol. 99, No. HY5, May 1973, pp. 833-839.

59. Tung, Y.K., and Mays, L. W. Optimal Risk Based Design of Hydraulic Structures. Technical Report No. 171. Austin: The University of Texas Center for Research in Water Resources, 1980.

60. U. S. Army Corps of Engineers, Fort Worth District, "Wastewater Management Plan, Colorado River and Tributaries, Texas," Technical Appendix, Vol. III, September 1973.

61. U. S. Environmental Protection Agency, "Construction Costs for Municipal Wastewater Treatment Plants: 1973-1977," Technical Report MCD-37, General Services Administration, Building 41, Denver Federal Center, Denver, Colorado, 80225, January 1978a.

62. U. S. Environmental Protection Agency, "Analysis of Operations & Maintenance Costs for Municipal Wastewater Treatment Systems," Technical Report MCD-39, General Services Administration, Building 41, Denver Federal Center, Denver, Colorado, 80225, FEbruary 1978b.

63. U. S. Environmental Protection Agency, Region VI, Dallas, Texas, "EIS: San Antonio Wastewater Treatment System," April 1978.

64. U. S. Environmental Protection Agency, "Construction Costs for Municipal Wastewater Conveyance Systems: 1973-1977," Technical Report MCD-38, General Services Administration, Building 41, Denver Federal Center, Denver, Colorado, 80225, May 1978c.

65. U. S. Environmental Protection Agency, "Estimating Water Treatment Costs, Volume 2 Cost Curves Applicable to 1 to 200 MGD Treatment Plants," Office of Research and Development, Cincinnati, Ohio, 45268, August 1979.

66. Weber, W. J. Physio-chemical Process for Water Quality Control. Environmental Science and Technology, Wiley-Interscience, New York, 1972.

67. Wesolowsky, G. O., "Dynamic Facility Location," Management Science, Vol. 19, No. 11, July 1973, pp. 1241-1248.

68. Wilde, D. J. and C. S. Beightler, Foundations of Optimization, Prentice-Hall, Englewood Cliffs, N. J., 1967.