TEXAS
WATER
DEVELOPMENT
BOARD

Report 131



# STOCHASTIC OPTIMIZATION AND SIMULATION TECHNIQUES FOR MANAGEMENT OF REGIONAL WATER RESOURCE SYSTEMS

JULY 1971

# TEXAS WATER DEVELOPMENT BOARD

## REPORT 131

# STOCHASTIC OPTIMIZATION AND SIMULATION TECHNIQUES FOR MANAGEMENT OF REGIONAL WATER RESOURCE SYSTEMS

## A COMPLETION REPORT

Prepared By
Systems Engineering Division
Texas Water Development Board
And
Water Resources Engineers, Inc.

The work upon which this publication is based was supported in part by funds provided under Grant No. 14-31-0001-3156 by the Office of Water Resources Research, United States Department of the Interior, as authorized under the Water Resources Act of 1964 as amended.

July 1971

Published and distributed
by the
Texas Water Development Board
Post Office Box 13087
Austin, Texas 78711

# FOREWORD

The Texas Water Development Board began a long-range program of applied research in 1967 in water resource system simulation and optimization. The objective was to develop a set of generalized computer-oriented planning tools for use in detailed planning, design, and management of water resource systems such as the Texas Water System, as proposed in the Texas Water Plan.

With the advice, encouragement, and financial assistance of the United States Department of the Interior, Office of Water Resources Research (OWRR), the guidance of an eminent research advisory panel, and the assistance of several consulting firms, the Texas Water Development Board has now completed the second phase of a three-phase program. This volume summarizes the results of this second-phase effort, the primary objective of which was to develop a practical methodology and attendant models for evaluating the impact that stochastic variability of both the supply of and demand for water has on planning for the optimal development of a complex water resource system.

This report has been prepared for widespread dissemination for the purposes of informing water resource planners of the techniques developed during the research that may be of use in applying systems analysis procedures to the planning of water and related land resource systems.

Harry P. Burleigh

Harry P. Burleigh
Executive Director
Texas Water Development Board

# PREFACE

## Research Objective

The research described in this report was conducted as the second step in developing a computer-oriented methodology for use in the planning, design, and long-range operation and management of a large complex multibasin water resource system such as the proposed Texas Water System. The research was undertaken to seek solutions for the problem of developing such a water resource system to achieve the objective of meeting a prespecified demand for water for all purposes over a 50-year planning horizon under the following conditions: both the supply and demand have a large stochastic component; and over this period of time costs can be reduced by incurring a certain level of tolerable shortages. The techniques developed in seeking a solution to this problem provide a means by which planning decisions can be reached as to when each of the elements of the system should be constructed and available for service, how large facilities such as reservoirs and canals should be; when, where, and how much water should be imported; when, where, and what levels of shortages should be incurred under various conditions; and how the system should be operated to meet the demand schedule with tolerable shortages at the lowest reasonable cost.

The researchers drew upon the techniques of the so-called "systems approach" to find a means of developing a comprehensive set of tools and application procedures for solving such a water resource development problem, thus permitting a systematic evaluation of the myriad alternatives available. Of particular importance was the development of a planning and design methodology for evaluating complex water resource systems not amenable to analysis by other techniques. For example, considerable attention was given to perfecting a detailed simulation and optimization model (SIM-III) to serve as the basic analytic tool for comparing design alternatives. With this model and its attendant application procedures, a system of reservoirs and points of water demand, connected in any possible network configuration, can be effectively evaluated. Similarly, by use of this model, a wide range of alternatives and numerous stochastic data sequences (in addition to the historical sequence) can be analyzed to determine the impact that the variability associated with the stochastic components of the supply and the demand has on the operational and cost efficiency of the system being simulated.

## Description of the Report

This report discusses the procedures developed as they are applied to an example problem. Information concerning the success of this application procedure, conclusions made, and recommendations for follow-on activities are presented in Chapter I. Also discussed in Chapter I is the relationship between the research reported herein, the first phase of research reported in Texas Water Development Board Report 118, and the third phase of research currently in progress.

Chapter II presents an overview of the six major steps comprising the overall planning procedure derived by this research, provides a general description of the SIM-III model capability, and describes the characteristics of the multibasin system used in the example problem.

Chapter III discusses "Data Base Development" for use with SIM-III and the other attendant programs. This discussion includes the rationale for developing a stochastic as well as an historical data base of supply and demand quantities. Chapter III also describes the development of all of the other physical and cost information necessary to permit simulation of the prototype.

Chapter IV discusses the work associated with "Plan Development." This consists of describing how a standard hydrologic analysis, "firm-yield analysis," is used to obtain a preliminary set of reservoir and canal sizes within each basin to meet expected demands for water at various future time periods. This chapter also identifies the "firm surplus" and "firm deficit" of each basin.

Chapter V presents the heart of the methodology referred to as "Plan Improvement." In this chapter the results of "Plan Development" are analyzed in other than a firm-yield context. The purpose is to identify the best system configuration and operating criteria for the problem being analyzed. This is done at various future time periods which correspond to various staging increments on a prespecified demand buildup curve. As in the Plan Development activity, to save computer time, only the historical data set is used upon which to base judgement.

Chapter VI describes a methodology for selecting a representative unbiased sample of sequences from numerous stochastic sequences. These sequences are generated as described in Chapter III.

Chapter VII describes the work associated with "Plan Optimization." It uses the initially estimated staging plan developed in Chapter V, both the historical and stochastic sequences of supply and demand quantities, and the SIM-III modeling capability. The objective of "Plan Optimization" is to help find the staging plàn that has the least expected cost, defined also as the "optimal" implementation plan.

Chapter VIII describes the work associated with the "Analysis of Variability." Specifically this chapter evaluates the impact that stochastic variability in the hydrologic input data has on the cost response of the prototype being simulated. This analysis also provides the framework for determining how stochastic variability might cause the planner to select other than the minimum-expected-cost plan. For example, the variability in the cost response of the minimum-expected-cost plan might be excessive. In this event, both the structural and economic performance of the prototype must be evaluated, and a criterion of acceptable risk versus cost must be established prior to selecting the "most reasonable" implementation plan.

Chapter IX describes the last portion of the work associated with planning and design activities—Sensitivity Analysis. This analysis is performed on the plan selected for implementation and is normally referred to as "post-optimal sensitivity analysis." In this analysis all of the input variables are subjected to perturbations from their most likely value to determine the impact that this variation has on the physical and cost performance of the prototype being simulated.

## Organization

The Texas Water Development Board was responsible for overall research project management, under the general direction of C. R. Baskin, Chief Engineer. Mrs. Jean O. Williams, Program Controller, Lewis B. Seward, Director of Planning, and Arden O. Weiss, Director of the Systems Engineering Division, were instrumental in initiating and sucessfully completing this project, and in establishing and maintaining liaison with the Office of Water Resources Research.

Arden O. Weiss and Dr. Wilbur L. Meier, Professor at Texas A&M University, served as Co-Principal Investigators for the research project, provided the administrative and technical guidance to the project, and were responsible for formulating and writing most of the material contained herein. Under the direction of Mr. Weiss, Carlos D. Puentes was responsible for the development and thorough testing of the SIM-III program, a major product of this research; the DEMAND-II program and portions of the data management programs were developed by Daniel E. Salcedo; Jack Ferguson was responsible for developing the CAPEX-I program.

Mr. Weiss, with the assistance of Dr. George K. Young of Water Resources Engineers, Inc., Leo R. Beard of the Consulting Panel, and Dr. Meier, was responsible for the sequence analysis and selection procedures used to make possible the efficient use of large volumes of stochastic data in conjunction with detailed optimization and simulation programs. Mr. Weiss was also responsible for formulating and implementing SEQUEN-I and most of the data management and analysis programs with the assistance of Mr. Puentes, Terry L. Ellis, Dr. Lial F. Tischler, and William A. White.

Water Resources Engineers, Inc., under the direction of its president, Dr. Gerald T. Orlob, developed portions of the programs documented in Volume II of this report and assisted the Systems Engineering staff of the Texas Water Development Board in developing and testing a solution methodology. Dr. George K. Young was Project Leader for the Water Resources Engineers, Inc., staff and with the assistance of Dr. Ian P. King, William R. Norton, Jerry Tierney, and Donald E. Evenson assumed the responsibility for the development of portions of this report and the models described herein.

Assistance in all phases of the research and of report preparation was received from the Consulting Panel; Dean Dean F. Peterson, Chairman; Harvey O. Banks; Leo R. Beard; Dr. Ven Te Chow; and Dr. Herbert W. Grubb. Throughout the project they reviewed progress and provided valuable guidance to the research staff.

## Acknowledgments

The output of this research as documented in Volumes I through III of this Completion Report would not have been possible without the enthusiastic support and involvement of many individuals and the agencies they represent.

First, the research team wishes to acknowledge the support given the project by the Texas Water Development Board, its members individually, Jack W. Fickessen, its former Acting Executive Director, and Harry P. Burleigh, Executive Director.

The advice and encouragement given by Dr. H. G. Hershey, Director, Mr. Eugene D. Eaton, former Associate Director, and Dr. Edward G. Altouney, Research Scientist, all of the Office of Water Resources Research, are greatly appreciated.

To these individuals, and others who have encouraged the program from its inception to its present state, the researchers express their profound appreciation.

Arden O. Weiss

Arden O. Weiss
Director, Systems Engineering Division

# ABSTRACT


**Stochastic Optimization and Simulation Techniques for Management of Regional Water Resource Systems**

Texas Water Development Board, Austin, Texas Water Resources Engineers, Inc., Walnut Creek, California Office of Water Resources Research, Washington, D.C.

*File Retrieval Descriptors:*
PLANNING Water Resources Development, Optimum Development Plans (Minimum Expected Cost), Model Studies, Stochastic Hydrology.
OPERATIONS RESEARCH Systems Analysis, Networks, Simulation, Optimization (Minimum Expected Cost), Linear Programming, Sampling.

This research represents the second phase of a three-phase research project leading towards the development of a computer-oriented planning system for use in the planning of large, multibasin systems of reservoirs and connecting transfer links (river reaches and pump-canals). Specifically, the research defines a methodology for finding an optimal staging plan for implementing a multibasin water resource system—a system that must meet, with an optimal level of shortages, prespecified but highly variable demands for water that increase over time—a system such as the proposed Texas Water System.

The methodology explicitly evaluates the impact that stochastic variability of the meteorologic variables and uncertainty in the remaining variables have on determining (1) which of an over-specified set of reservoirs and pump-canals should be constructed, (2) how large each of the reservoirs and pump-canals should be at various points on a demand-buildup curve, and (3) how the resulting optimized system should be operated both during and after the period in which facilities are being added or increased in size, to minimize the present worth of their construction costs, expected operation costs, expected maintenance costs, and expected penalty costs incurred for demands not met.

The computer programs developed during this research are designed to analyze a problem on a monthly basis using historical or stochastic hydrologic input data sequences, a specified demand-buildup period, and an economic life as defined by the user.

This research is documented in the following volumes:

| | |
|---|---|
| Volume I | – Introduction |
| Volume IIA | – SIM-III Program Description |
| Volume IIB | – FILLIN-I Program Description |
| Volume IIC | – AL-II Program Description |
| Volume IID | – DEMAND-II Program Description |
| Volume IIE | – SEQUEN-I Program Description |
| Volume IIF | – CAPEX-I Program Description |
| Volume III | – Data Management and Analysis Program Descriptions |

## TABLE OF CONTENTS

## TABLE OF CONTENTS (Cont'd.)

# TABLE OF CONTENTS (Cont'd.)



## TABLES

# TABLE OF CONTENTS (Cont'd.)



## FIGURES

## TABLE OF CONTENTS (Cont'd.)

## TABLE OF CONTENTS (Cont'd.)

# STOCHASTIC OPTIMIZATION AND SIMULATION TECHNIQUES FOR MANAGEMENT OF REGIONAL WATER RESOURCE SYSTEMS

### A COMPLETION REPORT

## I. SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

### Objectives of the Research

The applied research program of the Texas Water Development Board is designed to develop generalized computer-oriented planning models and procedures and to train a staff to use these models and procedures in the detailed planning, design, and management of complex water resource systems such as is proposed in the Texas Water Plan. Three separate but related projects, funded in part by the Office of Water Resources Research, are included in this overall applied research program. Each of the three research projects is of approximately one year duration.

Project I — Development of deterministic simulation and optimization techniques for helping the planner find the minimum-cost physical system and operational criteria for satisfying fixed water demands with a single set of prespecified hydrologic conditions.

This initial project was the first step toward developing models and procedures for use in planning large, multibasin systems of reservoirs and connecting transfer links (river reaches and pump-canals). Eight interrelated computer programs (four data management and four simulation/optimization programs) and a methodology for collectively using these programs were developed.

The data management programs provide a convenient means for organizing, in usable form, most of the data required by the simulation/optimization programs. The simulation/optimization programs collectively help define (1) when to construct proposed reservoirs and transfer links, (2) the maximum capacity of each facility, and (3) the operating policy for each facility, both during and after the period in which facilities are being added, so as to minimize the present worth of construction, operation, and maintenance costs.

Project II — Development of a set of practical procedures and techniques for quantifying the effect that stochastic variability has on the structure, implementation, and operation of the minimum-expected-cost physical system referred to above, and improvement of the simulation and optimization modeling techniques.

The objectives of the second project—the subject of this report—are to:

- further enhance the utility of the models and application procedures developed during Project I,
- develop, test, and evaluate methodologies for incorporating into planning analysis procedures a means for reaching planning decisions with adequate awareness of factors of uncertainty and of hydrologic and meteorologic stochastic variability,
- further develop the capabilities of a planning staff in using the developed techniques to help solve complex water resource planning problems, and
- develop a full set of documentation, including general and detailed descriptions of the results of this year's research.

Project III — Development of a set of practical procedures for interfacing the results of Projects I and II with the economic analysis of benefits so as to search out the physical system implementation plan and operating procedures that maximize net benefits over the planning horizon.

The third project, scheduled to be completed by January of 1972, has as its objectives to:

- further enhance the utility of the models and application procedures developed during both Project I and Project II,
- develop, test, and evaluate methods for measuring the economic impact that may be expected to result from shortages of water occurring in a large water resource system,
- improve development of operating rules,
- develop, test, and evaluate a model for determining the optimum level of water delivery that will insure the most effective economic and physical use of an available water supply, including evaluation of the effects of stochastic variability in the intensity, duration, frequency of occurrence, and timing of planned shortages, and
- document the results of all three research projects and present the final set of models and application procedures in a manner useful to the planning community.

## Summary

Project I is documented in a previous report,[1] Project II is the topic of this report, and Project III is currently in progress. Each of the projects is designed to fit together as a portion of the generalized computer-oriented planning system being developed at the Texas Water Development Board.

This computer-oriented planning system, scheduled for completion by 1972, and the procedures and techniques developed in the research projects are not intended to be all-powerful methods that will provide a detailed quantitative solution to every problem; nor do all programs in the system try to represent an exact simulation of the prototype. The set of computer programs comprising this system will, however, represent a set of mathematical techniques that will approximate the prototype at various degrees of fidelity and give information, at varying levels of accuracy, necessary to select between alternatives.

It is within these limits of applicability that these tools are intended to function. That is, not to replace the experience and judgment of the planner, but to help him obtain answers to his "what-if" questions and, thus, help him better understand the processes and interactions at work in complex water resource systems such as the proposed Texas Water System shown in Figure 1.

In that context, models and solution strategies can be developed to help answer almost any set of questions, and to provide support for most decisions. As tools, they provide a means for effective planning and for obtaining realistic and acceptable solutions by asking the right questions in a systematic manner.

For example, water supply development opportunities are physically or geographically dispersed along the rivers and streams of land masses, or are associated with ground-water occurrence. The water supply production points are specific with regard to river basin or aquifer location and unique with regard to topography, streamflow, water quality, and other factors. The number of potential surface storage sites varies from basin to basin, and the quantity of water that can be produced from each site depends upon the physical and meteorological characteristics of each site, and upon the effects on each site of prior basin development.

Also, water demands are dispersed at varying distances from potential surface water production sites—some water demands are nearby; many are miles upstream or downstream from the storage source; others are in neighboring or even remote basins.

Therefore, selection between water investment projects requires an evaluation of alternative projects in sufficient detail to support valid decisions regarding the relative costs and benefits, both direct and indirect, of each project. It is necessary to expand the scope of water planning to permit the simultaneous consideration of more than one water demand area, more than one set of potential water development sites per river basin, and more than one river basin. In brief, water planning must consider the myriad of physical and economic complexities involved in water resources development in a comprehensive and systematic way so that planning variables can be adequately considered, evaluated, and put into perspective when selecting projects for implementation.

The number of relevant planning variables is so great, and the physical and economic relationships underlying water systems are so complex, that computer-oriented techniques are needed to evaluate the many alternatives available. Only through effective use of these techinques can the bad alternatives be identified and eliminated concurrently with identifying the more attractive ones for detailed evaluation. During this process, huge quantities of data are required, millions of computations and comparisons are desired, and many simulated economic and physical observations are necessary prior to selection of the most reasonable implementation plan. With respect to the economic component, both benefit and cost response information are necessary.

[1] Texas Water Development Board and Water Resources Engineers, Inc., *Systems Simulation for Management of a Total Water Resource*, Completion Report to the Office of Water Resources Research, Volume I, August 1969, 132 p. Published as Texas Water Development Board Report 118, May 1970.

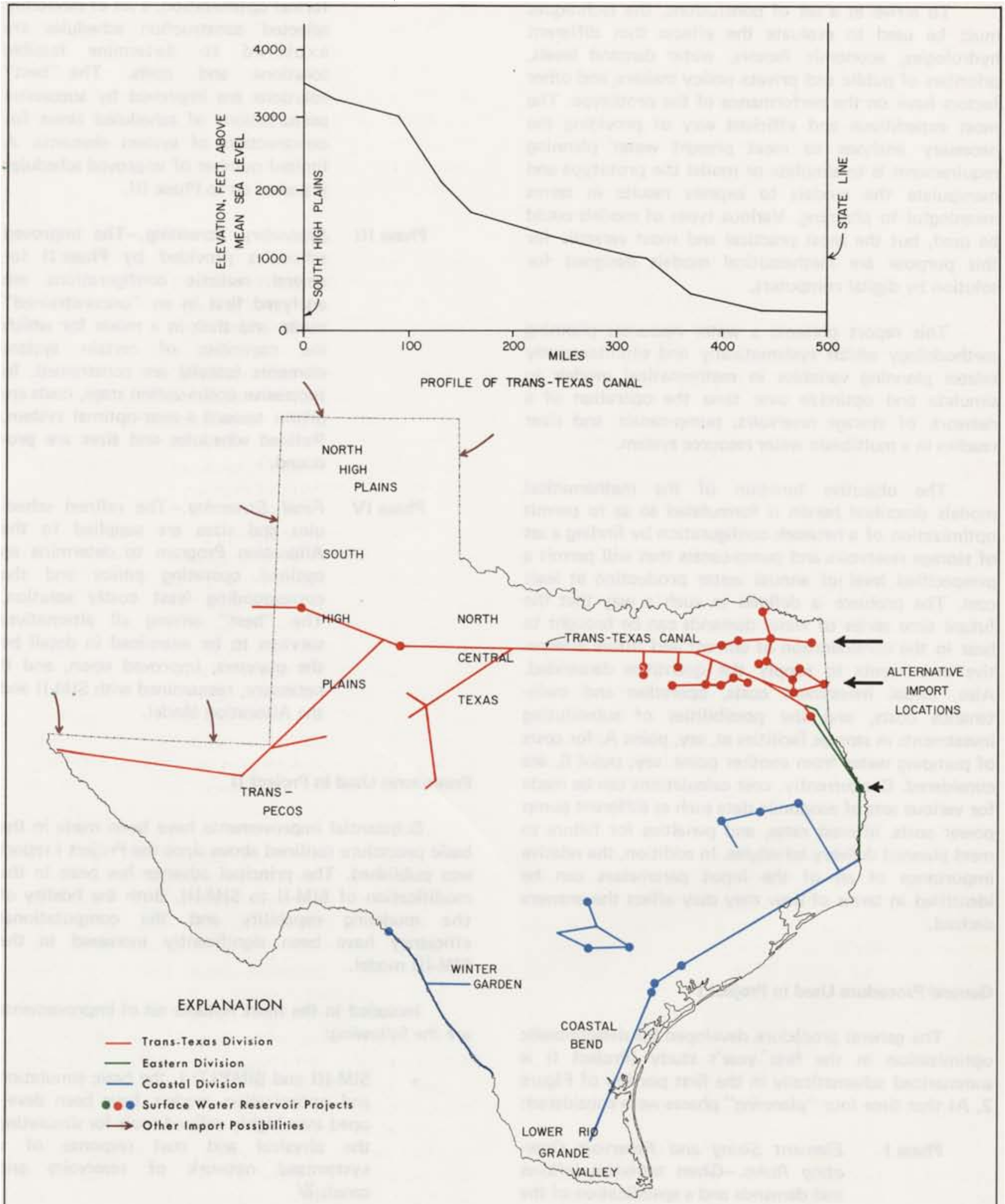



Figure 1

Schematic Diagram of the Texas Water System

(Includes major conveyance facilities and related reservoirs)

To arrive at a set of conclusions, the techniques must be used to evaluate the effects that different hydrologies, economic factors, water demand levels, priorities of public and private policy makers, and other factors have on the performance of the prototype. The most expeditious and efficient way of providing the necessary analyses to meet present water planning requirements is to simulate or model the prototype and manipulate the models to express results in terms meaningful to planning. Various types of models could be used, but the most practical and most versatile for this purpose are mathematical models designed for solution by digital computers.

This report presents a water resources planning methodology which systematically and simultaneously relates planning variables in mathematical models to simulate and optimize over time the operation of a network of storage reservoirs, pump-canals, and river reaches in a multibasin water resource system.

The objective function of the mathematical models described herein is formulated so as to permit optimization of a network configuration by finding a set of storage reservoirs and pump-canals that will permit a prespecified level of annual water production at least cost. The problem is defined in such a way that the future time series of water demands can be brought to bear in the consideration of current and future alternative investments to supply the quantities demanded. Also, initial investment costs, operation and maintenance costs, and the possibilities of substituting investments in storage facilities at, say, point A, for costs of pumping water from another point, say, point B, are considered. Concurrently, cost calculations can be made for various sets of economic data such as different pump power costs, interest rates, and penalties for failure to meet planned delivery schedules. In addition, the relative importance of all of the input parameters can be identified in terms of how they may affect the answers derived.

**General Procedure Used in Project I**

The general procedure developed for deterministic optimization in the first year's study (Project I) is summarized schematically in the first portion of Figure 2. At that time four "planning" phases were considered:

Phase I — *Element Sizing and Reservoir Operating Rules.*—Given tentative inflows and demands and a specification of the system configuration, a set of preliminary reservoir operating rules are determined by an application of the so-called Allocation Program.

Phase II — *Initial Screening.*—With operating rules given and the system configuration simplified to preclude the necessity for formal optimization, a set of randomly selected construction schedules are explored to determine feasible solutions and costs. The "best" solutions are improved by successive perturbations of scheduled times for construction of system elements. A limited number of improved schedules is passed on to Phase III.

Phase III — *Secondary Screening.*—The improved schedules provided by Phase II for several realistic configurations are analyzed first in an "unconstrained" mode and then in a mode for which the capacities of certain system elements (canals) are constrained. In successive optimization steps, costs are driven toward a near-optimal system. Refined schedules and sizes are produced.

Phase IV — *Final Screening.*—The refined schedules and sizes are supplied to the Allocation Program to determine an optimal operating policy and the corresponding least costly solution. The "best" among all alternatives survives to be examined in detail by the planners, improved upon, and if necessary, reexamined with SIM-II and the Allocation Model.

**Procedures Used in Project II**

Substantial improvements have been made in the basic procedure outlined above since the Project I report was published. The principal advance has been in the modification of SIM-II to SIM-III. Both the fidelity of the modeling capability and the computational efficiency have been significantly increased in the SIM-III model.

Included in the most notable set of improvements are the following:

- SIM-III and SIMYLD-I, the basic simulation and optimization models, have been developed into highly refined tools for simulating the physical and cost response of a systemized network of reservoirs and canals;[2]

[2] SIM-III is a substantially improved version of SIM-II, which was a product of last year's research (Project I). SIMYLD-I is a modified version of SIM-III used to compute individual or multibasin firm-yield information. Both SIM-III and SIMYLD-I are described in detail in Volume IIA of this report.

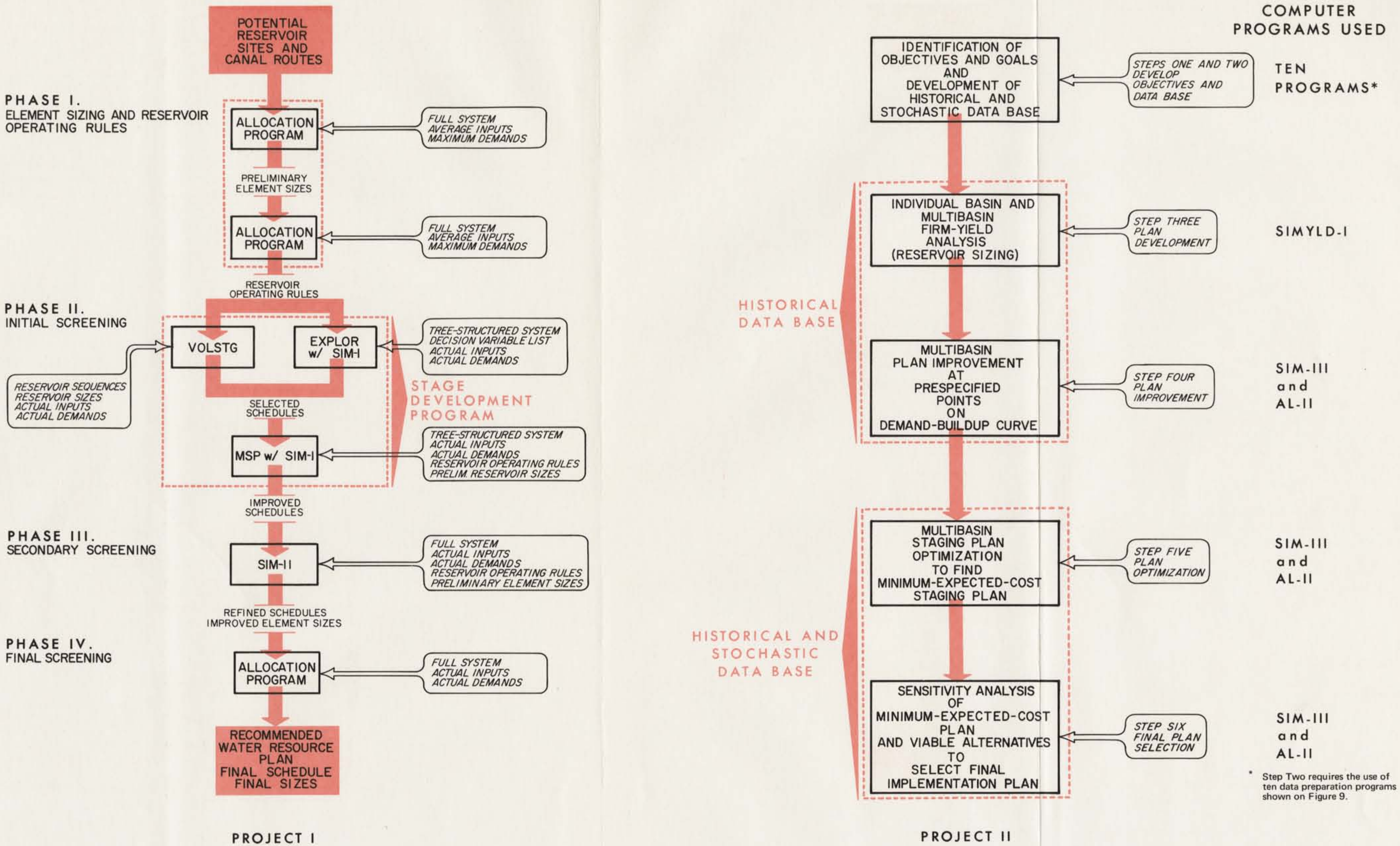



Figure 2
**Procedural Planning Steps for Projects I and II**

- a practical planning procedure founded upon a marriage of conventional practices and a systems analysis approach has been developed for identifying the extent to which a water resource should be utilized in order to meet demands for water with a proper amount of shortages; and

- a practical and efficient procedure for incorporating the analysis of risk into computer-based planning procedures on a comprehensive basis has been developed. This procedure is based upon using selected stochastic data sequences in conjunction with the historical sequence. This involves selecting from a stochastic data set a relatively small number of sequences that are representative of the results that would have been achieved by using the entire data set. Thus, for the first time, this procedure makes possible the use of extensive stochastic data sets in large and complex planning problems.

The second portion of Figure 2 illustrates schematically the general pattern of information and analysis flow developed for Project II. It also depicts the changes made in the planning procedures from those developed in Project I. The six planning steps used in Project II are briefly described as follows:

Step 1. *Identification of Objectives and Goals.*—This step outlines the problem to be solved, specifies in general the magnitude and location of demands to be met and the priorities associated with meeting each of the specified demands, identifies the sources of the water to be considered, and indentifies the criterion to be used in optimizing the selection of an implementation plan.[3] This step is further discussed in Chapter II.

Step 2. *Data Base Development.*—This step assembles an array of physical, economic, and hydrologic data describing the problem being analyzed. Both historical and stochastic hydrologic data sets are developed concurrent with the economic and physical data in the manner described in Chapters II and III.

Step 3. *Plan Development.*—Given the historical data base developed in Step 2, Step 3 analyzes each of the individual river basins to determine their firm-yield characteristics and the location of possible attractive basin import and export points. Either a firm surplus or a firm deficit is identified. This analysis is discussed further in Chapters II and IV.

Step 4. *Plan Improvement.*—Using the historical data base and the plans identified in Step 3, this step improves those plans such that minimum-cost plans are developed at each of several points on a prespecified demand-buildup curve. This step forms the core of the entire planning procedure and is described in detail in Chapters II and V.

Step 5. *Plan Optimization.*—Step 5 uses both the historical and the stochastic data base developed in Step 2, along with the results of Step 4, to optimize the staging plan over the time period when demands for water are increasing. In the procedure shown herein, this implies finding the minimum-expected-cost plan that meets the demands specified with an optimal level of shortages. The expected cost is an average present value cost developed by averaging the simulated cost response of several sequences of both stochastic and historical data. This step is described in Chapters II, VI, and VII.

Step 6. *Final Plan Selection.*—This is the final step in the planning procedure and involves testing the sensitivity of the cost and physical response of the simulated prototype to variations in all of the input parameters used by the models. This sensitivity information includes the measure of the impact that the stochastic variability of the supply and demand quantities has on the minimum-expected-cost staging plan identified in Step 5. Based upon this information and a measure of tolerable risk, the plan selected for implementation may be other than the minimum-expected-cost plan if the risk or variability for that plan is found to be unacceptably high. This step is described in Chapters II, VIII, and IX.

[3] The terms "implementation plan" and "staging plan" are used interchangeably in this report.

## Conclusions

The conclusions resulting from this research are grouped into two categories. The first category lists those that are general; i.e., not limited to the problem being analyzed. The second category lists those considered to be problem specific. These conclusions are not necessarily presented in order of importance. However, those due special note are stressed.

### General Conclusions

- The practicality of using systems analysis and optimization in the planning of large complex water resource systems has been demonstrated, and the methodology developed in this report is generally applicable to a wide variety of water planning problems involving complex multibasin interregional systems.

- The methodology developed, in which stochastic data generation, computer simulation, and gradient search optimization techniques are linked within a modular framework to provide a total algorithm, is computationally feasible for field-type water planning problems.

- Techniques for the generation of stochastic hydrologic and meteorologic data can be successfully combined with deterministic optimization techniques to assess the impact of hydrologic and meteorologic uncertainty on water resource planning decisions.

- Network optimization codes, such as the Out-of-Kilter Algorithm contained in all of the simulation models developed, provide an efficient and very effective means of optimizing water transfer in a complex multibasin network of reservoirs, demand nodes, and interconnective transfer canals.

- It is difficult, it not impossible, to use stochastically generated data directly in system-oriented planning models unless appropriate unbiased samples of representative stochastic data sets are selected in a manner identical or similar to that described in this report.

- Well defined system operating rules are essential to the effective use of the SIM-III model in simulating a complex mulitbasin system.

- The SIM-III model is now a highly refined modeling tool that, although fairly problem specific, has attributes that permit its conversion into a generalized modeling capability.

- Detailed simulation is necessary to simulate properly the physical and cost response of a large complex system. This is especially true if the primary variable driving the solution direction is the incurrence of shortages, because magnitude, duration, and frequency of occurrence of shortages are important variables in determining the tolerance of an irrigated agriculture to shortages. Only a detailed simulated response can generate this type of imformation. Further research will be required to develop a valid penalty function for quantifying responses to varying levels of shortage.

### Problem-Specific Conclusions

This set of conclusions was drawn from the testing and application of the procedures developed in the research efforts presented in this report. The procedures were applied to the planning problem associated with a simplified version of the Trans-Texas Division of the Texas Water System proposed in the Texas Water Plan, as described in Chapters IV through IX of this report. Conclusions drawn from this application are specific to the problem and associated data, and are listed as follows:

- The variation in the total cost that is attributable to the stochastic components in runoff, evaporation, and demand data is small.

- Economic responses are approximately normally distributed and are expected to fall within a range of 8 percent of the expected present value of the total cost. The 8-percent range corresponds to four standard deviations and includes 95 percent of the cost variation.

- Data on import costs, power costs, and total costs show relatively small variations.

- The position of droughts in time has little effect on total costs.

- Droughts of duration less than 42 months appear to be correlated with total costs and should have been considered in the sequence selection procedure. Although short-term droughts were not explicitly studied, a correlation study revealed that the shorter the drought the better the correlation between drought magnitude and total costs.

- Storage costs are the most significant cause of variation in total costs and of the variable cost components. This was revealed by a correlation study of the economic response data which also showed that the two other variable cost components, power and import costs, were weakly correlated to total costs. However, the variance in their costs is small and of little significance.

- The 18 hydrologic and demand sequences that were selected to study the effect of hydrologic uncertainty on costs were more than adequate. A procedure was developed and tested for selecting from a total set of stochastic data a subset which, when used as input to a detailed simulation program, yielded results representative of those which would have been obtained if the entire data set had been used. This methodology was used to select 18 hydrologic data sequences from a set of 99 for use in establishing the variability of system performance which could be expected to occur as a result of the stochastic variation in input data. Analysis of the 18 selected sequences indicated that they were, in fact representative of the total 99.

### Recommendations

It is recommended that the procedures and methodologies presented in this report be further developed and refined and that they be applied to additional water resource planning problems. Only additional research and applications will provide the necessary information upon which to generalize the problem-specific conclusions.

Although a workable approach has been developed, greater opportunity for future research exists in incorporation of improved techniques within each of the computer programs and application procedures. The needed improvements include:

- Improvement of the sequence selection procedures which permit selection of the least number of sequences consistent with computational accuracy requirements. These techniques will become apparent as more experience is gained in applying this approach to various problems.

- Incorporation of multilevel optimization and simulation procedures to permit varying computational precision from preliminary to detailed project planning. Time did not permit adequate evaluation of this aspect of the problem during this research project. However, much attention will be directed toward this item during Project III.

- Improvement of the currently available data fill-in and stochastic data generation procedures. Procedures currently available had to be modified for use in this research, but even with modifications, were found to be inadequate. Additional study on the means to preserve longer-term persistence characteristics of the generated data sets along with improved truncation criteria are needed to develop more representative data sets.

- Improvement of the current means by which demands for irrigation water are imposed upon the system. The current modeling procedures use a constant, prespecified acreage and cropping pattern upon which to compute the demands for irrigation water each year in the simulation period. Fluctuations in demands are caused solely by climatic variations because there is presently no capability to vary the amount of acreage planted according to expected supply conditions (i.e., existing storage plus expected inflow). This capability needs to be added to the Project II planning procedure and is scheduled for implementation during Project III. The impact of this improvement will be to make better use of an available supply, reduce the total amount of shortage incurred by a given system, and reduce the financial loss caused by shortages.

More attention should be given to defining problems where these systems analysis and optimization techniques can be effectively applied in the planning and design of real systems to identify needed modeling improvements. Better procedures are needed for explicitly evaluating water planning objectives in a form responsive to both the needs of water planners and those of systems analysts. It is imperative that experience in the application of procedures similar to those discussed herein be gained as soon as possible in widely separated areas with differing physical problems.

Economic analysis of the importance, impact, and tolerability of water shortages of varying magnitudes, durations, and frequencies of occurrence are needed. Specific attention towards the use of a nonlinear penalty function is needed in SIM-III for evaluating the impact of incurred shortages. Also, additional study of the uncertainty associated with economic parameters such as costs and benefits should be undertaken in actual planning studies.

Finally, a better methodology for evaluating both benefits and costs is needed, particularly in a format whereby they can be intergrated with the detailed computational procedures discussed herein.

Many conclusions were drawn that are specific to the demonstration problem considered in the research.

Only additional research and applications will provide the necessary information upon which to generalize the problem-specific conclusions. Toward that end, it is recommended that:

- Additional data fill-in and generation techniques be used to determine if they influence the impact of hydrologic uncertainty on project costs.

- Droughts of duration less than 42 months be considered in the analysis.

# II. THE PROBLEM AND A SOLUTION METHODOLOGY

## The Problem

Over the past decade engineers and economists, to the best of their ability and within the limits of computational capabilities, have defined meaningful advanced planning procedures and supporting analytical techniques. Largely, their use has resulted in planning strategies that depend upon the fast computational capability of the electronic computer to evaluate deterministically a myriad of plans and operational criteria identified as attractive by the planner in his attempt to find the optimal.[4] (Maass and others, 1962; Chow and Meredith, 1969a, 1969b; Hall and Dracup, 1970).

These activities have had good results, are generally well accepted, and represent a major step in improving man's capability to accumulate knowledge and gain insight about the problem he is trying to solve before having to make, in many cases, irreversible decisions about the destiny of a limited water and related land resource.[5] Terms such as minimum cost, maximum return, and maximum net benefits have emerged, and have been used extensively as the basis for quantifying optimality.

However, in the process of developing evaluative criteria for finding the optimal, planners have paid little attention to quantifying explicitly the impact that risk and uncertainty have on the decision process. For example, the hyrdologic risk portion of the planning process has normally been included implicitly in prespecified assumptions; thus, many water supply reservoirs are designed to meet projected demands in all droughts that, on the average, occur more often than once in any 50-year period. A priori value judgments such as this, for the most part, are based upon what is expected to be conservatively adequate and not necessarily on what is expected to be economically efficient, or optimal. Thus, the impact that various drought characteristics have on economic benefits, plan performance, or user repayment capability, in many cases, is assumed away in the probability of exceedence assumption, or more basically, in an improperly stated set of objectives.

Similarly, many times, variables that can affect the decision process the most (e.g., capital costs, power costs, interest rates, etc.) are assumed fixed when using models to help find an answer to the problem being studied. In other words, the impact that uncertainty and modeling assumptions have on planning decisions, in many cases, is assumed away or, at best, inadequately analyzed.

[4] Herein, "optimal" is defined as "most reasonable." Because of the characteristics of the problem, no rigorous optimal solution is likely.

[5] In this report, "water resource systems" is considered to include all related land resources.

The methodology presented herein seeks to find water resource development plans that satisfy a set of prespecified goals at minimum total expected cost[6] while striving to find plans that reduce the impact of risk and uncertainty. It was specifically developed for use in planning, design, and operation of a complex water resource system such as the proposed Texas Water System. However, because of the nature of the problem, the development activity created analytic procedures that should be generally applicable to many problems and related areas where

- stochastic variability in both the supply and demand is considered to be important in selecting design alternatives,
- detailed system simulation is considered necessary to properly represent the prototype,
- a large buildup in demands for water is contemplated over a long planning period,
- the major demand areas are significantly separated from the supply areas, and
- an import source can be utilized to help meet expected future demands for water.

It is stressed that the methodology is of limited value where the system's performance is insensitive to differences among various stochastic data sets. However, in many hydrologic systems, the sensitivity to stochastic variations is great, and a large number of probable occurrences must be used in order to adequately evaluate the range of possible system performances. It is in such applications that the methodology described herein is of great value.

In essence, the material presented herein suggests that explicit treatment of risk and uncertainty should be incorporated into the analysis of complex systems, and presents a generalized framework for accomplishing this efficiently at a detailed level of modeling simulation.

Stochastic data sets, by their very nature, are voluminous and often prohibitive to use in detailed modeling studies because of the tremendous data management and computational burden involved. It has only been with the advent of modern computational methods that the planner could begin to explore more or less

[6] Herein, "total expected cost" is defined as the sum of capital costs and annual costs. Capital costs are comprised of reservoir, canal, and pump station construction costs. Annual costs include reservoir and canal maintenance costs, energy for pumping costs, import water costs, and shortage penalty costs incurred and computed on a monthly basis. As appropriate the present value of these costs viewed from the start of the planning horizon is used to compare costs responses of various plans.

systematically the effects of stochastic processes on his decisions and to introduce the elements of risk and uncertainty into his analysis in a more rigorous way.

Typically, the data available on the stochastic nature of hydrologic system inputs consists of a limited set of observations (e.g., monthly rainfall, runoff, and evaporation data at desired locations in a river basin). Very rarely is there considered to be a sufficient period of record available (even after missing data are estimated) to span all possible ways that the phenomenon being analyzed might occur. Similarly, little attention is normally given to evaluating the relative accuracy or degree of uncertainty associated with each set of recorded observations in light of how its use may affect the solution obtained.

It is known, however, that there is a large amount of information contained in recorded data that is not effectively used when using the data only in their original order of observation. Many other possible orderings of the same data, in conjunction with other magnitudes of each data event, can be statistically inferred from information contained in the available recorded data. Therefore, it is slowly becoming accepted practice in hydrologic studies, to examine the cross-correlation, auto-correlation, and other important statistical characteristics (e.g., mean, standard deviation, and skew) of the recorded data for several related data types, locations, and intermittent periods. It is also becoming common to develop, from these statistical relationships, models that fill in the missing data and then generate for further analysis any number and length of related stochastic data sequences (Beard, 1965).

If the model is properly developed and applied, the resulting filled-in and stochastic data sequences, by definition, contain statistical properties similar to those data observed and used as input to the model. Both the magnitude and the order in which the data occur within these generated sequences are computed to reflect other equally likely ways in which observations of the past might occur in the future. By using detailed simulation models to evaluate (1) the single filled-in sequence, and (2) numerous equally likely stochastic sequences, it is reasoned that the entire range of significant system responses can be simulated. Similarly, the system's expected performance (average) and the range of its performance can be evaluated.

While it is generally wise to evaluate the impact caused by numerous[7/] equally likely sequences in order to

- adequately evaluate expected performance, and
- be aware of the impact that extreme conditions contained in both the observed and the stochastic data sets have on system performance,

it is usually unwise or economically inefficient to select a design that adequately performs during the worst possible observed or stochastically generated conditions unless the penalties for inadequate performance are extremely severe.

Therefore, to properly evaluate inherent trade-offs in most hydrologic systems, it is necessary to define and select a criterion to help decide which of the various levels of severity must be handled adequately, or conversely, what degree of inadequacy can be tolerated.

In hydrologic systems designed to meet prespecified demands at a given level of tolerable shortages, it is not easy to prespecify optimal (1) magnitude of tolerable shortages, (2) frequencies of tolerable shortages, or (3) the magnitudes versus frequencies of tolerable shortages.

Also, the many problems identified in a planning process are normally very complex and interrelated; thus, they should not be resolved independently. There are definite trade-offs between the capital costs of canals and reservoirs and the operation costs of transferring water. If a reservoir is constructed too small or too late, or both, system operation costs may be forced upward by the need to meet demands by supplying water from a remote reservoir or through a canal with a higher pumping cost. As a consequence of this interdependence, the whole assemblage of discrete problems must be considered concurrently in an organized stepwise manner in order to obtain realistic answers.

Regardless of how much analytic capability and information a water resource planner can usefully organize and analyze, he must ultimately select an implementation plan. The selected plan must, by definition, include definite facilities of specified sizes and purposes to be constructed at specific times during the planning period. Therefore, with this in mind, the methodology is designed to help the planner consider the risk and uncertainty inherent in the problem, and help answer the four following questions:

- Which of an over-specified set of reservoirs and canals should be constructed?
- When should each of the reservoirs and canals be constructed?
- How large should each of the reservoirs and canals be at various points on the demand-buildup curve?
- How should the resulting optimized system of canals and reservoirs be operated, both

[7/] Twenty or thirty sequences are considered to constitute a small sample and approximate the minimum number of independent observations required to make reliable statistical estimates.

during and after the period in which facilities are being added or increased in size?

The methodology is comprised of six major steps (as shown in Figure 2), none of which is different from current water resource planning analyses, but which collectively represent a more systemized and thorough analytic treatment of the risk and uncertainty associated with the problem and the decision process than was previously available. These six steps essentially provide the framework of this report and the basis for answering the four basic questions itemized above.

## A Solution Methodology

The purposes of this section of the report are to discuss, in general, each of the six major steps of a solution methodology (Weiss and Beard, 1970) shown in the second portion of Figure 2 and to provide an introduction to their detailed explanation in subsequent chapters in this report.

### Step One—Identification of Objectives and Goals

Step One consists of identifying the goals to be met and the purposes to be served. This is perhaps the most difficult job of the planning process, but is the most important, and must be done before a solution can become obvious or an optimal implementation plan can be found. It is suggested that the planners using this strategy should (1) specify an objective or goal that serves the purposes defined as important in their orders of priority, (2) strive to find the optimal implementation plan to meet the goal, and (3) then decide, based upon the trade-offs present and risks involved, if the optimal development plan or a modified version of it representing a lower risk plan should be implemented. Meeting demands at minimum expected cost, with tolerable shortages, is only one of the possible objectives that normally could be specified; however, for the purposes of this report and the modeling capability, it is satisfactory for demonstrating the worth of explicitly evaluating risk and uncertainty in the planning process.

### Step Two—Analysis and Development of Data Base

Step Two consists of developing a comprehensive data base (a tape file) for use in Steps Three through Six. This step is comprised of two major types of data preparation activities. The first activity is that of developing, for use by the simulation and optimization models, a sound historical and stochastic hydrologic data base comprised of

- refined runoff or reservoir inflow data,
- gross evaporation or climatic index data,
- net lake-surface evaporation data developed from rainfall data and gross evaporation data,
- irrigation water requirements developed by a consumptive use model, and
- municipal and industrial water requirements.

The second activity is comprised of developing parameters which describe the system and the problem being studied, such as

- Problem title information,
- cost-capacity-elevation-area relationships for all of the reservoirs and canals being considered in the analysis,
- the interest rate, repayment period, reservoir financing lag time, and pump-canal financing lag time used to calculate present value costs of capital investment and operation and maintenance costs, and
- data describing the physical and other characteristics of the system being analyzed.

From the hydrologic viewpoint, this step, if done correctly, involves considerable effort in the detailed refinement of basic surface- and ground-water data at various projected levels of basin development (e.g., 1980 conditions, 2000 conditions, etc.).

To enhance the results of this step, trend analysis programs, fill-in programs, stochastic data generation programs, and flow refinement and projection programs are used to help preserve the appropriate cross and serial correlations within all of the data sets, and thus develop a sound comprehensive data base at various levels of basin development for all subsequent steps in the planning and design process.

One of the unique characteristics of this methodology is the treatment of the element of risk (the stochastic element) in both the runoff and the demands for water. Therefore, in addition to using a refined historical filled-in data set, a large number of stochastic data sets (e.g., 98) of rainfall, runoff, evaporation, and unit demands for water, are required. For the example problem, 36-year historical and stochastic data sets were used. The 36-year period corresponds to the demand-buildup period (1985-2020) as shown in Figure 3.

The need for treating risk and uncertainty in this manner arises from the recognition that in many irrigation service areas significant useful amounts of rainfall occur during many years. That rainfall reduces the amount of irrigation water needed to serve a given

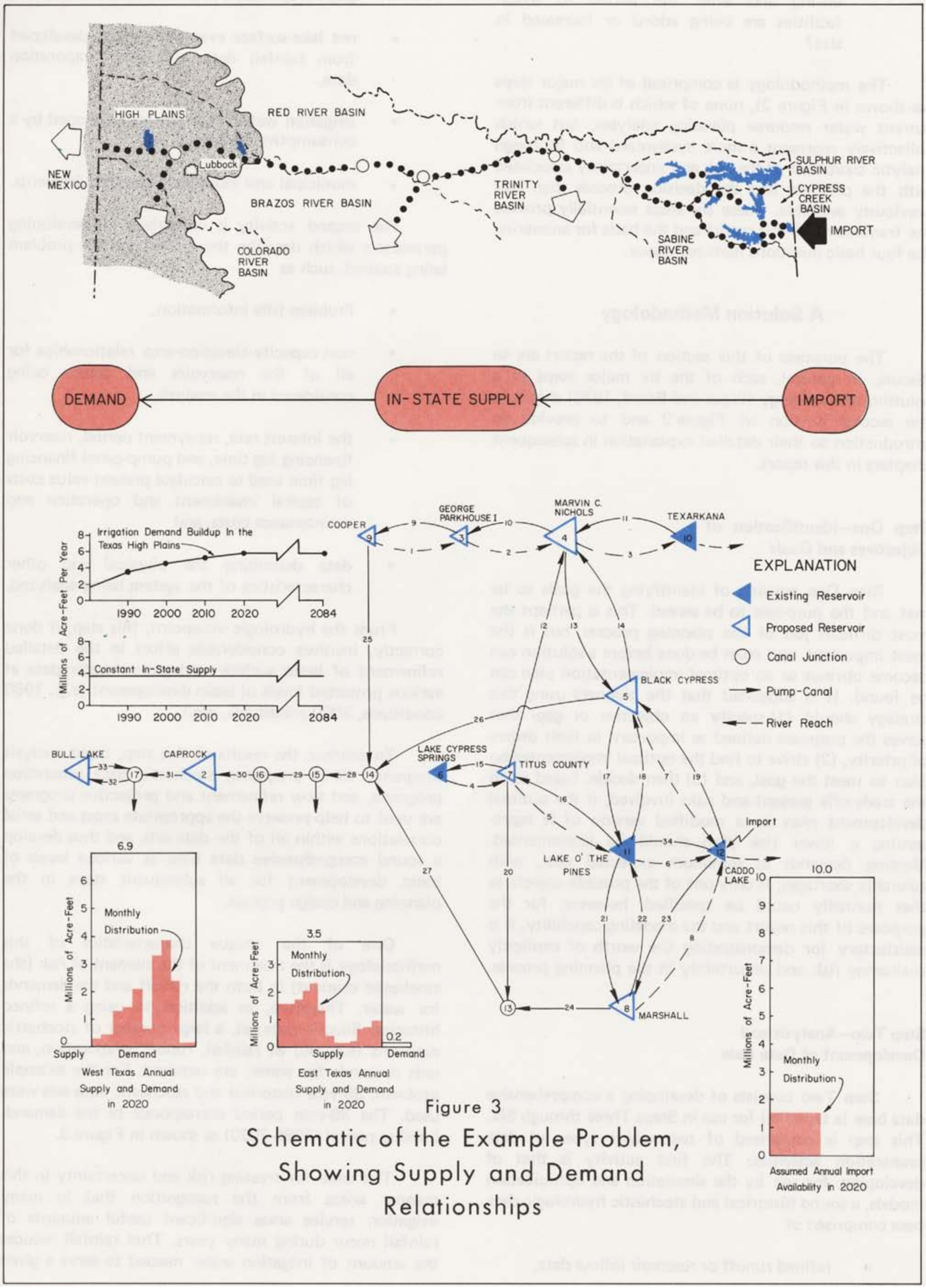



Figure 3

Schematic of the Example Problem, Showing Supply and Demand Relationships irrigated acreage, and thus, has an impact on the efficient design and operation of the required storage and transfer facilities. Since rainfall contains a stochastic component and there exists a definite deterministic relationship between rainfall and the need for supplemental irrigation water, a straightforward method to transpose the stochastic characteristic of the rainfall data to the demands for water is to use a soil moisture and consumptive use model, along with rainfall data, gross pan evaporation data, and other soil and cropping data and irrigation efficiency to generate monthly unit-acre irrigation demands.

Because the procedure is structured on a typical cropping pattern and a unit-acre basis, the results must be multiplied by the number of acres within each irrigation subdistrict. The total demands for each subdistrict (plus losses) must then be summed to get the actual total monthly demand for irrigation water at each demand point within the network structure of reservoirs and canals.

For the demand points within the Texas High Plains this procedure results in a demand sequence that varies about a trend line as shown in Figure 4. The trend line is a direct function of both the number of acres that are irrigated with surface water and the average annual rainfall contributions, whereas the jagged line represents the actual water usage based upon rainfall and evapotranspiration stochastic variability. The trend line shown in Figure 4 is comprised of the average stochastic irrigation demand plus a non-stochastic municipal and industrial demand quantity.

The supply also has a stochastic component. The variability of that component may be as great or greater than the demand variability, depending on the characteristics of the problem. An indication of the relative variability of the demand and supply is given in Figure 4 for the 36-year demand-buildup period. Close inspection of the data supporting Figure 4 will reveal that after about year ten, the average supply is insufficient to meet the average demand. Therefore, for most of the time during the demand-buildup period, import water is required to meet, on the average, demands for water imposed by a prespecified irrigated acreage.

Net lake surface evaporation data are also computed. This is done using the rainfall and gross pan evaporation data for both the supply and terminal storage reservoirs.

**Step Three—Plan Development Based on Historical Data**

Step Three consists of a "first-pass" analysis of the river basins and portions of river basins comprising the multibasin planning problem. The purposes of this analysis are to

- determine how best to control the available runoff,
- compute the amount of water that the system can be expected to yield,
- determine preliminarily how to develop the best set of storage and transfer facilities to move available supplies to use areas, and
- determine preliminarily the magnitude of the demands that can be met with the available supply.

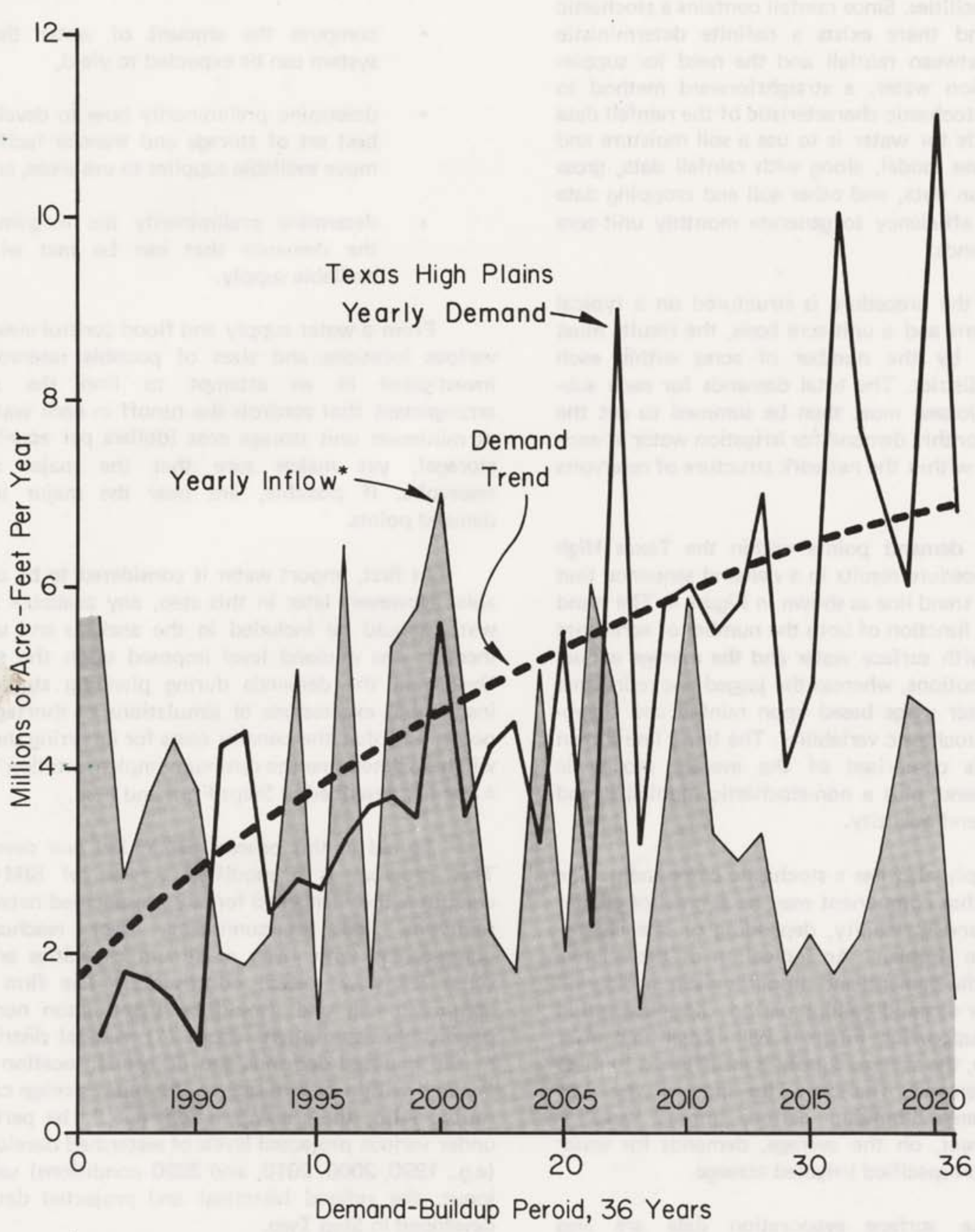
From a water supply and flood control viewpoint, various locations and sizes of possible reservoirs are investigated in an attempt to find the storage arrangement that controls the runoff in each watershed at minimum unit storage cost (dollars per acre-foot of storage), yet makes sure that the major storage reservoirs, if possible, are near the major in-basin demand points.

At first, import water is considered to be unavailable. However, later in this step, any available import water should be included in the analysis and used to increase the demand level imposed upon the system. Oversizing the demands during planning studies will insure that expressions or simulations of shortages will occur, and that the penalty costs for incurring shortages will help determine the optimum implementation plan in a manner described in Steps Four and Five.

To aid in this process SIMYLD-I was developed. This program is a modified version of SIM-III. It computes the firm yield for any prespecified network of reservoirs and interconnecting river reaches and pump-canals with given maximum capacities and seasonal low-flow release constraints.[8] The firm yields computed can and should be based upon numerous practicable assumptions about (1) seasonal distribution of the imposed demands and (2) spatial location of the demand within or external to the basin storage configuration. Also, these computations should be performed under various projected levels of watershed development (e.g., 1990, 2000, 2010, and 2020 conditions) using, as input, the refined historical and projected data base developed in Step Two.

A set of reservoirs in the supply basins, having specific locations and sizes such as those shown in Figure 3, is a partial result of this step.

[8] SIMYLD-I and SIM-III are detailed simulation programs with built-in flow optimization criteria. These programs are used to help the planner evaluate proposed prototypes and fine the optimal. They are described in more detail in Volume IIA of this Completion Report. Also, the basic concepts behind these models were presented in Chapters V and VI of Texas Water Development Board Report 118.




*The sum of unregulated inflows to the reservoirs comprising the example problem; not including import.

Figure 4

**Stochastic Variability of Inflow and Demand**

**Step Four—Plan Improvement Based on Historical Data**

Step Four uses SIM-III and AL-II[9] to help find "good" fixed plans at various demand levels (e.g., the 1990, 2000, 2010, and 2020 levels) using the refined historical data base as projected to various future times on the demand-buildup curve. This analysis is based on evaluating system performance of selected alternative sets of canals, reservoirs, and operation criteria over a prespecified economic life. SIM-III and an analysis period (e.g., 36 years) equal to the time period over which demands are increasing are recommended; however, the procedure presented is independent of the length of simulation period used as long as it is of sufficient duration to generate a realistic total-cost response of the system being simulated.

For the example problem, finding the optimal development plan begins with analyzing the full-development conditon (e.g., the 2020 condition). This is done to obtain an approximate size and shape of the ultimate system configuration, especially the size of that portion of the canal facility (the ditch portion) that cannot be increased in size (staged) over the demand-buildup period shown in Figure 4, but must be built initially at 2020-level size. In the example problem, it was assumed that the ditch portion of the canal included right-of-way costs, relocation costs, bridge costs, pump-station foundation costs, ditch excavation and living costs, and associated items; the pump, motor, power, and their housing components are the portions of the canal facility that can be staged. The 36-year historical hydrologic sequence projected to 2020 conditions (Figure 4), and the 2020 level-demand data, developed in Step Two, are used as input to SIM-III along with a whole array of physical costs, and control parameter data. The over-specified network of potentially attractive canals and river reaches shown in Figure 3 is also used.

Based upon a series of "first-try" simulations of the entire network, with each canal's maximum capacity set at a relatively high value, the models compute

- the amount of usage that each of the canals would get during the 36-year simulation period,
- the absolute maximum flow in each of the canals, and
- the ratios of maximum to mean flow in each of the canals.

[9] The AL-II computer program is described in Volume IIC. The concepts behind this model are contained in Chapter VI of Texas Water Development Board Report 118. Its purposes are to develop initial estimates of required canal capacities and reservoir operation rules using average hydrology and demands equal to those which occurred in the most critical year in the period being analyzed.

Based upon these observations and the change in the economic response of the system (i.e., the total-cost change) resulting from the iterative use of SIM-III and AL-II, certain canals of very low usage can be eliminated from further consideration. The maximum-capacity constraints of each of the canals left in the network can be successively reduced, from simulation to simulation, to levels that approach a minimum-cost solution. Here, the total-cost response is the sum of (1) the construction costs multiplied by a present value factor equal to unity, and (2) the average annual operation costs multiplied by the total area under the 100-year present value curve shown in Figure 5 (e.g., 24.50).

Upon preliminarily sizing the ultimate ditch portion of the canal facility, the analysis is directed towards finding an optimal system (location, size, and operation criteria) for prespecified points on the demand-buildup curve starting with the earliest point first. At each of the demand points, SIM-III is used in the user-oriented iterative manner based upon the steepest-gradient-search philosophy discussed in detail in Chapter V. The point of observation for measuring the economic response of the system at each of the demand points (e.g., 1990, 2000, 2010, and 2020) is the beginning of the planning or construction period (e.g., 1985). Again, a 100-year economic life, a 4 percent discount rate, and a 36-year simulation period are used in the demonstration problem.

The need to prespecify staging time increments is basic to the analysis procedure. The time increments need not be equal. In fact, their lengths should be based upon an analysis of the shape of the demand-buildup curve, the shape of the present value curve, shortage-penalty costs, excess-capacity costs, and the greater cost of constructing facilities in stages instead of constructing them to their ultimate size initially. A more detailed discussion of the selection and use of near-optimal staging increments along with a more detailed treatment of the entire plan optimization process is presented in Chapter V.

**Step Five—Plan Optimization Based on Historical and Stochastic Data**

Step Five is designed to analyze and improve the "good" but sub-optimal plans derived in Step Four, using both the historical and selected stochastic sequences of hydrologic and corresponding demand data generated in Step Two. Step Five is also designed to

- quantify the impact that location of drought within the demand-buildup period, in addition to magnitude, duration, and frequency of drought occurrence, has on selecting the optimal implementation plan,
- quantify what changes in the "good" plans derived in Step Four are required to cause more cost-effective performance, and

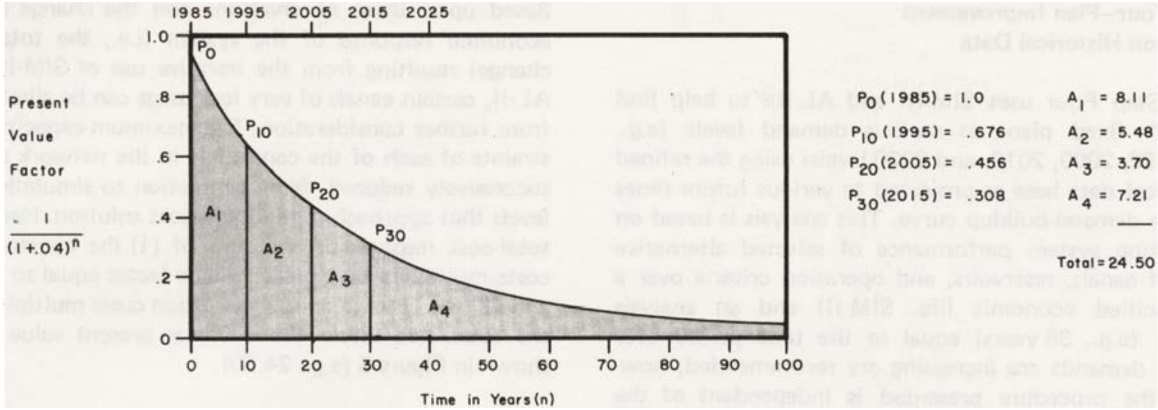



**Figure 5.—Characteristics of the 4 Percent Present Value Curve**

- find the single implementation plan (the minimum-cost plan) which performs better against the historical and synthetic buildup in demand and projected supply sequences than any other plan.

The first portion of this step is comprised of selecting a repersentative few of the 98 or more synthetic supply and demand sequences generated in Step Two for use with the simulation and optimization models. This is done using a procedure that

- analyzes the specific drought characteristics of the historical sequences plus each synthetic sequence,
- categorizes the set of 99 or more sequences[10/] into selected subsets according to their drought characteristics, and
- selects, in a manner to reduce small-sample bias, a representative few of these sequences that closely approximate the variability contained in the 99 sequences.

In order to select a small representative number of sequences from a large number, it is desirable to determine a single characteristic of the sequence that substantially influences the performance of the system. If there is more than one important independent characteristic, it is necessary to classify sequences on the basis of each characteristic. For example, for hydrologic systems there exists the strong conviction among many planners that the *magnitude of the most critical drought* within a sequence is an especially important characteristic. Another important characteristic is the *location of the drought* within the sequence, if, over time, the staging of facilities to meet an increasing demand for water is to be analyzed. The *duration of the drought* is also important in influencing the impact of the drought on system performance.

Although three important drought characteristics (magnitude, location, and duration) were identified for the example hydrologic system, it was found that the three-dimensional problem could be reduced to an equivalent two-demensional problem by preselecting a limited range of critical period durations found to control the selection of an optimal plan. Thus, the magnitude of the most critical drought within each sequence and its location within the sequence were used as the two characteristics controlling sequence selection.

Based on a selection strategy discussed in Chapter VI, 18 sequences (17 stochastically generated sequences and one historical sequence) were selected for use as input to SIM-III to help find a minimum-expected-cost plan. In this step, SIM-III is used to simulate through the demand-buildup period, and through a sufficient number of years of the ultimate-demand-level (2020) plan, to generate a present value cost of system performance both during (1985-2020) and after (2021-2084) the demand-buildup period. As in Step Four, a 100-year economic life and a 36-year simulation period are used.

In this analysis, location of drought during the demand-buildup period is important to the success and meaningfulness of the solution; therefore, multiplication of each year's simulated annual costs by corresponding present value factors is used to compute the total present value of annual costs. The capital expansion costs incurred at the various staging points are, of course, also multiplied by the appropriate present value cost component. For the years after the demand-buildup period (2021-2084), the simulated average annual cost component is multiplied by the area under the last 64 years of the present value curve shown in Figure 5 (i.e., 5.60 for the 4 percent case). The three present value cost components for all 18 sequences are added together and divided by 18 to compute the average total

10/ A total of 99 sequences is comprised of one historical sequence and 98 stochastically generated sequences. The number 99 was selected such that, when the sequences were subdivided into three categories as described in Chapter VI, an equal number of sequences would be allocated to each category. Furthermore, 99 sequences form a rather large sample designed to eliminate the effect of small-sample bias.

present value cost. This total cost is then used as the single measure of cost response for finding the minimum-expected-cost implementation plan. A more detailed description of the methodology used to compute the above values and, thereby, find the minimum-expected-cost plan is discussed in Chapter VII.

**Step Six—Variability and Sensitivity Analysis**

Step Six is the last major step in the multibasin planning strategy discussed herein, and consists of an extensive variability and sensitivity analysis. The purpose of this analysis is to subject the minimum-expected-cost plan found in Step Five to conditions other than the prespecified "best estimate" conditions assigned to many of the independent variables at the beginning of the analysis. In essence, this step involves evaluating the economic and physical performance of the minimum-expected-cost plan by taking a single-variate cross-section on every variable supplied as input to the SIM-III program. Similarly, multivariate cross-sections are also taken where the results can be meaningfully interpreted. Typical data varied include the canal cost data, the reservoir cost data, the initial storage conditions, the buildup rate in the number of acres to be irrigated, the cropping pattern data, the mean available water supply, the municipal and industrial demand levels, the amount and time at which import water is available, the mean of the evaporation data, and the unit power cost data. The results of the analysis performed on the demonstration problem and the insight gained during its conduct are presented in detail in Chapter IX.

## The Major Modeling Capabilities and Inherent Assumptions

It has been stated that modeling is the process of approximating the prototype for the purpose of evaluating the performance of the prototype. Because prototypes are generally more complex than models can ever be, certain simplifying modeling assumptions must be made in order to provide a practical and cost-effective problem analysis capability.

Because this is phase two of a three-phase model development project, it was necessary to limit the breadth of the study in order to provide for an orderly implementation of the eventual modeling capability. Additional modeling capability will be implemented during next year's (Project III) research.

At the conclusion of last year's research (Project I), considerable limitations existed in the resulting modeling capability (SIM-II and AL-I 1/). In striving to use the models during the first portion of this year's work these limitations were identified and considerable added capabilities were found to be required. Therefore, SIM-II and AL-I have been considerably improved and renamed as SIM-III and AL-II. These models are also considerably more efficient computationally than they were at the conclusion of the first year's research.

Therefore, in order to describe the current modeling capability in relation to last year's capability, the following list of SIM-III capabilities is presented. Those items that represent an added capability are identified with a triangle, whereas those capabilities carried over from last year, without modification, are identified with a circle.

- ● Only mass balance quantitative surface water is to be modeled. That is, no water-quality parameters or conjunctive use of ground water is included in the modeling capability.
- ● Monthly time increments are used in simulating the system; thus, operations of canals and reservoirs for routing flood waves are not considered. Therefore, the total travel time within the system must be less than one month.
- ● The models are capable of analyzing a network configuration of reservoirs, pump-canals, and river reaches interconnected in any possible manner.
- ● The resolution of modeling accuracy is currently set at 1,000 acre-feet as controlled by the resolution of the input data.
- ▲ Both a "perfect knowledge" and a "forecast" version of modeling capability are available. By definition, the "perfect knowledge" capability looks one year ahead at the input data prior to solving the problem for that year, whereas the "forecast" capability does not look ahead at the data prior to solution.
- ▲ Two options are available upon which to optimize monthly internodal water transfers. One uses only unit pumping costs; the other uses the unit pumping costs plus prorated capital costs to calculate total unit cost to pump.
- ▲ Lower constraints can be set on demands to reflect, at each node, how much of a prespecified demand must be met regardless of the magnitude of shortages incurred. If the lower bounds are set too high, an infeasible solution may result.
- ● Staging increments of ten years will be used in the general analysis.

1/ AL-I is the "Allocation Program" described in Texas Water Development Board Report 118.

- ● All demands for and inputs of water are prespecified except for import waters. The maximum available import water will be prespecified. In other words, runoff, evaporation, system losses, and demands for water are forced upon the system, but import water is drawn upon only when needed.

- ▲ Demands for water, reservoir inflow quantities, and evaporation rates are capable of being varied on a month-by-month basis to permit accounting for a demand buildup, a runoff depletion, and stochastic variability in all of these quantities.

- ▲ The methodology is capable of handling a problem with a 100-year economic life and a 36-year simulation period (e.g., 1985 to 2020).

- ▲ A minimum cost objective function is used in conjunction with penalty cost functions. If these two parameters are properly used, a criterion for maximizing net benefits can be imposed.

- ▲ Unit penalty costs for incurred shortages can be varied by node by season, whereas storage arc pricing preferences can be varied by reservoir by season.

- ▲ Because an economic objective criterion is specified, a prespecified economic value for meeting demands versus the economic value of spilling water and the economic value of storing water is required. Therefore, it is assumed that demands for water will be met only if the value for meeting demands is greater than the penalty for not meeting them. The value of having water in specific reservoirs on a seasonal basis can be specified, but it is not permitted to interact with the value of meeting demands.[12/] Spills out of the system are, by definition, the most expensive alternative use of water. Therefore, spills will occcur only as a last resort.

- ● The physical system can be represented by a set of interconnected nodes and links. Links correspond to river reaches and lengths of canals, while nodes represent reservoirs and link junctions.

- ● All water demands and inputs can occur only at nodes.

- ● Canal evaporation can be estimated for long reaches and withdrawn at nodes.

- ● Canal seepage losses can be estimated for long reaches and withdrawn at nodes.

- ● Import can occur at any one storage or non-storage node in the system during any limited part of the year up to the maximum monthly availability that was prespecified.

- ▲ The maximum amount of import water available can be changed at any yearly interval with a maximum of four different levels being permissible. However, a constant seasonal distribution of the available import water is assumed.

- ▲ Those reservoirs that are capable of accepting import water can be specified as a means to control the amount of water imported and its location of interim storage.

- ▲ Both reservoirs and canals can be added to the network of active facilities at any given year in the simulation period. A maximum of four sizes (stages) can be specified during this period. This can be increased with little trouble.

- ● Both minimum and maximum flow and storage capacities can be specified for canals and reservoirs, respectively.

- ▲ Canal costs are divided into two components—that component which cannot be staged (e.g., ditch, right-of-way, and related costs) and that component which can be staged (e.g., pump, motor, power, and housing costs). For those staged components, the simulation model is capable of imposing a penalty cost for capital expansion, expressed as a percentage of the capital expansion cost.

- ▲ The preference to pump upstream from a reservoir, instead of releasing water downstream when the reservoir is overflowing, can be specified on a link-by-link basis.

- ▲ The transmission capacity of the ditch portion of the canal facility at the last year of the simulation period can be larger than the actual pump capacity of the canal facility.

- ● Spills out of the system (system losses) can be controlled to occur at only those reservoirs specified as spill nodes.

12/ By design, the third phase of this project will permit the value of storing water to interact with the value of meeting demands on a monthly basis.

- Initial storage contents of all reservoirs are specified and can be varied by reservoir.

## The Example Problem

The methodology described herein is tested and applied to an example problem associated with a portion of the Trans-Texas Division of the Proposed Texas Water System shown in Figure 1. The portion of the Trans-Texas Division that is used in the example is comprised, as illustrated schematically in Figure 3, of three major components; a major demand area lying primarily in the High Plains of West Texas, an in-state supply area comprised of two river basins in East Texas, and an out-of-state source of water which may be drawn upon to meet required demands in excess of in-state supplies. A distinguishing feature of this system is its overall size; more than 700 miles separate the major demand centers from the sources of imported water. In addition to the hundreds of miles of interconnected canals and natural waterways, there are numerous reservoirs in the system. Pumping facilities will be required to lift flows through about 3,200 feet of elevation from near sea level to the High Plains of West Texas.

The system has the following unique characteristics which further complicate the planning problem:

- the potentially developable terminal storage sites in the demand area are limited,
- the only sources of water supply in the major demand area (West Texas) are ground waters and these are being rapidly depleted,
- the potentially developable reservoir sites in the in-state supply basins have a cumulative capacity to supply the maximum system demand for only a single year of operation,
- the surface-water supplies of in-state basins and the demands for water are highly variable, both seasonally and annually,
- the proposed sources of imported water can be drawn upon for only a portion of the year, and
- the maximum demands on the system may be expected to occur during the months when import water will not be available and runoff is low, hence peak demands must be met primarily from stored supplies.

For the example problem, an over-specified set of 12 reservoirs, 26 pump-canals, and 8 river reaches has been identified that might be necessary to accomplish the desired water transfers. The objective is to find which combination of these reservoirs and canals will satisfy a set of prespecified demands for water at minimum total expected cost, where both the demands for water and the surface-water inflows to reservoirs (supply) have stochastic components.

Of the 12 reservoirs considered, 3 are existing, and 9 (including one enlargement of an existing reservoir) are proposed to be constructed during the period of demand buildup. The eastern part of the network encompasses the Sulphur River and Cypress Creek basins. The eastern portion extends a distance of nearly 200 miles between reservoir 12 (on the eastern boundary of Texas—see Figure 3) and junction 14 (just north of the Dallas-Fort Worth area). Westward of junction 14, a large proposed pump-canal carries water over a distance of nearly 400 miles to two large terminal reservoirs, Caprock and Bull Lake (reservoirs 2 and 1) in the High Plains area of West Texas.

Because of the dominance of irrigation demands in the system, almost 90 percent of the total demand occurs at junction 17 between Caprock and Bull Lake Reservoirs. Smaller demands for local municipal, industrial, and agricultural use are imposed at other reservoirs and junction points. The average pumping lift across the eastern portion of the system (reservoir 12 to junction 14) is about 350 feet, while the lift westward to the high plains is roughly 2,800 feet. Over the entire system the average lift is roughly 5 feet per mile.

The volume and cost characteristics of the 12 reservoirs in the example problem are presented in Table 1. Most of the reservoirs have been sized as shown in the Texas Water Plan to provide a maximum volume which is consonant with topography and geology of the site and which is sufficient to maintain a firm yield of the tributary inflow over the historical period 1941-57. This was done to reduce the amount of computation time required to demonstrate the solution methodology described herein. A total capacity of 11.32 million acre-feet is provided, slightly more than the average annual local runoff.

Figure 6 shows the relative size and location of the water-supply reservoirs in relationship to average hydrologic parameters of the Sulphur River and Cypress Creek basins. In contrast to the 40 to 50 inches of average annual precipitation shown in Figure 6, the demand area has an average annual precipitation of approximately 18 inches.

Canals in the system shown in Figure 3 were sized in stages to allow for capacity expansion as demand builds up over the planning horizon. This characteristic of the problem is unique in the second year's study as contrasted to the earlier investigation in which only the ultimate capacity of canals was considered. Cost variations with canal size were described by second-order polynomials—one for each canal. A comparison of costs by reach is presented in Table 2 for a fixed level of capacity.

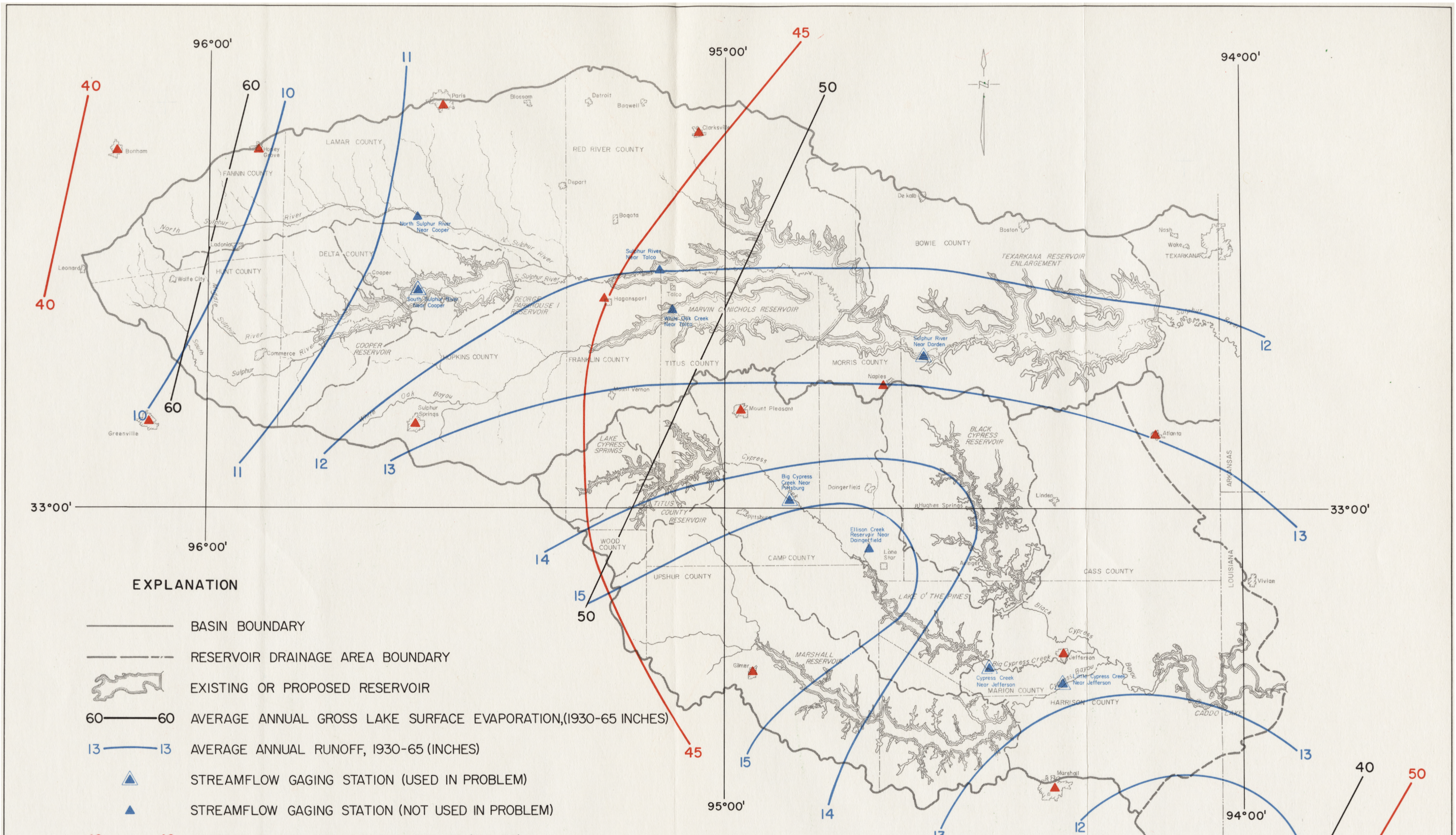



Figure 6

Hydrologic Information Used in the Example Problem

Sulphur River and Cypress Creek Basins

## Table 1.—Reservoir Characteristics and Costs

| NUMBER | NAME | STATUS | AVERAGE ANNUAL INFLOW (THOUSANDS OF ACRE-FEET) | VOLUME* (THOUSANDS OF ACRE-FEET) | UNIT COST (DOLLARS PER ACRE-FOOT) | TOTAL COST (MILLIONS OF DOLLARS) |
|---|---|---|---|---|---|---|
| 1 | Bull Lake Reservoir | Proposed | – | 3,000 | 25.0 | 75 |
| 2 | Caprock Reservoir | Proposed | – | 1,500 | 37.3 | 56 |
| 3 | George Parkhouse I Reservoir | Proposed | 106.9 | 635 | 42.5 | 27 |
| 4 | Marvin C. Nichols Reservoir | Proposed | 1,600.2 | 2,457 | 42.3 | 104 |
| 5 | Black Cypress Reservoir | Proposed | 213.1 | 824 | 41.3 | 34 |
| 6 | Lake Cypress Springs† | Existing | 42.9 | 73 | – | – |
| 7 | Titus County Reservoir | Proposed | 113.2 | 314 | 38.2 | 12 |
| 8 | Marshall Reservoir | Proposed | 399.4 | 782 | 35.8 | 28 |
| 9 | Cooper Reservoir | Authorized | 231.4 | 311 | 93.2 | 29 |
| 10 | Texarkana Reservoir | Existing (Proposed Enlargement) | 193.1 | 929 | 21.5 | 20 |
| 11 | Lake O' the Pines | Existing | 327.5 | 255 | – | – |
| 12 | Caddo Lake | Existing | 233.9 | 252 | – | – |
| | TOTAL | | 3,461.6 | 11,320 | 34.0 | 385 |

* The reservoir volume at the top of the conservation pool.
† Formerly known as Lake Franklin County; name changed by owner April 2, 1971.

In this example problem, import water was assumed to enter the Cypress Creek or Sabine River basins in East Texas when needed, from where it could be pumped overland through canals by several alternate routes to the High Plains of West Texas. Local demands were to be met at either reservoir or junction points along the route, with major deliveries scheduled for terminal reservoirs in the High Plains.

**Table 2.—Canal Construction Costs for a Capacity of 20,000 cfs***

| LINK NUMBER | CONSTRUCTION COST (MILLIONS OF DOLLARS) |
|---|---|
| 9 | 64.2 |
| 10 | 119.8 |
| 11 | 57.2 |
| 12 | 106.3 |
| 13 | 47.7 |
| 14 | 73.9 |
| 15 | 51.4 |
| 16 | 199.9 |
| 17 | 62.0 |
| 18 | 62.0 |
| 19 | 108.1 |
| 20 | 329.4 |
| 21 | 24.7 |
| 22 | 24.7 |
| 23 | 83.3 |
| 24 | 106.9 |
| 25 | 201.2 |
| 26 | 518.8 |
| 27 | 299.1 |
| 28 | 474.8 |
| 29 | 364.2 |
| 30 | 124.3 |
| 31* | .3 |
| 32* | .2 |
| 33* | .01 |
| 34 | 56.0 |

* Capacity is 10,000 cfs in links 31, 32, and 33.
NOTE: Links 1 through 8 are existing river reaches, hence have no construction costs.

## Mathematical Description

Mathematically, the equations to be solved are of the simplest algebraic type, merely statements of the law of conservation of mass. The basic equation, known in hydraulics as the Equation of Continuity and in hydrology as the Storage Equation, may be stated as:

$$\frac{\Delta \text{ Storage}}{\Delta t} = \Sigma \text{ Inflows} - \Sigma \text{ Outflows}$$

The several terms of the equation as it applies to a typical storage element (reservoir) in the Trans-Texas System are illustrated schematically in Figure 7, and are represented algebraically by the following statement:

$$\text{The Rate Of Change In Storage} = \left[\text{Upstream Releases} + \text{Pumped Inflows} + \text{Unregulated Inflows} + \text{Imports (If Any)}\right] - \left[\text{Controlled Releases} + \text{Pumped Outflows} + \text{Local Demands} + \text{Evaporation Losses}\right]$$

A complete set of continuity equations must be written for each discrete time step during which flows are considered as steady, i.e., the rate of change of storage is considered as constant. The set must include an equation for each reservoir and each canal or river junction, even though in the later case no net change in storage can occur. These equations must be solved for the system over the entire planning period to find canal and river flows, reservoir storage changes, and imported water required to meet demands, i.e., to satisfy continuity.

It is apparent from the dimensions of the problem that many more unknowns (canal flows, inputs, and reservoir levels) than equations can be identified. Thus, theoretically an infinite number of solutions is possible. However, since a minimum-expected-cost solution is desired, the problem can be constrained in such a way as to result in a unique solution, or that configuration, schedule, and operation plan which corresponds to the least expected cost. Conventional analytical solution techniques are not satisfactory for this task, and optimization procedures (Wilde and Beightler, 1967), such as those utilized in operations research, must be involved.

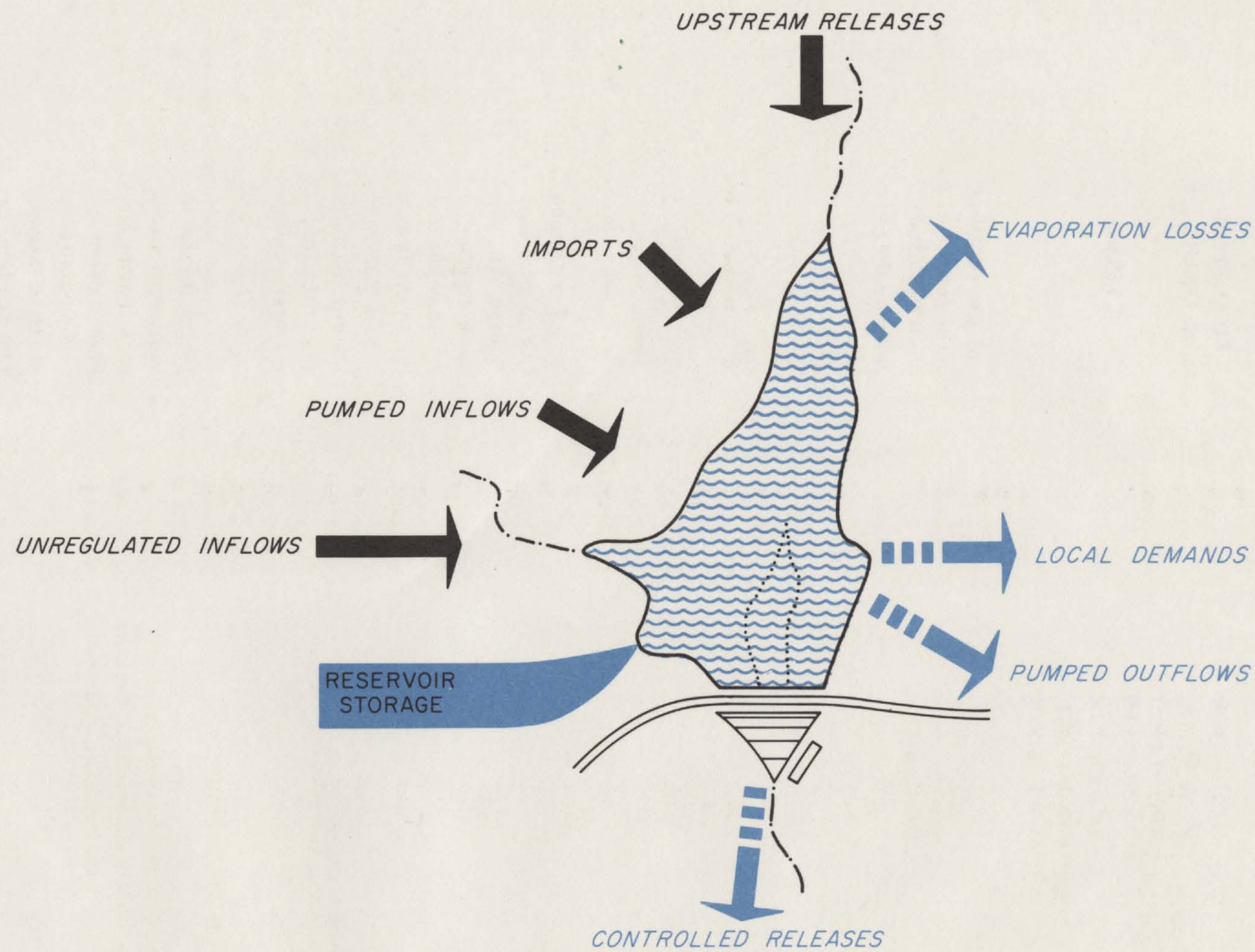



Figure 7
Terms in the Continuity Equation for a Typical Reservoir

# III. DATA BASE DEVELOPMENT

This chapter discusses the development of a data base for use by the simulation and optimization models in analyzing a complex water resource system. It covers that work referred to in Figure 2 as Step Two. A major portion of this step, Activity One, involves the development of the hydrologic and demand data base, whereas Activity Two involves the development of the physical, cost, and problem control data required to properly analyze a complex hydrologic system. In general, the flow of information and sequence of computations involved in this step are summarized in Figure 8.

A more detailed representation of the individual tasks involved in each activity in developing the required data base is displayed in Figure 9.

## Developing a Hydrologic and Demand Data Base (Step Two—Activity One)

As referred to in Chapter II, developing a hydrologic data base for the example problem involves six major types of data:

- refined runoff or reservoir inflow data,
- gross evaporation or climatic index data,
- rainfall data,
- net lake-surface evaporation data developed from rainfall and gross evaporation data,
- demands for irrigation water, and
- demands for municipal and industrial water.

Proper development of these data requires considerable effort. For example, it should involve a complete analysis of drought frequency, duration, and magnitude at various projected levels of watershed development (e.g., 1990, 2000, 2010, and 2020 conditions as shown in the demand-buildup curve in Figure 3). If irrigation purposes are to be served, as in the example problem, it involves the development of a good set of crop consumptive use data at the various projected levels of development. Similarly, for other demand types, good estimates of actual consumptive use and return-flow relationships are required at each of the projected levels of development to be analyzed.

### Enhancement of Historical Data Base (Task One)

To enhance the historical hydrologic and meteorologic data base, trend analysis techniques, fill-in programs, drought analysis procedures, flow-data refinement techniques, and flow-data projection techniques are used to preserve the appropriate cross and serial correlations within all of the historical, filled-in, and projected meteorologic and hydrologic data sets. The result is the development of a sound, comprehensive hydrologic and meteorologic data base for all subsequent planning studies.

Developing the broad data base for use in analyzing the example problem dictated that particular attention must be given to preserving the appropriate historical mean and projecting accurately future-condition means because, in general, varying the mean of the available runoff or rainfall data has considerably more impact on the cost response of the water transfer and storage facilities and resulting planning decisions than do changes or errors in the second and third moments (standard deviation and skew) of the basic data. In addition, specific attention should be given to preserving the longer-term drought characteristics in each of the data sets, because firm supply levels of water availability are, in general, highly correlated with long-term droughts. Preservation of this type of variability in the data can become of equal importance with preserving the appropriate historical mean.

Data sets typical of meteorology in the Southwest have considerable non log-normal variability, which complicates the use of the currently available data fill-in and stochastic generation programs referred to earlier. Therefore, in all of these programs, truncation criteria[13/] had to be specified in order for the programs to properly preserve the standard deviation and mean of the data set. Normally, those data truncated are above a certain prespecified recurrence interval (e.g., the 200-year interval). The truncation level chosen can have a drastic effect on the magnitude of the preserved mean and should receive considerable attention when using the currently available programs or any set of programs where inherent statistical assumptions do not exactly match the statistical characteristics of the data sets being processed.

Because of the time and budgetary constraints on this project, it was assumed that no significant trend existed in the supply data;[14/] thus no trend analysis or flow-refinement programs were developed for inclusion in this report. Both data fill-in and stochastic data generation programs were either developed or acquired for use in testing the methodology presented herein.

As was previously discussed, there is normally an insufficient length of historically continuous records at an insufficient number of locations within a water

13/ Herein, truncation criteria are defined as procedures which reduce high flow values (physically unreasonable values) in either the filled-in portion of a data set or in a stochastic data set.

14/ Later analysis of the supply data indicated that trends did exist; however, they were not large enough to invalidate the results of the data fill-in and stochastic generation programs. The trends were, however, large enough to affect the mean of the projected available supply at, for example, the 2020 conditions.

resource system to adequately describe, without analytic assistance, the system's total hydrologic characteristics. Therefore, to help improve the data base, correlation is generally used to estimate the missing data in noncontinuous records.

Regardless of the complexity of the procedure used, the objective of all data fill-in procedures is to obtain a complete record at the available gages, within a given system being analyzed, for as long a period of time as is thought to be statistically and otherwise practicable.

For the example problem, the objective of the data fill-in procedure was to obtain a complete record of precipitation, evaporation, and streamflow data for the 36-year period[15] from 1930 to 1965.

The 10 gages contained in Data Set 3, as shown in Table 3, are those that were required in this problem by the stochastic data generation programs and the simulation and optimization models, SIM-III and AL-II. Because these 10 gages were not without missing information, and the fill-in program used[16] could handle only a maximum of 10 gages, the logic of the fill-in process dictated that a three-stage fill-in procedure be used. In the example problem, the missing data for the two East Texas rainfall stations contained in Data Set 3, Hagensport and Jefferson, were filled in using eight other related rainfall stations (via Data Set 1). Next, in stage two, the missing data for one evaporation station, Hagensport, were filled in using the two filled-in rainfall station records developed in stage one plus seven other, related meteorologic gage records. Finally, in stage three, the two filled-in rainfall gage records and the one filled-in evaporation gage record were used to fill in the records from five runoff stations and thereby develop the final 10-gage data set (Data Set 3). This three-stage procedure is shown in Figure 10 and resulted in the development of a 36-year filled-in record at the 10-gage Data Set 3 shown in Table 3. A total of 25 gages were used in the fill-in process.

Further describing the fill-in process, Data Set 1 consists of filled in or complete historical data sequences for 10 rain gages. The observed or recorded data sequences are characterized by spotty data gaps that, when taken collectively, span less than 1 percent of the period of record. After missing data were filled in, the data from the Hagensport and Jefferson rain gages were selected for inclusion in Set 2 and in Set 3, the Generation Set. The observed record at Hagensport had 19 data gaps in the 36-year period, while observations at Jefferson had only two data gaps. The resulting filled-in data records consist of 432 events (36 years times 12 months per year).

[15] The 36-year historical period (1930-65) is equal in length to the projected demand-buildup period (1985-2020).

[16] The program used was the Monthly Streamflow Simulation Program developed at the Corps of Engineers' Hydrologic Engineering Center, Davis, California. This program was also used to generate stochastic data in the manner discussed later.

Data Set 2 consists of filled in or complete historical data records for the Hagensport and Jefferson rain gages and eight other meteorologic gages. Filled-in data records at the two rain gages were obtained from Set 1. Recorded data for the eight other meteorologic gages were complete except for evaporation data at Hagensport and Jefferson. Observations for these two gages were synthesized from average evaporation data compiled in *Monthly Reservoir Evaporation Rates for Texas, 1940 Through 1965* (Kane, 1967). These reservoir evaporation data are weighted averages of the published evaporation rates for nearby quadrangles and exist only for the period 1940-65. Thus, the period 1930-39 was filled in. An analysis or correlation of the filled-in lake evaporation sequences for Hagensport and Jefferson indicated that correlation is extremely high. Consequently, only evaporation data from Hagensport were carried forward for fill-in of the Generation Set (Set 3).

Data Set 3, the Generation Set, consists of filled in or complete historical records for precipitation gages at Hagensport, Jefferson, and Lubbock, evaporation gages at Hagensport and Lubbock, and five streamflow gages as listed in Table 3. Two of the streamflow gages are in the Sulphur River basin, and three are in the Cypress Creek basin. Figure 6 shows the location of the gages in East Texas relative to the reservoir sites.

All precipitation and evaporation data records were either complete or filled in in Stages One and Two. On the other hand, all five streamflow data records had gaps. Figure 11 illustrates the gaps in the data of these five gages. Once filled in, this data set, the Generation Set, was used as input to Task Two of the data base development activity, which is described next.

It is stressed that there is nothing unique about the 36-year data fill-in period used in the demonstration case except that it was equal to the length of the assumed demand-buildup period. As will become evident in Chapter VII, the simulation period analyzed must be at least as long as the time over which reservoirs and canals are being increased in size to meet increasing demands. To demonstrate this point, suppose that a 60-year demand-buildup period is being analyzed and that it is impossible to develop a statistically acceptable filled-in historical record for this entire period. The suggested methodology for coping with this problem is to develop, for stochastic generation purposes, a shortened, filled-in data set. Stochastic sequences 60 years or longer can then be generated and a matching period of 60 years of historical data developed (filled-in), by whatever means possible, to properly support the application methodology described herein. If a filled-in record of sufficient length cannot be developed adequately, reusing portions of the historical record should be considered. Prior to the staging analysis it will be demonstrated that the shortened record used as the basis for stochastic generation is adequate and, in fact, more desirable than the longer and less reliable, inadequately filled in historical data set.

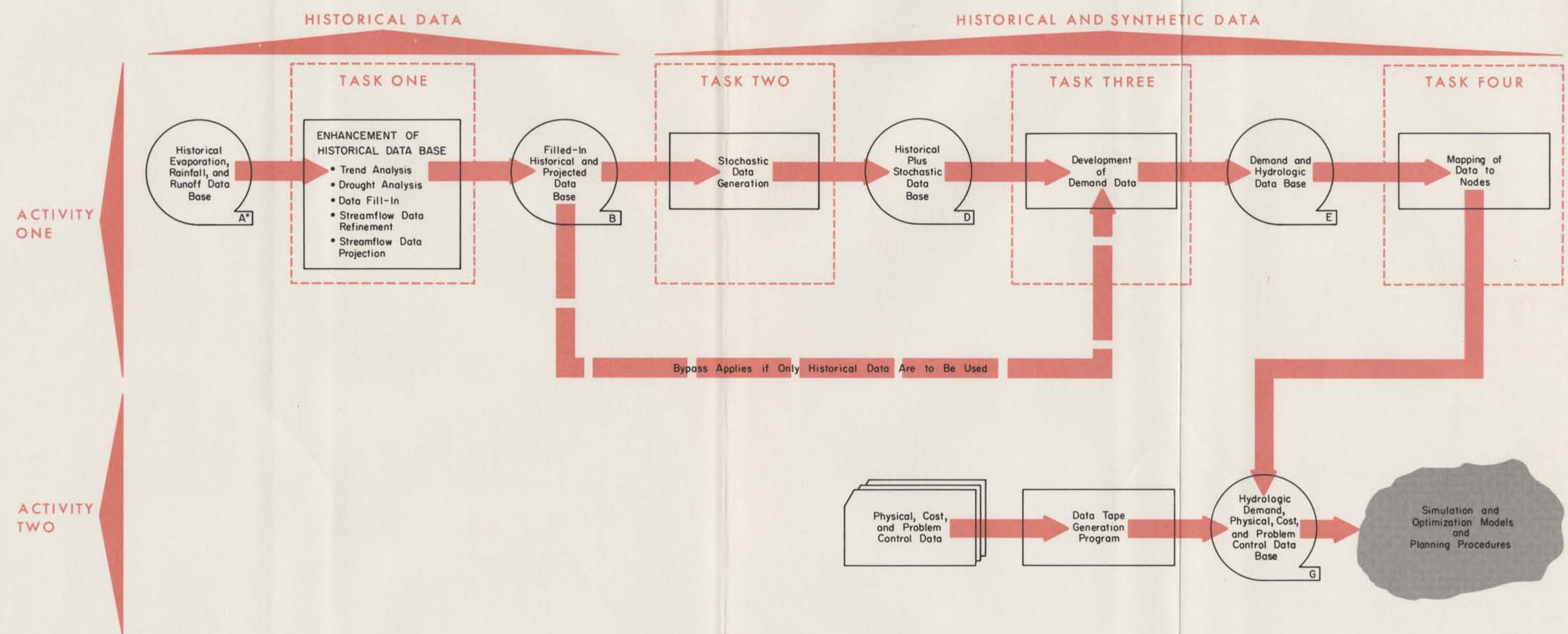



Figure 8
Summary of Data Base Development Activities (Step Two)

* Letters A, B, etc. are tape file labels as referred to in Volumes II and III of this report.

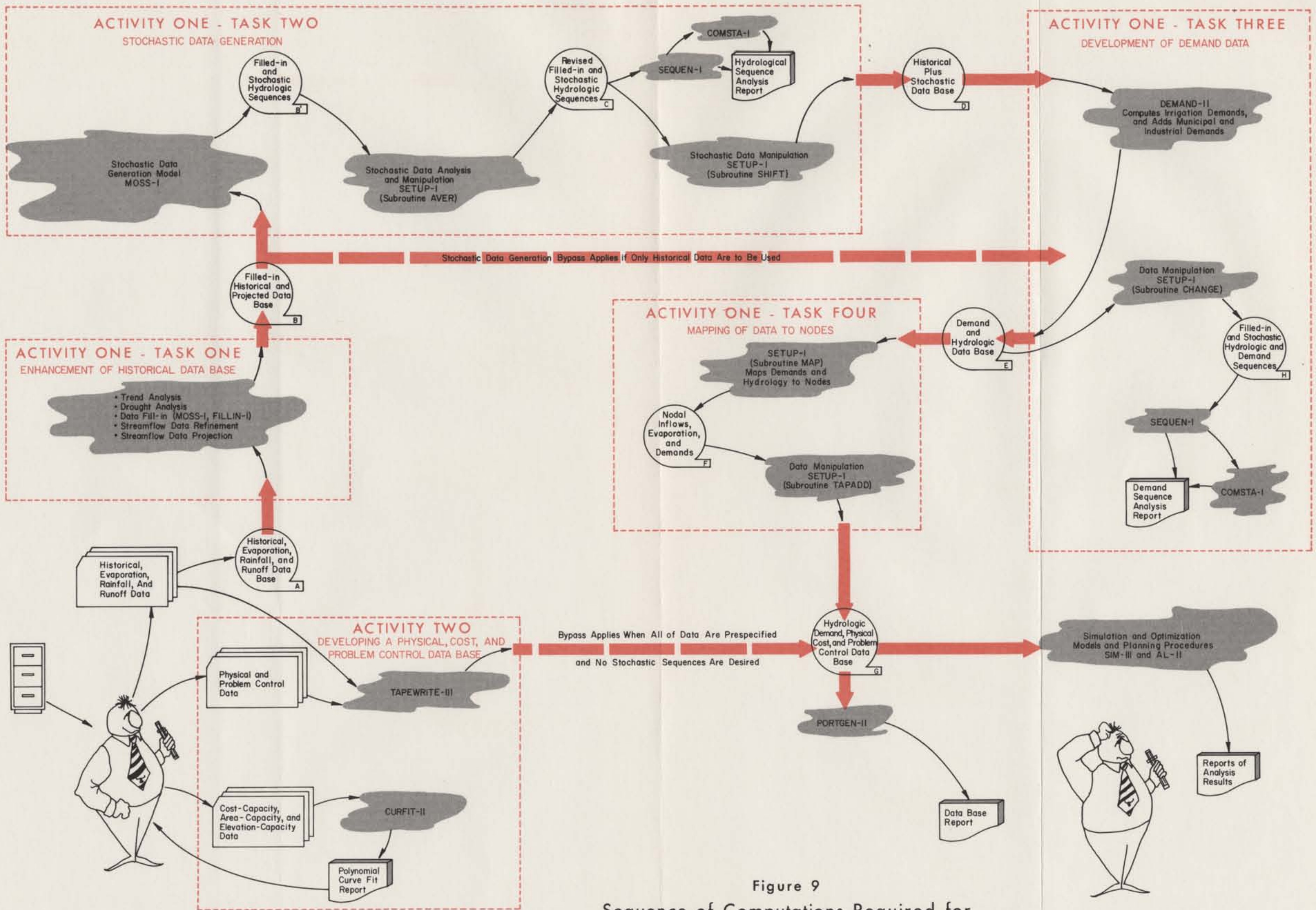



Figure 9

Sequence of Computations Required for Data Base Development (Step Two)

NOTE: CURFIT-II, TAPEWRITE-III, PORTGEN-II, and SETUP-I are described in Volume III of this report; SIM-III, FILLIN-I, AL-II, DEMAND-II, and SEQUEN-I are described in Volumes IIA through IIE; and COMSTA-I is described in Volume IIE. MOSS-I is described in the U.S. Army Corps of Engineers, Hydrologic Engineering Center publication HEC-4, *Monthly Streamflow Simulation, Generalized Computer Program 723-340*, February 1971.

**Table 3.—Gage Records and Data Sets Used in the Three-Stage Data Fill-In Process**

| GAGE LOCATION* | TYPE OF RECORD | DATA SET 1 | DATA SET 2 | DATA SET 3† |
|---|---|---|---|---|
| | **Precipitation Data** | | | |
| Hagensport | 1. Rain | X | X | X |
| Sulphur Springs | 2. Rain | X | | |
| Paris | 3. Rain | X | | |
| Clarksville | 4. Rain | X | | |
| Mount Pleasant | 5. Rain | X | | |
| Jefferson | 6. Rain | X | X | X |
| Atlanta | 7. Rain | X | | |
| Naples | 8. Rain | X | | |
| Gilmer | 9. Rain | X | | |
| Marshall | 10. Rain | X | | |
| Lubbock | 11. Rain | | | X |
| | **Other Meteorologic Data** | | | |
| Hagensport | 12. Evaporation (Lake) | | X | X |
| Dallas | 13. Percent Sunshine | | X | |
| Dallas | 14. Wind Movement | | X | |
| Dallas | 15. Humidity | | X | |
| Dallas | 16. Temperature | | X | |
| Clarksville | 17. Temperature | | X | |
| Mount Pleasant | 18. Temperature | | X | |
| Jefferson | 19. Evaporation (Lake) | | X | |
| Lubbock | 20. Evaporation (Pan) | | | X |
| | **Runoff Data** | | | |
| South Sulphur River near Cooper | 21. Runoff | | | X |
| Sulphur River near Darden | 22. Runoff | | | X |
| Big Cypress Creek near Pittsburgh | 23. Runoff | | | X |
| Cypress Creek near Jefferson | 24. Runoff | | | X |
| Little Cypress Creek near Jefferson | 25. Runoff | | | X |

* Gage locations are shown on Figure 6, except for Dallas and Lubbock gages.
† Set 3 is also referred to as the Generation Set.

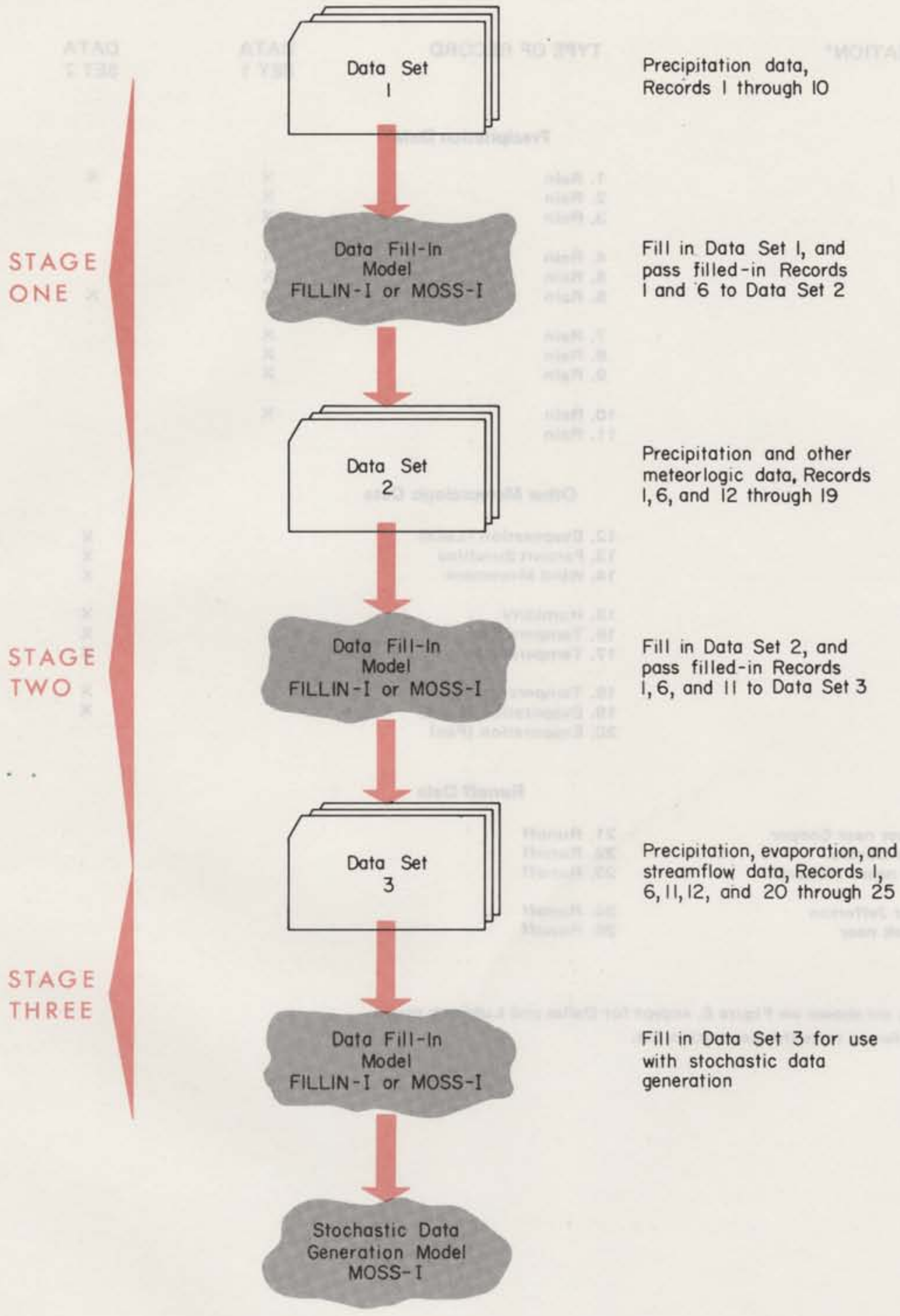



Figure 10

**Example of a Three-Stage Data Fill-In Process (Task One of Activity One, Step Two)**

Numbered records and data sets correspond to those in Table 3.

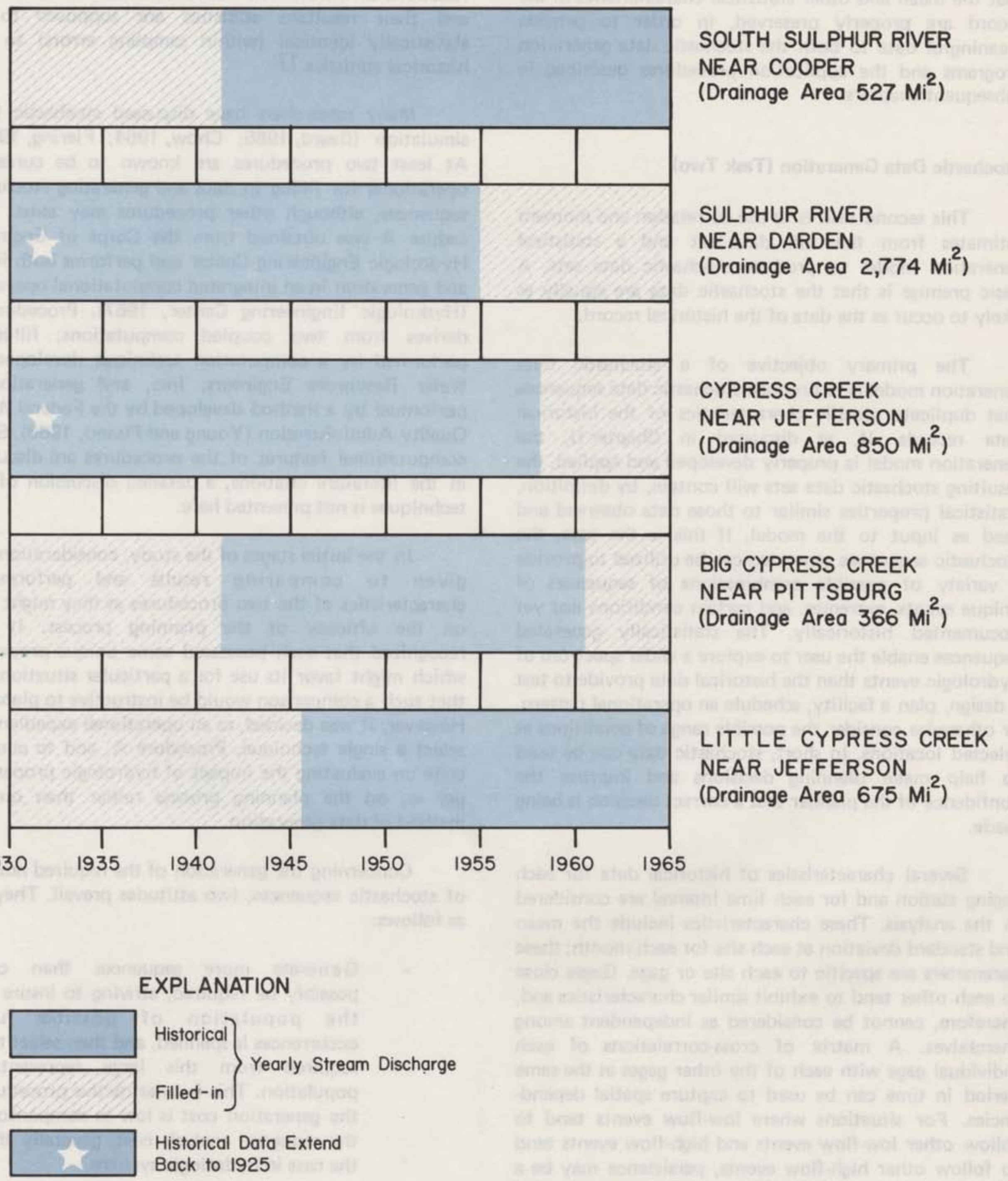



Figure 11

Length of Record for Stream-Gaging Stations Used in the Example Problem

In selecting a shortened record for generation purposes, as well as developing a filled-in historical sequence, close attention should be given to insuring that the mean and other statistical characteristics of the record are properly preserved, in order to provide meaningful data to both the stochastic data generation programs and the application procedures described in subsequent chapters.

**Stochastic Data Generation (Task Two)**

This second task is to use correlation and moment estimates from the full data set and a statistical generation model to produce stochastic data sets. A basic premise is that the stochastic data are equally as likely to occur as the data of the historical record.

The primary objective of a stochastic data generation model is to produce stochastic data sequences that duplicate specific characteristics of the historical data records. If, as discussed in Chapter II, the generation model is properly developed and applied, the resulting stochastic data sets will contain, by definition, statistical properties similar to those data observed and used as input to the model. If this is the case, the stochastic sequences generated can be utilized to provide a variety of possible combinations of sequences of unique events, extremes, and certain conditions not yet documented historically. The statistically generated sequences enable the user to explore a wider spectrum of hydrologic events than the historical data provide to test a design, plan a facility, schedule an operational pattern, or otherwise consider the possible range of conditions at selected locations. In short, stochastic data can be used to help make planning decisions and improve the confidence of the planner that a correct decision is being made.

Several characteristics of historical data for each gaging station and for each time interval are considered in the analysis. These characteristics include the mean and standard deviation at each site for each month; these parameters are specific to each site or gage. Gages close to each other tend to exhibit similar characteristics and, therefore, cannot be considered as independent among themselves. A matrix of cross-correlations of each individual gage with each of the other gages at the same period in time can be used to capture spatial dependencies. For situations where low-flow events tend to follow other low-flow events and high-flow events tend to follow other high-flow events, persistence may be a factor for consideration. Matrices of lag correlations of each gage in the current time period with itself and with each of the other gages in the preceding time periods are used to describe such temporal dependencies. A so-called "Markov" assumption is imposed in such cases (Fiering, 1967).

Stochastic hydrology fill-in and generation techniques extract statistics from the historical record; these statistics summarize central tendency, dispersion, and persistence of the data. The statistics are used to construct a model, the coefficients of the model being functions of the statistics. Stochastic data sets produced and their resultant statistics are supposed to be statistically identical (within sampling errors) to the historical statistics.17/

Many researchers have discussed stochastic flow simulation (Beard, 1965; Chow, 1964; Fiering, 1967). At least two procedures are known to be currently operational for filling in data and generating stochastic sequences, although other procedures may exist. Procedure *A* was obtained from the Corps of Engineers Hydrologic Engineering Center and performs both fill-in and generation in an integrated computational operation (Hydrologic Engineering Center, 1967). Procedure *B* derives from two coupled computations; fill-in is performed by a computation technique developed by Water Resources Engineers, Inc., and generation is performed by a method developed by the Federal Water Quality Administration (Young and Pisano, 1968). Since computational features of the procedures are discussed in the literature citations, a detailed discussion of the techniques is not presented here.

In the initial stages of the study, consideration was given to comparing results and performance characteristics of the two procedures as they might bear on the efficacy of the planning process. It was recognized that each possessed some unique properties which might favor its use for a particular situation and that such a comparison would be instructive to planners. However, it was decided, as an operational expedient, to select a single technique, Procedure *A*, and to concentrate on evaluating the impact of hydrologic properties, per se, on the planning process rather than on the method of data generation.

Concerning the generation of the required number of stochastic sequences, two attitudes prevail. They are as follows:

- Generate more sequences than could possibly be required, striving to insure that the population of possible future occurrences is spanned, and then select those required from this large representative population. This is an attractive procedure if the generation cost is low in comparison to the sequence analysis cost; generally this is the case in hydrologic systems.

- Generate a few sequences (five or six) and evaluate the system's performance against each sequence. Then, compute their expected performance, the standard error of

17/ "Identical" as used here implies that statistical tests of significance of differences applied to the two sets, historical and stochastic, would not be expected to reject the null hypothesis that the differences between the two sets equal zero.

their performance, and the total number of sequences required to reduce the standard error of the expected performance to an acceptable quantity. If normality is assumed, the standard error should decrease in proportion to the square root of the total number of years in the sequences. This procedure is attractive if the generation cost is relatively high with respect to analysis costs; this may be the case in complex hydraulic systems.

For the example problem, the first attitude was adopted and a total of 98 stochastic sequences were generated for each of the 10 sites previously mentioned. Each sequence was 36 years long—a period of time equal to the length of the assumed demand-buildup period. However, as described in Chapter VI, only a selected few of these 98 sequences are used in conjunction with the simulation and optimization models and the application procedures described in Chapters VII and VIII.

The use of data generation and fill-in techniques requires care in application. Four data attributes are identified which may cause the techniques to fail completely or partly in their objectives; the four attributes violate the assumptions which underlie both Procedures *A* and *B*. The four items of concern are:

- The numerical truncation which is required to constrain values to be greater than zero.—This truncation has the effect of shifting the mean of the generated data to values higher than the historical data. Data having coefficients of variation greater than 0.5 cause truncation problems.

- The form of the transformed probability distributions at each site.—If the marginal distribution is not Gaussian (or cannot be transformed to approximate a Gaussian distribution), problems in preserving moments can be expected. A practical guide is to check all marginal distributions prior to the implementation of generation techniques.

- The degree of cross-correlation between sites.—Should there be too strong a relationship (correlations approaching unity), the resultant collinearity causes numerical problems; the determinate of the correlation matrix approaches zero which makes matrix inversion subject to numerical error. In such cases, sites of lesser importance or sites which can be estimated from other nearby sites in the set can be removed. In other words, reduce the number of sites to reduce the collinearity.

- The effect of trends.—Quite often man's land-use activities over time have produced changes in runoff. For example, the amount of impervious surface in a catchment area may change, causing the average runoff to increase as a function of time. Historical data for use in planning should be checked for trends and detrended to provide statistically stationary data for data generation purposes.

For example, a troublesome data record is that from the stream gage Sulphur River near Darden. The stream essentially goes dry (probability of runoff being less than 10 cfs in October is estimated to be 41 percent) during certain months. This finding evolved after problems in stochastic generation for this gage were noted. Analysis of the observed data revealed a bimodal log transform distribution for the gage for the month of October as shown in Figure 12. The October distribution is based on 27 observed flows. Neither Procedure *A* nor Procedure *B* can cope with bimodal distributions. Bimodal distributions were not present in other gages in the Generation Set. The lesson, however, is clear: check the distributions and relate these to the assumptions of generation techniques.

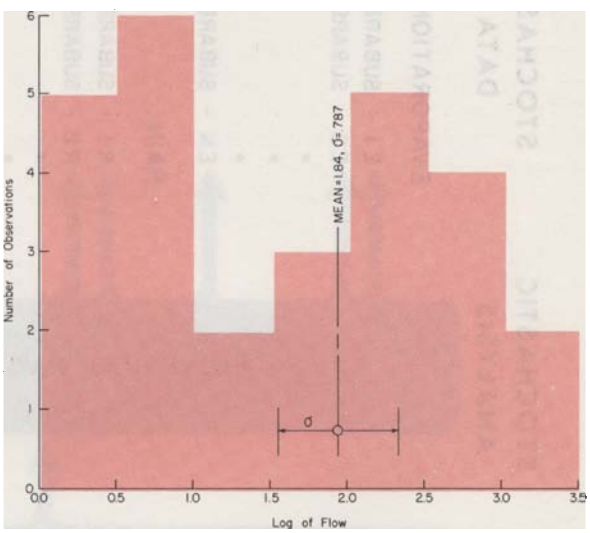



Figure 12.—Marginal Log Transform Distribution, Stream-Gaging Station Sulfur River near Darden, October (1930-65)

## Development of Demand Data (Task Three)

This section describes the general procedure whereby water demands, essentially irrigation requirements in West Texas, are determined from stochastic rainfalll and evaporation data. A so-called "irrigation macro-model", DEMAND-II, is described and discussed herein. Figure 13 shows how DEMAND-II is used.

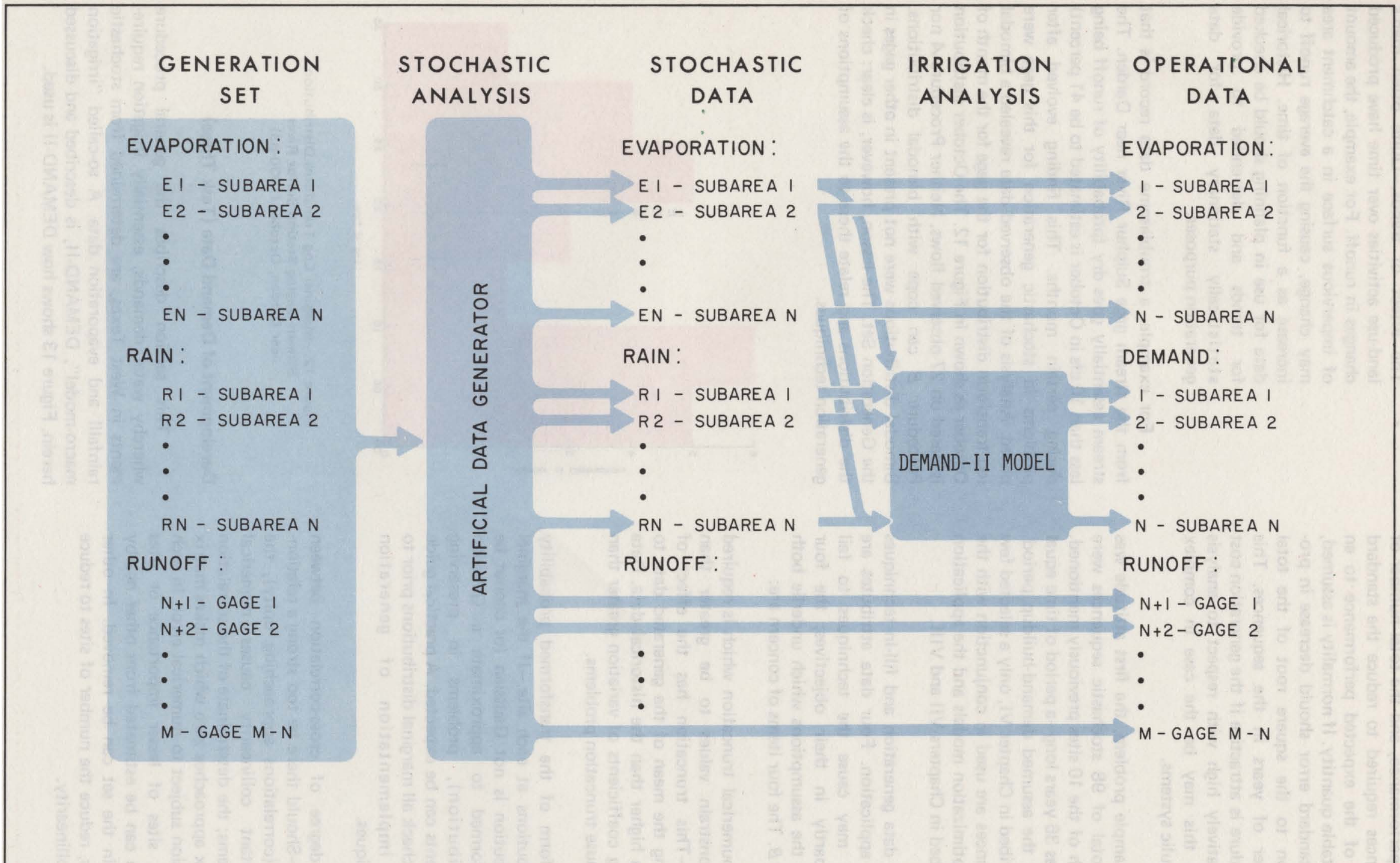



Figure 13
Data-Model Interactions, Area-Wide Stochastic Demand Model

NOTE—Each Subarea Has Soil And Crop Data For Input To The Macro-Irrigation Model; See Text For More Detail. Demands Are On A Per-Acre Basis.

In order to supply the simulation and optimization models with realistic estimates of demand for water, it was necessary to account for the stochastic variability of climatologic-meteorologic factors which influence crop water requirements. The most important stochastic variables are precipitation and evaporation. Both affect the short-term water needs of agriculture through their influence on soil moisture availability during the growing season. Evaporation rates, of course, are directly relatable to crop evapotranspiration and thus to crop water demand.

Water use by crops is also dependent on the type of crop, type of soil, drainage, degree of cultivation, and a variety of lesser factors. The variations in demand induced by such factors may be considered in the planning process by providing for deterministic spatial and temporal descriptions or, if the detail is not required, these variations can be treated in the aggregate. For the purposes of this study, variations were minimized by considering only three major agricultural demand areas, four soil types, and four major crops.

The unit acre demands for water were computed using the DEMAND-II model (see Figure 14), as documented in Volume IID of this report. Gross evaporation and rainfall data as filled in during Task One and as stochastically generated in Task Two comprise a portion of the input data required to compute the demands for irrigation water. In addition, data describing cropping pattern, acreage, soil condition, planting date, irrigation efficiency, and similar information are required.

Because other than irrigation demands do exist at nodes within most water resource systems, a capability was designed into DEMAND-II to accept via input cards prespecified demands which do not have a stochastic component. For example, these demands may be municipal and industrial requirements which can change each year in the simulation period but must have a seasonal distribution that is also supplied as input to DEMAND-II. These prespecified demands are added to the irrigation demands computed by DEMAND-II prior to generating, as output, a detailed tape file containing the monthly demands for each node in the system being simulated.

**Mapping of Data to Nodes (Task Four)**

The 98 stochastic data sets derived from the Generation Set for the example problem must be transformed into information useful to the optimization procedures; the Generation Set, itself, must also be transformed. This section considers the statistical adjustment of the Generation and stochastic data sets to convert gaged and stochastic runoff data into reservoir inflows, and the computation of lake surface evaporation from the evaporation and precipitation data.

Mean values of generated evaporation and rainfall data agreed with the filled-in historical means of the Generation Set. The generated runoff data had a mean that was 3 percent higher than the filled-in historical data. This discrepancy is assumed to be caused either by numerical truncation to avoid negative flows or by a lack

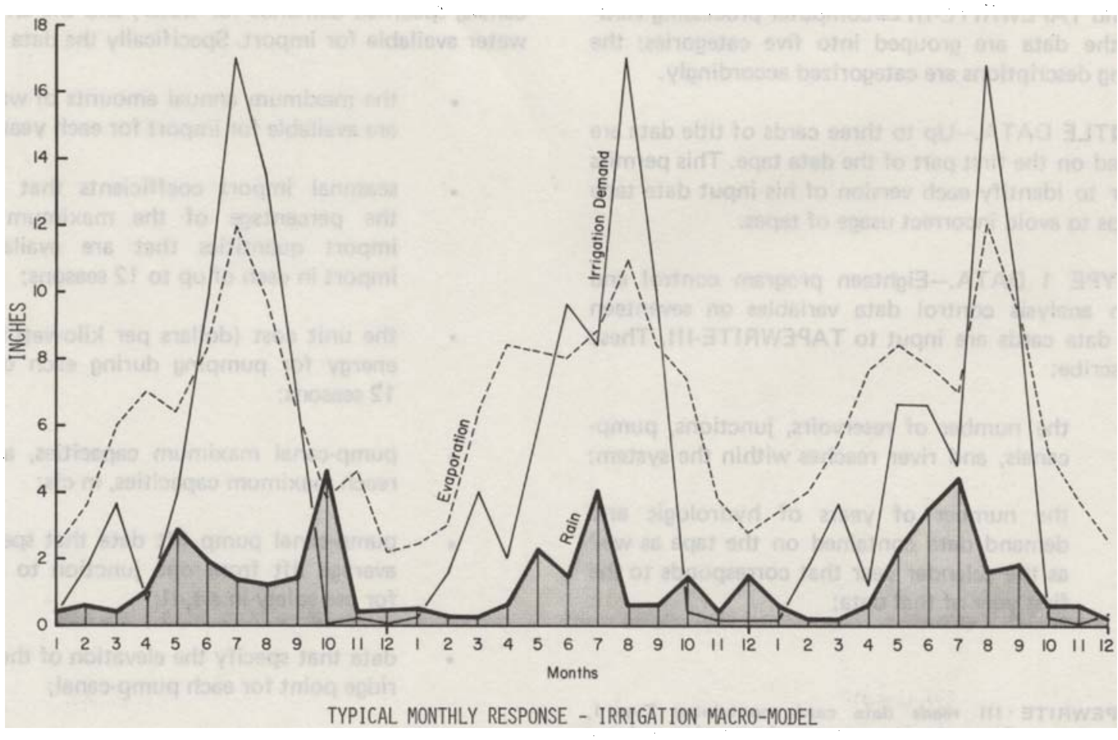



**Figure 14.—Typical Monthly Response of the Irrigation Macro-Model, DEMAND-II**

of fit of the observed distributions with the Gaussian form required by the generation technique. To offset this discrepancy, the generated runoff data were multiplied by factors (which varied from gage to gage) to force their means to conform to those of the historical data (the Generation Set). For example, the stochastic data for the Sulphur River near Darden were multiplied by a factor of 0.97. As a result of these transformations the mean flow for the generated data was made to correspond to the mean flow of the historical data.

Table 4 shows the coefficients used to transform gaged streamflow into reservoir inflow. The coefficients were derived from drainage-area ratios. For example, to compute the inflow into reservoir 11, Lake O' the Pines, multiply the Gage 23 flow by 1.0 and add the product of −.74 times the Gage 24 flow.

The rain and evaporation gages in the Generation Set are used to jointly compute net lake surface evaporation data for all reservoirs. The transformation considers the nearness of the appropriate Generation Set gages to the individual reservoirs and uses a procedure which is contained in another report (Kane, 1967) in implementing the assignment of lake surface evaporation values.

## Developing a Physical, Cost, and Problem Control Data Base (Step Two—Activity Two)

The purpose of this section is to itemize and describe, in detail, the non-hydrologic data used as input to the simulation and optimization models. From a user's and TAPEWRITE-III [18] computer processing viewpoint, the data are grouped into five categories; the following descriptions are categorized accordingly.

TITLE DATA.—Up to three cards of title data are contained on the first part of the data tape. This permits the user to identify each version of his input data tape and helps to avoid incorrect usage of tapes.

TYPE 1 DATA.—Eighteen program control and problem analysis control data variables on seventeen Type 1 data cards are input to TAPEWRITE-III. These data describe:

- the number of reservoirs, junctions, pump-canals, and river reaches within the system;
- the number of years of hydrologic and demand data contained on the tape as well as the calendar year that corresponds to the first year of that data;
- the number of the node at which water can be imported;
- the number of seasons per year (the analysis time increment—normally months);
- the unit cost of water at the import point;
- the annual operation and maintenance cost for reservoirs, and the operation, maintenance, and replacement costs of pump-canal facilities, both expressed as percentage of their total capital cost; and
- the repayment period in years, the interest rate, and the reservoir and canal finance lag time.

TYPE 2 DATA.—System configuration data describing the connection of reservoirs and junctions with pump-canals and river reaches constitute this type of data. This includes:

- the junction and/or reservoirs at the ends of each pump-canal;
- the junctions and/or reservoirs at the ends of each river reach; and
- indication of whether the import node is a reservoir or a non-storage junction.

TYPE 3 DATA.—These types of data consist generally of cost and capacity data for reservoirs and canals, specified demands for water, and the amount of water available for import. Specifically the data include:

- the maximum annual amounts of water that are available for import for each year;
- seasonal import coefficients that describe the percentage of the maximum annual import quantities that are available for import in each of up to 12 seasons;
- the unit cost (dollars per kilowatt-hour) of energy for pumping during each of up to 12 seasons;
- pump-canal maximum capacities, and river reach maximum capacities, in cfs;
- pump-canal pump lift data that specify the average lift from one junction to another, for use solely in AL-II;
- data that specify the elevation of the highest ridge point for each pump-canal;

[18] TAPEWRITE - III reads data cards containing Type 1, Type 2, and Type 3 data for the purpose of preparing an input tape for SIM-III and AL-II. Its use is described in Volume III of this report.

**Table 4.—Mapping Matrix: Coefficients Used to Transform Gaged Streamflow Into Reservoir Inflow**

| STREAM GAGE | GEORGE PARKHOUSE I RESERVOIR 3 | MARVIN C. NICHOLS RESERVOIR 4 | BLACK CYPRESS RESERVOIR 5 | LAKE CYPRESS SPRINGS 6 | TITUS COUNTY RESERVOIR 7 | MARSHALL RESERVOIR 8 | COOPER RESERVOIR 9 | TEXARKANA RESERVOIR 10 | LAKE O' THE PINES 11 | CADDO LAKE 12 |
|---|---|---|---|---|---|---|---|---|---|---|
| 21. South Sulphur River near Cooper | .32 | 1.223 | | | | | .903 | −1.223 | | |
| 22. Sulphur River near Darden | | .957 | | | | | | .284 | | |
| 23. Cypress Creek near Jefferson | | | | | | | | | 1.0 | −1.0 |
| 24. Big Cypress Creek near Pittsburgh | | | | .2 | .54 | | | | − .74 | .8 |
| 25. Little Cypress Creek near Jefferson | | | .52 | | | .97 | | | | .6 |
| Yearly average reservoir inflow (thousands of acre-feet) | 106 | 1,600 | 213 | 42 | 113 | 399 | 231 | 195 | 327 | 233 |

NOTE: The coefficients are determined using drainage-area ratios. It may be necessary to modify the coefficients to account for spatial variations in yield (inches per year); this possibility should be checked in other applications. The corrections for this study are found to be less than 1 percent and are neglected; this is because the reservoirs are near the gages.

- second-order polynomial coefficients that describe the capital cost-capacity relationships for each pump-canal (pump and ditch costs);

- third-order polynomial coefficients that describe the area-capacity relationships and elevation-capacity relationships for each reservoir;

- maximum and minimum storage capacities and initial storage contents (percentage full) of all reservoirs; and

- estimates of the average annual surface area of each reservoir.

TYPE 4 DATA.—These data consist of the seasonal data listed below, which are developed in this activity only when all of the demands, evaporation rates, and reservoir inflow data are prespecified by a procedure other than those described in Activity One. This situation is referred to in the lower part of Figure 9 as the hydrologic and demand data base bypass.

- unregulated inflow into each reservoir for each year contained on the tape and for up to 12 seasons per year,

- demand data for each node and for up to 12 seasons per year, and

- net evaporation data for each reservoir and season of the year.

These data are organized for input to TAPEWRITE-III on a series of problem-oriented input data forms. Figure 15 shows a typical example of one of these forms.

Keypunch 026 029

User________________ Form (A) of (I)

TEXAS WATER DEVELOPMENT BOARD
DATA MANAGEMENT SUBSYSTEM
TAPEWRITE III

Page____of____

TITLE DATA

| |
|---|
| TITLE1 |
| TITLE2 |
| TITLE3 |
| ENDTITLE |

*Type 1 Data*

| | *CROSS OUT THE UNITS DESCRIPTOR NOT APPLICABLE* | | *INDICATE MAGNITUDE OF FLOW UNITS (1000's Ac-Ft.)* | *UNITS AND OUTPUT OPTION CONTROL DATA* |
|---|---|---|---|---|
| FLØW UNITS | ACFT | CFS | . | |
| EVAP UNITS | INCHES | FEET | | |
| RAIN UNITS | INCHES | FEET | | |

*Type 1 Data*

| | |
|---|---|
| LIST CARD INPUT | *CROSS OUT THOSE OPTIONS NOT DESIRED* |
| EDIT DATA | |
| LØAD DATA ØN TAPE | |
| WRIT REPØRT ØNE | |

PROBLEM ANALYSIS CONTROL DATA

*Type 1 Data*

| | | | |
|---|---|---|---|
| CALENDAR YR START TIME = | . | NUMBER ØF RESERVØIRS = | . |
| NUM YRS PAST CAL YR STRT = | . | NUMBER ØF JUNCTIØNS = | . |
| LENGTH ØF ANALYSIS YEARS = | . | NØDES = RESEVRS + JUNCTS = | . |
| NUMBER ØF SEASØNS / YEAR = | . | NUMBER ØF RIVER REACHES = | . |
| REPAYMENT PERIØD IN YRS = | . | NUMBER ØF PUMP CANALS = | . |
| PERCENT INTEREST RATE = | . | LINKS = CANALS + REACHES = | . |
| RESEVR FINANCE LAG TIME = | . | NØDE WHERE IMPØRT ØCCURS = | . |
| CANAL FINANCE LAG TIME = | . | $/AC-FT TØ IMPØRT WATER = | . |
| Ø&M ANNUAL RATE % CANAL = | . | Ø&M ANNUAL RATE % RESEVR = | . |
| ENDATA1 | | | |

Figure 15

**Example of Input Data Preparation Form for TAPEWRITE-III**

# IV. PLAN DEVELOPMENT

This chapter discusses the problems facing the planner when he initially analyzes a multibasin water resource system knowing that he must eventually decide

- which of an over-specified set of reservoirs and canals should be built,
- how large and when each of these facilities should be built, and
- how each should be operated to make efficient use of the available water.

In that context, a fairly standard hydrologic analytic approach, "firm-yield analysis," is coupled with an advanced firm-yield analysis program, SIMYLD-I,[19] to obtain preliminary sizes for the set of storage and transfer facilities required to meet expected demands for water at various future time periods. Also, if little is known about the magnitude of the future demands, this procedure can be used to develop a conservative estimate of the available water in a water resource system.

## Perspective

Because of time and monetary limitations, this project only discusses the surface water and conservation storage aspects of the water resource planning system. However, to put the limited, yet complex, analysis procedure in perspective, the following three subsections briefly discuss the other aspects of the problem prior to presenting the procedural details of plan development.

### The Resource

In a complex system such as the proposed Texas Water System, comprehensive treatment of the problem should consider the following major sources of supply:

- surface water resources, including dam and reservoir sites,
- underground resources, which include not only ground water but also underground storage capacity and transmission capability,
- atmospheric waters (the amount of surface water available in the future may be increased by weather modification),
- return flows or waste waters, and
- saline or brackish waters.

Surface water, ground water, atmospheric waters, and return flows are highly interrelated. The amounts of surface water and ground water available vary widely over time and space; this is due not only to natural causes but also to such other factors as the level of resource development and land treatment and use. The planner must relate both the quantity and the quality of these resources to the requirements of the water users to successfully meet his stated objectives.

### Water Uses

The uses of water resources may be categorized as (a) direct use for human needs, (b) uses necessary to develop and make other resources available, and (c) other benefits achieved through water development. These uses are not as clearly distinguishable as they appear from this relatively simple categorization. In the evaluation of uses, full consideration must be given to ecological and environmental inputs and impacts, as well as the effects of waste flows and effluents returned to the stream.

Direct uses include:

- domestic use,
- municipal use (exclusive of industrial use from municipal systems),
- use for final disposal of treated municipal wastes and urban runoff,
- recreation, and
- hydropower generation.

The uses of water for the development of other resources include:

- industrial uses (including cooling),
- irrigation,
- maintenance of fish and wildlife resources,
- mining and oil production,
- navigation, and
- use for final disposal of treated industrial wastes, including heat, and agricultural drainage.

[19] SIMYLD-I is a program developed during this research project for computing the firm yield of an entire basin or multibasin network of storage reservoirs. The details of this program's capabilities are discussed in Volume IIA of this report.

A major portion of the total use of water resources is involved in making available or utilizing other resources, particularly land resources, to meet human needs.

A number of other direct benefits stem from water resource development; these include flood protection and water-quality control. Water-quality requirements for these many uses vary widely, and these requirements must be fully taken into account in the planning process. As water uses increase, both in variety and number, it is necessary to plan for and manage conjunctively water quantity and quality.

### The Physical Facilities

To support the above uses with the available resource and within the recognized constraints, systems of physical facilities have become much more complex; they have progressed from single- or dual-purpose projects involving simple works to multi-facility projects serving several purposes over wide geographical areas. The physical facilities involved in these complex systems may be extensive and include such diverse facilities as:

dams and reservoirs,

canals and other types of conveyance works,

pumping plants, including energy sources,

hydropower plants,

wells,

artificial recharge works,

distribution systems,

water treatment plants,

waste water treatment plants,

waste water reclamation plants,

channel improvements,

flood retardation facilities,

land treatment measures for runoff and erosion control, and

weather modification facilities.

Many regional systems of the future will probably encompass most, if not all, of these facilities as well as some that are not listed. These must be planned, designed, and operated conjunctively to develop the resources, to meet the demands, and to accomplish the objectives.

Complex water and related land resource systems may and probably will encompass the facilities financed, designed, and operated by both private and governmental entities. Institutional arrangements among the varied interests for financing, designing, operating, and managing such systems are necessary.

These extensive regional systems will normally be very complex; they will draw water from diverse sources—surface, underground, desalted, and reclaimed water—and they will be operated as integrated systems. At the same time, demands will increase while the constraints imposed, legal and otherwise, will become more limiting. Also, the planner will have to give more attention to ecological and environmental impacts, both beneficial and detrimental. As a consequence, the alternatives he will have to consider to plan, design, and operate these complex systems will greatly increase. The facilities proposed in the Texas Water Plan are an example of such a complex system.

## The Analysis Approach

The material herein expands that provided in Chapter II concerned with Step Three, Plan Development Based on Historical Data. In addition to discussing demand versus supply relationships, it discusses suggested procedures for the preliminary sizing of reservoirs and transfer facilities within a single basin or multibasin water resource system. This is a systems approach to the conventional reservoir yield analysis used by water planners. In this context the approach uses, as input, the results of Step Two described in Chapters II and III; Step Two developed and organized all of the cost-capacity, elevation-capacity, area-capacity, projected water usage, and historical hydrologic data (no stochastic data sequences) associated with an over-specified set of attractive reservoir sites within each of the basins comprising the multibasin water resource system.

### Analysis of Single-Basin Systems (Step Three—Activity One)

The first portion of this step concerns itself with analyzing each basin separately to determine

- how best to physically control the available runoff in each of the individual basins,
- the various amounts of water that each basin can be expected to yield,
- a relationship of cost versus firm yield for each basin,

- the minimum unit cost firm surplus[20] or firm deficit[21] of each basin, and

- the most likely points at which exports from and imports into each basin might best be accommodated.

This is accomplished in an orderly manner which first strives to find the firm yield of each individual basin (based only on the historical data sequence) while controlling the basin's runoff with no capital improvements, except for additional storage facilities. In other words, no extensive pump-back facilities are assumed. This is the conventional analysis procedure and provides the planner with conservatively low estimates of the amount of surplus water available for export from each "water rich" basin, and conversely, high estimates of the amount of import water required to meet demands for water in each "water poor" basin.

It should be noted that if, over the planning horizon, a significant increase or decrease in the supply is expected, due to changing watershed conditions (i.e., urbanization or other land-use changes), it becomes necessary to determine projected firm yields at various points in time within the planning horizon in addition to current firm yields. For the example presented herein, a trend-free historical supply is assumed.

Because numerous arrangements of reservoir locations, sizes, the operation criteria can be postulated which will give identical estimates of firm yield, it is necessary to develop a criterion for choosing among alternative arrangements.

The methodology presented herein uses a criterion of minimum unit storage cost (dollars per acre-foot of storage) for selecting the best configuration, where costs included are

- capital investment costs amortized over the economic life of the project (e.g., 100 years),

- the annual maintenance cost,

- the annual cost of the loss of any water from the system, and

- any other annual costs that are changing with the storage capability of each basin.

Figure 16 is presented in dimensionless terms to demonstrate the general functional relationship that results when using the minimum-unit-cost criterion for selecting a firm-yield plan. The potential range of physical facility costs required to support a given firm yield are also portrayed in Figure 16. This range defines an envelope curve, and by definition the best plan is located at the minimum-unit-cost point on the curve of lowest-cost alternatives as shown in the upper portion of this figure.

By performing this type of analysis on each basin within a multibasin system, a conservative estimate is developed of the system's capability to yield. Also, conservative estimates result of the location and magnitude of firm surpluses and/or firm deficits. By analyzing the firm deficit and surplus information, the planner can select desirable points at which export from and import into each basin might best be accommodated. This information is used in the latter portion of this Step Three to further refine each of the basin implementation plans.

If the sum of the firm surpluses minus the firm deficits for all basins being analyzed indicates that a rather large system-wide deficit can be expected, this procedure provides a quantitative means to

- preliminarily size the required external import source,

- reduce the extent of the contemplated service areas, or

- explore the possibilities of expanding the supply basins within the multibasin complex to include basins with firm surpluses.

However, if the sum of the surpluses minus the deficits is within 10 to 20 percent of meeting the expected demands, experience on the example problem as shown in Figure 17 indicates that an optimized system plan might eliminate what appears to be a total system deficit. The 10 to 20 percent estimate is problem-specific and dependent on the magnitude of the variability contained within both the supply and the demand. In general, the greater the variability, the greater is the apparent firm deficit that can be eliminated by means of systematized analysis and operation criteria.

**It should be noted that in many existing planning studies, as was the case in developing the Texas Water Plan, quantitative hydrologic analysis ends with the completion of the first portion of this Step Three—computation and use of individual river basin firm yields, firm surpluses, and firm deficit information on a reservoir-by-reservoir basis.** This means that, from a planning perspective, using only firm-yield analysis in conjunction with conservative estimates of demands for water is considered sufficient by many planners to develop estimates of the sizes of the storage and transfer facilities that are required to safely meet the prespecified

[20] Firm surplus is defined as the quantity of water that can be exported out of a basin after meeting all in-basin demands.

[21] Firm deficit is defined as the quantity of import water required to meet in-basin demands if only the firm yield of the basin can be used to meet its in-basin demands.

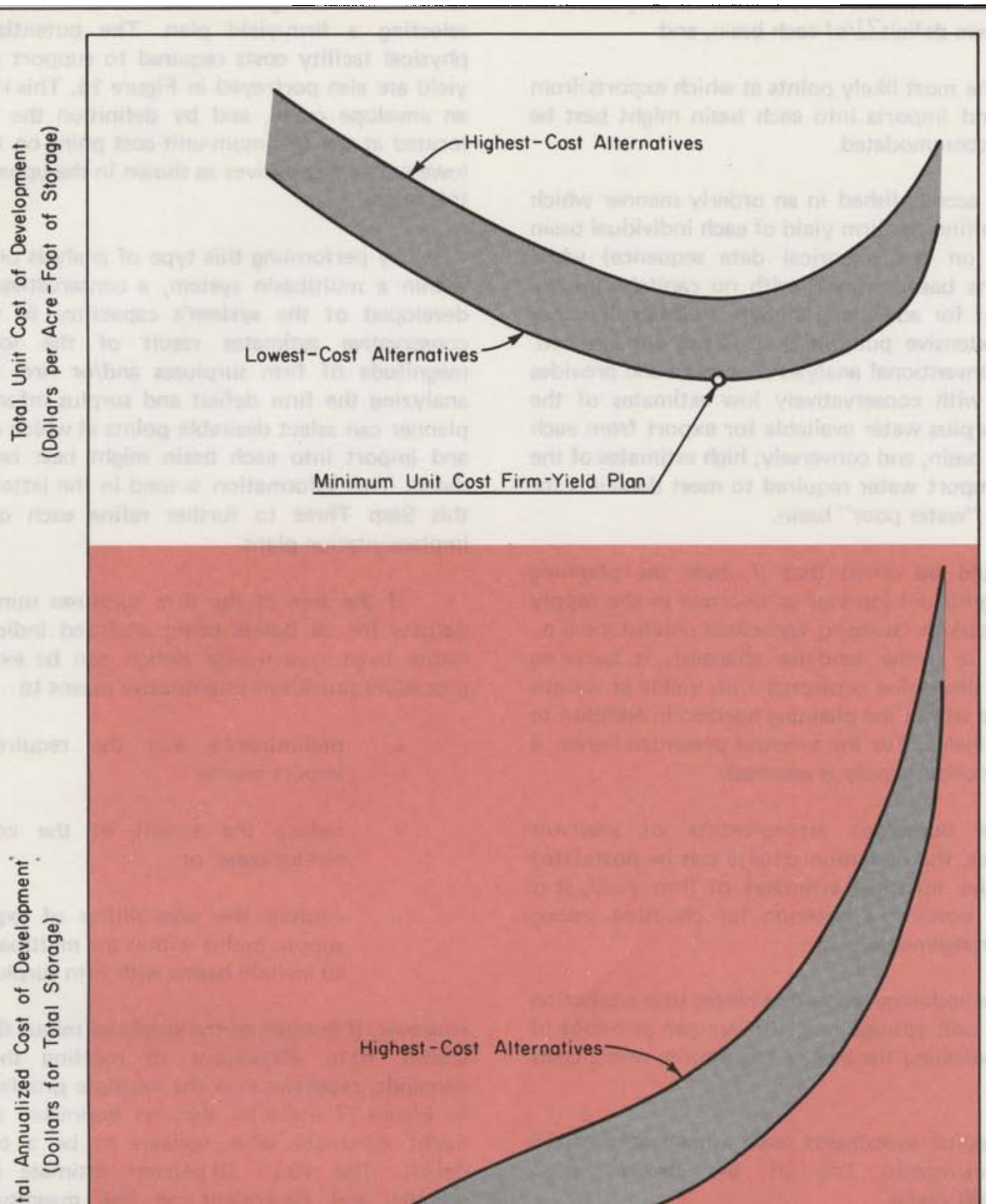



**Figure 16**

**Selecting a Firm-Yield Basin Plan**

demands for water. In fact, currently most water rights are based upon this somewhat limited analysis.

In many cases this analysis is sufficient, especially where

- the effects of stochastic variability in the supply and demand are significantly damped by the use of large storage reservoirs,

- large amounts of import water are available from a source external to the system being analyzed,

- the unit penalty costs for incurred shortages are large in comparison to the unit costs of delivery (i.e., the cost of meeting demands), as in most municipal and industrial demands for water, or

- the system being analyzed is simple, thus, cannot be designed and operated in a manner to realize the full advantage of good versus bad operation criteria.

Conversely, in many other cases the analysis described to this point is insufficient to provide sufficient planning information on the hydrologic behavior of a complex water resource system. This is especially true where

- the stochastic variability is large,

- unit penalty costs for incurring shortages are not significantly greater than the unit costs of delivery, as in many irrigation projects, or

- no import water external to the system or no extremely large storage reservoirs are available to dampen both the extremely intense short-duration droughts and moderately intense but longer-duration droughts.

It is to the latter type of problem that the remaining portion of this report directs its attention and where the greatest benefits from using the procedures described herein can be realized.

**Analysis of the Multibasin System (Step Three—Activity Two)**

The second portion of Step Three looks at the entire multibasin complex acting as an interconnected network to evaluate the worth of various attractive interconnective configurations and the effects of systematized operation criteria, and thereby determines

- how the reservoir sizes chosen during Activity One of this step need to be modified to increase the amount of water available,

- relationships of capital cost versus firm yield for the entire system,

- the increase in the system's total firm surplus or decrease in total firm deficit attributable to operating the multibasin complex as a system,

- how best to control the available runoff in the multibasin system of reservoirs and canals, and

- the amount of water that the entire system can be expected to (firmly) yield over and above that amount each basin yields.

This is accomplished by first developing cost-capacity and other physical relationships for an over-specified set of potentially attractive interbasin transfer canals and intrabasin pump-back facilities. With this information at hand, to supplement the information developed in Step Two, the SIMYLD-I program is used to find an updated set of reservoir sizes and desirable interbasin canals that have the greatest effect upon increasing the firm yield of the multibasin system without substantially increasing the capital expenditures.

Even though considerable attention is given to identifying the unit-cost impact of added interbasin canals, the major emphasis is placed upon determining the extent by which the reservoir sizes determined in Activity One of this step should be modified to create a better overall multibasin system of storage facilities. However, the objective still concerns itself with identifying the minimum unit cost firm-yield plan in a manner similar to that portrayed in Figure 16.

By approaching the problem in this manner, the hydrologic and cost effects of each basin on every other basin can be quantified. This information is very useful during the allocation process to help determine how the various users should share the available waters and how each should pay for the water used. Also, during this step, the problem arises of using firm-yield information, based upon a sub-optimal plan, to estimate the magnitude of the demands that can be met with an optimal plan at an optimal level of shortages.

The closer that the user is able to estimate the level of both the firm yield (the condition of no shortage) and the actual yield (the condition of optimal shortage) of a multibasin system, the less analysis and fewer simulation iterations are required to find the level of demands that can be optimally met. However, to be able to closely estimate the optimal based upon firm-yield information, considerable experience is required in applying the models to various types of problems. In the example problem presented herein, Figure 17 indicates

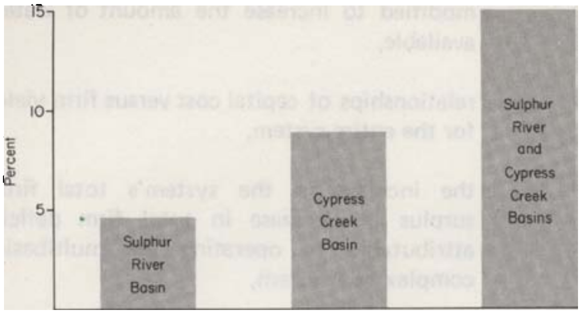



Figure 17.—Percentage Increase in Basin Firm Yield Due to Systematized Operation

that systemized operation causes about a 15-percent increase in the firm yield of the individual reservoirs when both the Cypress Creek and Sulphur River basins are operated as a composite system. Each basin operating by itself in a systematized manner resulted in a total increase of about 13 percent. The sum of the firm yields of the reservoirs in each basin, acting independently without the pump-back facilities, was approximately 2.2 million acre-feet per year (63 percent of the average annual runoff); whereas, using the results of Steps Four and Five which "optimize" the size and operation of each of the required canals, river reaches, and reservoirs, the actual yield (not firm yield) was increased to approximately 3.1 million acre-feet per year or 89 percent of the average runoff of 3.5 million acre-feet per year.

It should be recognized that both the 63 percent and the 89 percent values are problem-specific. Therefore, in reality, considerable quantitative knowledge is required about both the prototype's performance capability and the variability of the supply and the demand, prior to being able to predict the actual amount of water that a basin can yield.

The magnitude of the optimal usable yield of a multibasin supply system is dependent upon the magnitude of the unit penalty costs associated with incurring shortages and the degree to which the basins can be optimally interconnected with transfer canals. In essence, the higher the penalty costs,

- the larger the capacity and more expensive will be the optimal interconnective pump-canals,
- the smaller will be the optimal shortage amount, and
- the greater will be the actual yield or conversely the smaller will be the external spills and evaporation losses.

Therefore, as a means to overcome the requirement of estimating accurately the demands that can be met, the procedure herein purposefully oversizes the demands that can be expected to be met to insure that computed shortages will occur, and that the penalty cost data will interact with the total cost of allocating and delivering water to the various demand points within the system. Additionally, if the penalty cost information is structured correctly, the use of penalty function data in a minimum-cost simulation or optimization program such as SIM-III results in a procedure which tends to maximize net benefits upon finding a minimum-cost solution.

Upon systematically analyzing the numerous storage facility size alternatives, in the context of striving to identify a minimum unit cost firm yield storage plan and interbasin transfer plan for the entire multibasin system, this information is passed on to the next step of this planning procedure (Step Four as described in Chapter V). The information passed to Step Four serves as a preliminary basis upon which to further analyze the set of firm-yield plans identified as attractive during this step.

Up to this point the concept of firm-yield analysis has played the primary role in the analysis procedure. In Step Four, discussed in Chapter V, this is not the case. In fact, the concept of using the no-shortage condition to develop an optimal plan is considered inappropriate and thus is eliminated from consideration. The reason it is contained in the procedure at all is two-fold. First, it provides an estimate of the absolute minimum amount of water that is expected to be available for use. Secondly, it provides a basis to determine what the advantages are of systematized operational plans over and above the normally applied water-rights and water-supply analysis procedures.

It should also be recognized that up to this point only the historical sequence is being used to develop preliminary estimates of the sizes of the facilities required to meet the demands that the water resource is capable of serving. Use of only the historical sequence will continue in Step Four but will prepare the way for using stochastic sequences in Step Five to analyze the impact that stochastic variability has on the decisions considered desirable.

The procedure described in this chapter for estimating water yields from single-basin and multibasin systems was not applied exactly as presented. For computational convenience, the procedure depicted in Figure 16 was bypassed. Because of the availability of prior investigations, it was not believed to be necessary to follow the procedure explicitly in the example problem, and reservoir sizes shown in Table 1 were accepted as the proper sizes that would normally be developed by this step. However, in the initial planning of a new system, a procedure similar to that described in this chapter should be followed.

# V. PLAN IMPROVEMENT

This chapter describes, in detail, a procedure by which the set of reservoirs and canals identified as attractive in Chapter IV (Step Three as shown in Figure 2) is analyzed in other than a firm-yield context. The purpose of this analysis is to strive to identify the best system configuration and operation criteria for the problem being analyzed at various prespecified staging times on the demand-buildup curve.

The increasing demands, as presented in the example problem, require that canal capacities and reservoir sizes increase over time. In addition, shortages are tolerated economically and physically, but are assigned a penalty cost consistent with the cost of not delivering the amounts of water desired. Also, to help reduce computation time, this step uses only the filled-in and projected historical sequence upon which to base its conclusions; none of the stochastic sequences generated in Step Two are used. The stochastic sequences are used later, in Step Five, to identify the impact of hydrologic variability and to modify as required the system configuration arrived at in this step. This is done in the manner described in Chapters VII and VIII.

Based upon the above and previous steps in this planning procedure, Step Four is designed to evaluate the trade-off between the decreased cost of building smaller storage and transfer facilities and the resulting increase in cost associated with shortages. The criterion used to evaluate this trade-off is a minimum cost objective function, where the costs included are the capital costs, the annual operation costs, the penalty costs for shortages, and the cost of any water imported, all discounted as appropriate to a present value quantity for all plans analyzed.

## General Approach

The approach to plan improvement presented here (selection of a staging plan for use in the Step Five analysis) is divided into four major activities. They are shown graphically in Figure 18 and briefly described as follows.

Activity One determines a good estimate of the size and desirable operation criteria of the required ultimate system components (e.g., the 2020 plan). This estimate will be later improved in Activity Four as may be necessary.

Activity Two determines the size and desirable operation criteria of the required initial system components for use in Step Five (e.g., the 1990 plan).

Activity Three determines the size and desirable operation criteria of system components at preselected, intermediate staging times on the demand-buildup curve (e.g., the 2000 and 2010 plans). Selection of the required system components is conditioned by the fact that those chosen at early points on the demand-buildup curve are in existence when selecting and sizing subsequent components and contribute to the present value cost of the entire implementation plan.

Activity Four determines to what extent the system estimated in Activity One needs to be modified as a result of the Activity Two and Three analyses. If major changes in the system are found to be desirable, this may lead to a reevaluation of each of the chosen system configurations prior to selecting the final staging plan for use in Step Five.

This plan improvement procedure was tested on the example problem and appears to be reasonable. It is recognized that other procedures may be employed; in fact, an unlimited number of possibilities theoretically exist. The one selected meets the test of practicality and shows planning concepts in sufficient detail to guide others in their selection of approaches to particular cases. The remainder of this chapter discusses, by activity, the suggested procedure and the results of applying it to the example problem.

## Sizing the Ultimate System Components and Developing Operation Criteria (Step Four—Activity One)

The product of Activity One is a good estimate of the sizes and desirable operation criteria of the minimum-cost network of canals and reservoirs comprising the ultimate system. The primary reason for analyzing the ultimate system first is to get an accurate estimate of the size of the ditch portion of the canals to be constructed.

In developing the general analysis procedure, it was assumed that the excavation, rights-of-way, bridges, pump-housing foundation, and other related appurtenances (the ditch portion of the canal facility) would have to be purchased and constructed to its ultimate size at the time of initial construction. Conversely, it was assumed that the pumps, motors, power facilities, and related appurtenances could be staged over time. As was demonstrated in the example problem, where the portion of the ditch to be staged comprised about 50 percent of the total cost of the entire canal facility, optimized staging can save as much

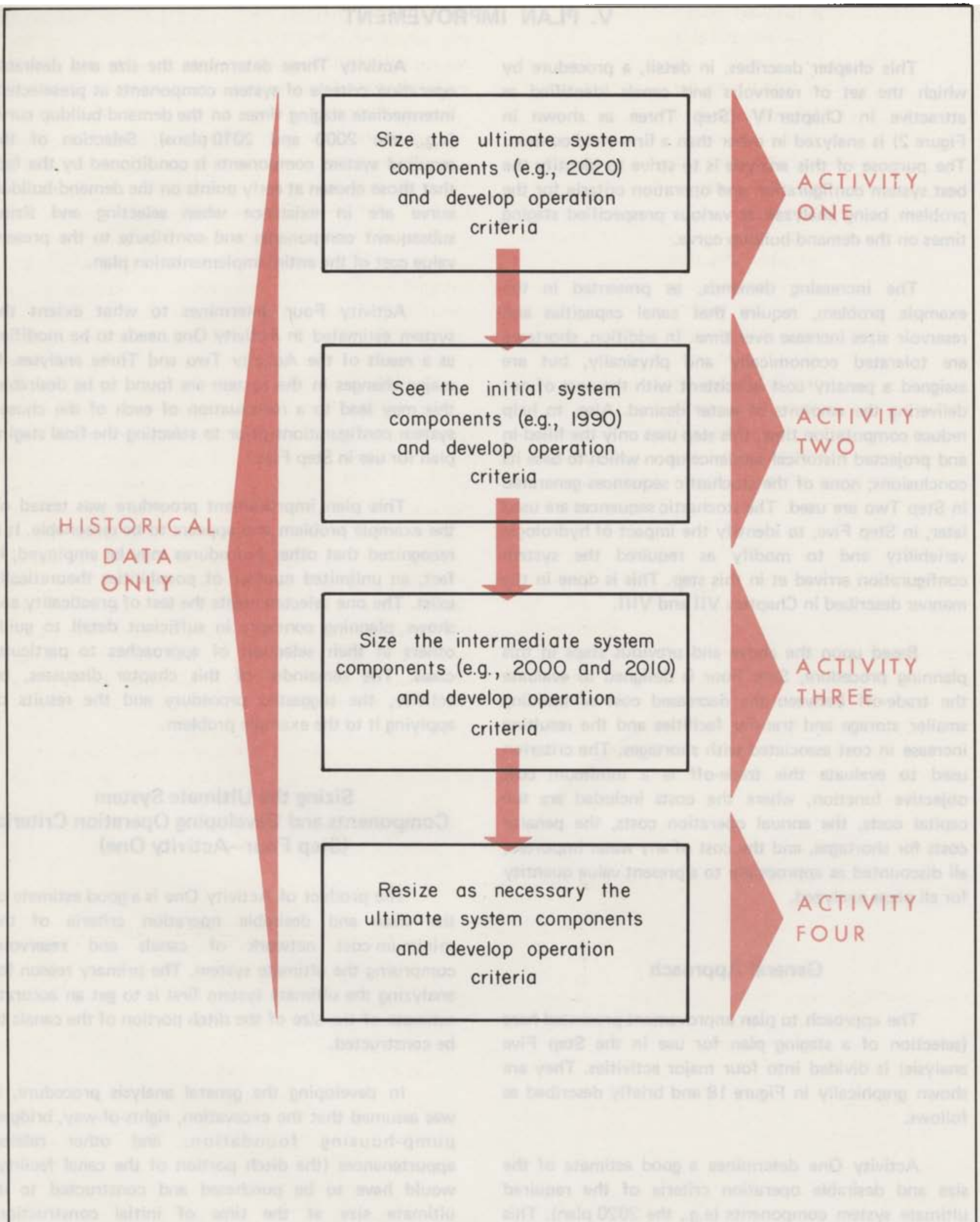



Figure 18

Activities in Step Four, Plan Improvement as 10 percent of the cost of the pump-canal facilities over the life of the project. With this in mind,

- the 36-year historical reservoir inflow and prespecified demand sequence, projected to the 2020 condition,
- the physical and cost data developed in Step Two,
- an over-specified set of interconnected canals as shown in Figure 19a, and
- the set of reservoir sizes identified in Step Three as displayed in Table 1

are used as input to SIM-III to evaluate the system performance of selected alternate system configurations and operation criteria over a prespecified economic life (e.g., 100 years) and simulation period (e.g., 36 years) to find by the process of orderly elimination the minimum-cost ultimate system.

The following describes the concepts behind this analysis procedure and discusses it in the context of finding a solution to the example problem. In determining the ultimate configuration a user-controlled gradient search procedure is used to help identify which facilities of an over-specified set are required to meet the demands imposed upon the system. It starts by identifying which of the facilities within the system are the most expensive from a unit-cost standpoint. For example, in the case of reservoir storage, the cost per acre-foot of maximum capacity is quantified for all reservoirs in the manner shown in Table 1. By looking at this information the user can determine which of the reservoirs can be increased or decreased in size to help minimize the unit storage cost.

Next the unit cost of canal capacity is analyzed with all canals set at the same size, as shown in Table 2. Those canals with higher unit costs that are located on alternate delivery paths are earmarked for possible elimination. Because canals have two components of cost (a capital cost and an operation cost which is a function of head), both must be comparatively considered in arriving at the unit cost of the canal facilities. In the example problem it was found that, prior to optimization, the pump-power cost converted to a present value figure was approximately equal to the capital cost; however, after optimization, the capital cost component was reduced to less than half the present value of the power cost. It was also identified that the unit cost and total cost of the canal facilities far exceeded the corresponding costs of the storage facilities. The allocation of the costs among the various components of the system was approximately as shown in Table 5. Here, and throughout Activity One, the present value cost response is the sum of (1) the construction costs multiplied by a present value factor equal to unity, and (2) the average annual operation costs multiplied by the total area under the 100-year present value curve shown in Figure 5 (i.e., 24.50).

Note that the percentages in the "initial plan" column of Table 5 are not for the optimal plan, but are

**Table 5.—Present Value Costs of System Components**

| SYSTEM COST COMPONENTS | INITIAL PLAN* PRESENT VALUE COST (BILLIONS OF DOLLARS) | INITIAL PLAN* PERCENT OF TOTAL | BEST PLAN† PRESENT VALUE COST (BILLIONS OF DOLLARS) | BEST PLAN† PERCENT OF TOTAL |
|---|---|---|---|---|
| Reservoirs | 0.37 | 3.8 | 0.38 | 5.6 |
| Canals—Total | 4.47 | 46.4 | 1.96 | 28.0 |
| *Ditch Portion* | *1.56* | *35.0* | *.71* | *35.0* |
| *Staged Portion* | *2.91* | *65.0* | *1.25* | *65.0* |
| Power Cost (Pumping) | 4.28 | 44.4 | 4.20 | 60.1 |
| Import Water | .31 | 3.2 | .37 | 5.3 |
| Deficits | .21 | 2.2 | .07 | 1.0 |
| TOTAL | 9.64 | 100.0 | 6.99 | 100.0 |

* This "initial plan" corresponds to simulation run 1 shown in Figure 19a.
† This "best plan" corresponds to the one shown in Figure 25d.

based on the system configuration with all of the canals shown in Figure 19a built at a large capacity (20,000 cfs). The "best plan" column represents the plan determined, later in this step, to be the best ultimate configuration based upon only the historical data sequence.

It is apparent from Table 5 that the largest savings in cost can be obtained by either reducing the size or number of canals or by optimizing the operation policy of the system in a manner that would reduce the power costs. With this in mind, the gradient search procedure sought to find which of the over-specified canals could be eliminated, and by how much this canal elimination process would reduce the total cost of the system. A 20 to 30 percent reduction in canal costs was expected; however, as can be seen in Table 5, the canal costs were reduced by more than 55 percent while the total cost was reduced by 25 percent. The procedure used to accomplish this is described in the following paragraphs.

Because the distribution of storage affects the degree to which optimal operation of a system is possible, and because the decision to eliminate canals of high unit cost should be based upon deviating from a system with proper distribution of storage facilities, a series of runs with all canals built was made first to determine the proper allocation of storage in the eastern and western areas. To limit the size of the problem being analyzed for the example considered here, only the sizes of reservoir 1 in the west and reservoir 5 in the east were varied from the sizes discussed in Chapter II. Using the over-specified system shown in Figure 19a as the point of departure, the analysis concentrated upon finding which of the 33 links in Figure 19a could be eliminated without significantly reducing the system's capability to deliver needed water.

With the east versus west storage distribution being solved only for the "all-canals-built" case and the need for three canals entering node 14, in question (canals 25, 26, and 27), several simulation runs were made to determine which of the canal routes shown in Figure 19a should be eliminated to arrive at the lowest cost system.

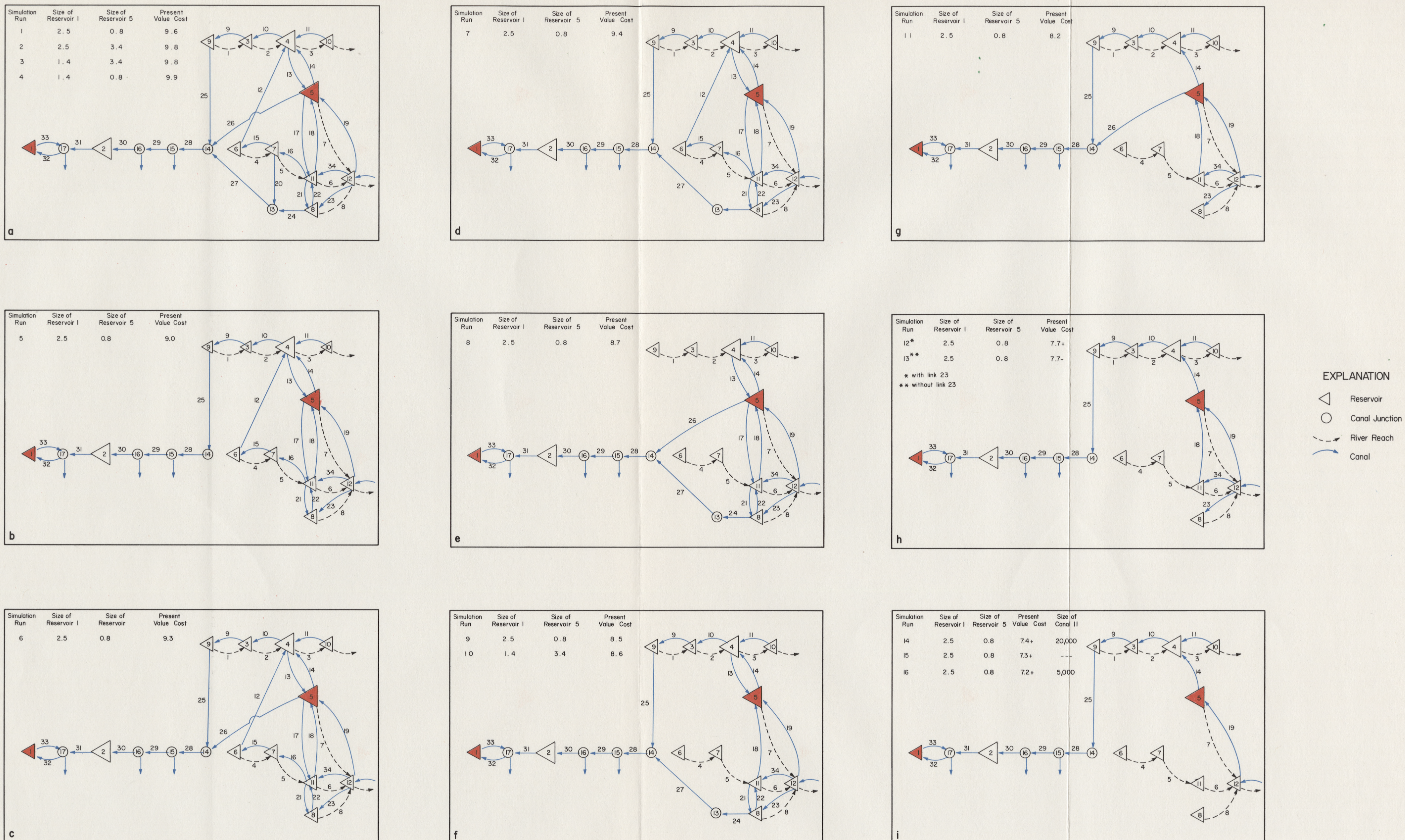
In the example problem most of the 2020 demands (56 percent, as shown on Figure 20) are met by import supplies through reservoir 12. Of that supplied by in-state sources, about half (46 percent) originates from reservoir 4. Therefore it seems reasonable that an attractive transfer system should pass through both reservoirs 12 and 4 on its way west to the high-demand areas adjacent to reservoirs 1 and 2. Unit canal costs calculated in previous runs also indicated that the route westward through reservoir 4 was the most attractive. To verify this solution several simulation runs were made with and without canals 25, 26, and 27. The routes selected for each of these runs and the results are shown in Figures 19b, 19c, 19d, and 19e. These results verified the manual estimate, and Figure 19b shows that the canal 25 route is the alternative of lowest cost at a present value cost of $9.0 billion (a reduction of $0.6 billion).

The next task is to find whether canals with low utilization rates[22](i.e., canals 12, 13, 15, 16, 17, 20, 21, and 22) that were not eliminated previously can be eliminated without drastically reducing the capability of the system to deliver water. Another set of computer runs was made as shown in Figures 19f, 19g, and 19h. As portrayed in Figure 19h, the route through canal 25 without the low-utilization canals gives the lowest cost configuration of the three analyzed at a present value cost of $7.7 billion (an additional reduction of $1.3 billion).

Attention is now turned toward finding which of the canals connecting reservoirs 5, 8, 11, and 12 are desirable. Figure 19h shows that by eliminating canal 23 an even further reduction in cost is obtained. Similarly, by analyzing in detail the unit cost output of this run, canal 19 between reservoirs 12 and 5 is found to be more attractive than the combination of canals 34 and 18. To verify this point, the run represented by Figure 19i was made. The results indicate that it is cheaper on the average to permit flow coming from reservoirs 6, 7, 8, or 11 to flow downstream to reservoir 12 and then be pumped up through canal 19 instead of being pumped across through canal 18. The money saved in not constructing the additional canals apparently is greater than the extra power cost incurred by pumping additional water through canal 19. The present value cost is now $7.2 billion, reflecting a fatal reduction of $2.4 billion from run 1.

The results of this analysis eliminate most of the pump-canal facilities and lead to a solution consisting of a single stem of canals that intercepts most of the storage and in-system supply on its way toward the points of high demand in West Texas. Reflecting upon this solution, it becomes somewhat obvious that, in general, it should be cheaper to concentrate as much

[22] "Utilization rate" refers to the ratio of the actual amount of water transferred by a canal during the simulation period versus the amount of water it could have transferred flowing constantly at maximum capacity. The results to this point indicate less than 5 percent utilization rate for canals 11, 12, 13, 15, 16, 17, 20, 21, and 22, whereas a normal canal utilization rate should exceed 60 percent.




a

| Simulation Run | Size of Reservoir 1 | Size of Reservoir 5 | Present Value Cost |
|---|---|---|---|
| 1 | 2.5 | 0.8 | 9.6 |
| 2 | 2.5 | 3.4 | 9.8 |
| 3 | 1.4 | 3.4 | 9.8 |
| 4 | 1.4 | 0.8 | 9.9 |

b

| Simulation Run | Size of Reservoir 1 | Size of Reservoir 5 | Present Value Cost |
|---|---|---|---|
| 5 | 2.5 | 0.8 | 9.0 |

c

| Simulation Run | Size of Reservoir 1 | Size of Reservoir | Present Value Cost |
|---|---|---|---|
| 6 | 2.5 | 0.8 | 9.3 |

d

| Simulation Run | Size of Reservoir 1 | Size of Reservoir 5 | Present Value Cost |
|---|---|---|---|
| 7 | 2.5 | 0.8 | 9.4 |

e

| Simulation Run | Size of Reservoir 1 | Size of Reservoir 5 | Present Value Cost |
|---|---|---|---|
| 8 | 2.5 | 0.8 | 8.7 |

f

| Simulation Run | Size of Reservoir 1 | Size of Reservoir 5 | Present Value Cost |
|---|---|---|---|
| 9 | 2.5 | 0.8 | 8.5 |
| 10 | 1.4 | 3.4 | 8.6 |

g

| Simulation Run | Size of Reservoir 1 | Size of Reservoir 5 | Present Value Cost |
|---|---|---|---|
| 11 | 2.5 | 0.8 | 8.2 |

h

| Simulation Run | Size of Reservoir 1 | Size of Reservoir 5 | Present Value Cost |
|---|---|---|---|
| 12* | 2.5 | 0.8 | 7.7+ |
| 13** | 2.5 | 0.8 | 7.7- |

* with link 23
** without link 23

i

| Simulation Run | Size of Reservoir 1 | Size of Reservoir 5 | Present Value Cost | Size of Canal 11 |
|---|---|---|---|---|
| 14 | 2.5 | 0.8 | 7.4+ | 20,000 |
| 15 | 2.5 | 0.8 | 7.3+ | --- |
| 16 | 2.5 | 0.8 | 7.2+ | 5,000 |

NOTES:
1) Reservoir sizes are in millions of acre-feet.
2) Costs are in billions of dollars; these are present value system costs.
3) Shaded reservoirs are those in which size changes are permitted; others are fixed as shown in Table 1.
4) Canals 31, 32, and 33 are sized at 50,000 cfs to avoid physical distribution restrictions; however, they were priced at 10,000 cfs to simulate maximum size of any single distribution canal.
5) Other canal capacities are at 20,000 cfs, except as shown in Figure 19i where Canal 11 size is varied.

Figure 19
Plan Improvement Runs for Step Four, Activity One

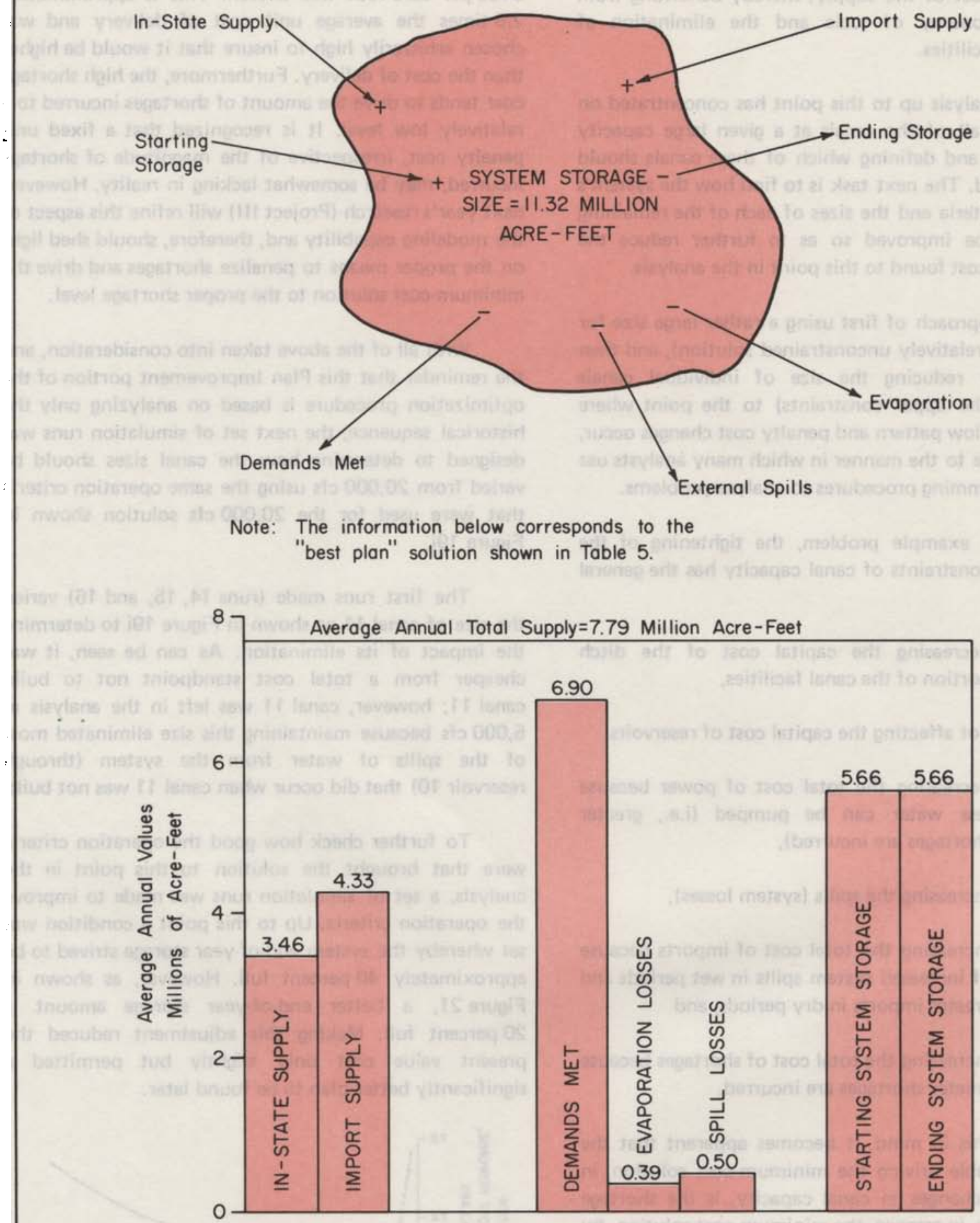



Figure 20
System Mass Balance

capacity into a single set of capital facilities that intercepts most of the supply, thereby benefiting from both an economy of scale and the elimination of redundant facilities.

The analysis up to this point has concentrated on maintaining all of the canals at a given large capacity (20,000 cfs) and defining which of these canals should be eliminated. The next task is to find how the system's operation criteria and the sizes of each of the remaining canals can be improved so as to further reduce the $7.2 billion cost found to this point in the analysis.

The approach of first using a rather large size for all canals (a relatively unconstrained solution), and then subsequently reducing the size of individual canals (tightening the upper constraints) to the point where undesirable flow pattern and penalty cost changes occur, is comparable to the manner in which many analysts use linear programming procedures to analyze problems.

In the example problem, the tightening of the upper flow constraints of canal capacity has the general effect of:

- decreasing the capital cost of the ditch portion of the canal facilities,
- not affecting the capital cost of reservoirs,
- decreasing the total cost of power because less water can be pumped (i.e., greater shortages are incurred),
- increasing the spills (system losses),
- increasing the total cost of imports because of increased system spills in wet periods and greater imports in dry periods, and
- increasing the total cost of shortages because greater shortages are incurred.

With this in mind, it becomes apparent that the primary variable driving the minimum-cost solution, in response to changes in canal capacity, is the shortage costs.[23] Thus, in essence, the minimum-cost solution, by definition, becomes a solution which identifies the proper level of shortages. This solution, as will be shown later, is very dependent upon the unit cost assigned to incurred shortages.

[23] There is a very high correlation between reduced power costs and the shortages incurred, as should be expected.

For the example problem, a penalty cost of $100 per acre-foot was chosen. This is approximately 2.5 times the average unit cost of delivery and was chosen arbitrarily high to insure that it would be higher than the cost of delivery. Furthermore, the high shortage cost tends to drive the amount of shortages incurred to a relatively low level. It is recognized that a fixed unit penalty cost, irrespective of the magnitude of shortage incurred, may be somewhat lacking in reality. However, next year's research (Project III) will refine this aspect of the modeling capability and, therefore, should shed light on the proper means to penalize shortages and drive the minimum-cost solution to the proper shortage level.

With all of the above taken into consideration, and the reminder that this Plan Improvement portion of the optimization procedure is based on analyzing only the historical sequence, the next set of simulation runs was designed to determine how the canal sizes should be varied from 20,000 cfs using the same operation criteria that were used for the 20,000 cfs solution shown in Figure 19i.

The first runs made (runs 14, 15, and 16) varied the size of canal 11 as shown in Figure 19i to determine the impact of its elimination. As can be seen, it was cheaper from a total cost standpoint not to build canal 11; however, canal 11 was left in the analysis at 5,000 cfs because maintaining this size eliminated most of the spills of water from the system (through reservoir 10) that did occur when canal 11 was not built.

To further check how good the operation criteria were that brought the solution to this point in the analysis, a set of simulation runs was made to improve the operation criteria. Up to this point a condition was set whereby the system end-of-year storage strived to be approximately 40 percent full. However, as shown in Figure 21, a better end-of-year storage amount is 20 percent full. Making this adjustment reduced the present value cost only slightly but permitted a significantly better plan to be found later.

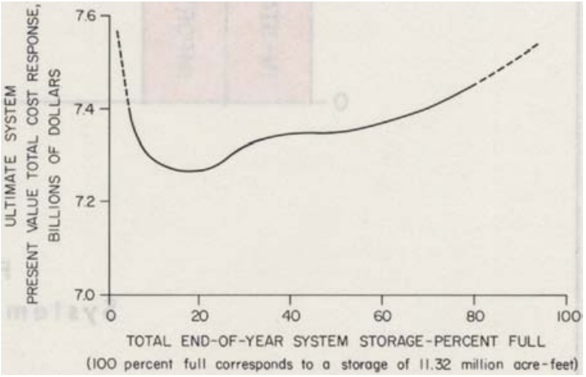



Figure 21.—Total Cost Response Versus End-of-Year Storage Criterion

Next, simulation runs were made to find how the system would respond to resizing the canals on the mainstem (canals 19, 14, 10, 9, 25, 28, 29, and 30). These runs were divided into two sets. The results of the first set of runs are shown in Figure 22. In these runs the capacity of the canals on the mainstem was varied uniformly from 14,000 to 22,000 cfs to determine the present value cost response of canal size changes. The results indicate that, for the "all-canal-sizes-equal" case, the 20,000 cfs size is the best or very near the best size for the mainstem. However, the present value cost response is less sensitive to increased canal capacity than to decreased capacity as more and more shortages are incurred. This can be explained rather easily by considering that for each acre-foot shorted a penalty of $100.00 is incurred; whereas, for each 1,000 cfs increase in the capacity of all canals in the mainstem, an increase in the average annual cost of approximately $1.50 per acre-foot is incurred. In essence, this indicates that at high penalty costs it is attractive to build "capital heavy" plans instead of taking the chance of incurring excessive penalties due to under-designed facilities. In other words, the higher the penalty costs are,

- The larger capacity and more expensive will be the optimal interconnecting pump-canals,
- the less will be the optimal shortage amount, and
- the greater will be the actual yield, due to decreased system losses (less spills).

The second set of runs made was designed to find how different sized canals at various portions of the system would change and hopefully further reduce the present value of total system cost response. Herein knowledge of the characteristics of the demand distribution versus the supply and import distribution was beneficial in guiding how the system should be sized and operated. As shown in Figure 23,

- most of the demands located at node 17 occur in July, August, and September (75 percent),[24]
- most of the in-state supply occurs prior to the end of June, and
- import waters are assumed to be available from January to June.

Therefore, the problem became one of providing sufficient storage in East Texas and West Texas to store, on an interim basis, the runoff and import waters needed to meet the July, August, and September demands in West Texas (node 17).

As shown in Figure 23 and verified with the previous simulation runs, the average storage needed in the west (reservoirs 1 and 2) is about 4.5 million acre-feet during June to meet average flow conditions if no pumping is possible during July, August, and September. However, as shown in Figure 24, this 4.5 million acre-foot capacity can be reduced to approximately 3.5 million acre-feet if the mainstem canal capacity westward is set at approximately 13,800 cfs and pumping continues throughout the year. In addition, the 3.5 million acre-foot capacity can be reduced to 2.8 million acre-feet if the mainstem canal capacity westward is set at 15,500 cfs. However, this will handle adequately only the average conditions; a more droughty condition (a condition of higher demand) requires either more storage or more canal capacity in order to perform adequately. With this in mind, it was contemplated that at least a 20 to 30 percent increase in the canal capacity over and above that required to meet average conditions would be required; therefore, a minimum capacity of approximately 17,000 to 18,000 cfs in canals 25, 28, 29, and 30 with a corresponding 3.0 to 4.0 million acre-foot total storage in the west in reservoirs 1 and 2 would probably be in the neighborhood of that required to perform adequately.

The 17,000 to 18,000 cfs canal size might also apply to canals 9 and 10, but it is doubtful whether 18,000 cfs will be adequate for canals 14 and 19 for the following problem-specific reasons.

First of all, reservoir 12 (the point at which import is accepted) has only 0.25 million acre-feet of storage capacity—very little buffering capability. Import water is assumed to be available at a maximum of 1.6 million acre-feet per month and this is roughly equivalent to 26,500 cfs. In simulation runs with canals 14 and 19 set at 25,000 cfs, for the historical data sequence, both of these canals flowed at the maximum capacity at least once in every two years but yet recorded only about a 30 percent utilization rate for the entire simulation period.

Therefore the proper size for links 14 and 19 must be somewhere between 20,000 and 25,000 cfs, depending upon the total unit cost of canal usage[25] and

[24] Ninety-five percent of the total demands occur at node 17, and 75 percent of these demands occur during July, August, and September.

[25] The "unit cost of canal usage" refers to the total cost of a canal (capital plus power plus maintenance costs converted to an average annual figure) divided by the amount of water actually transferred through it during the simulation period.

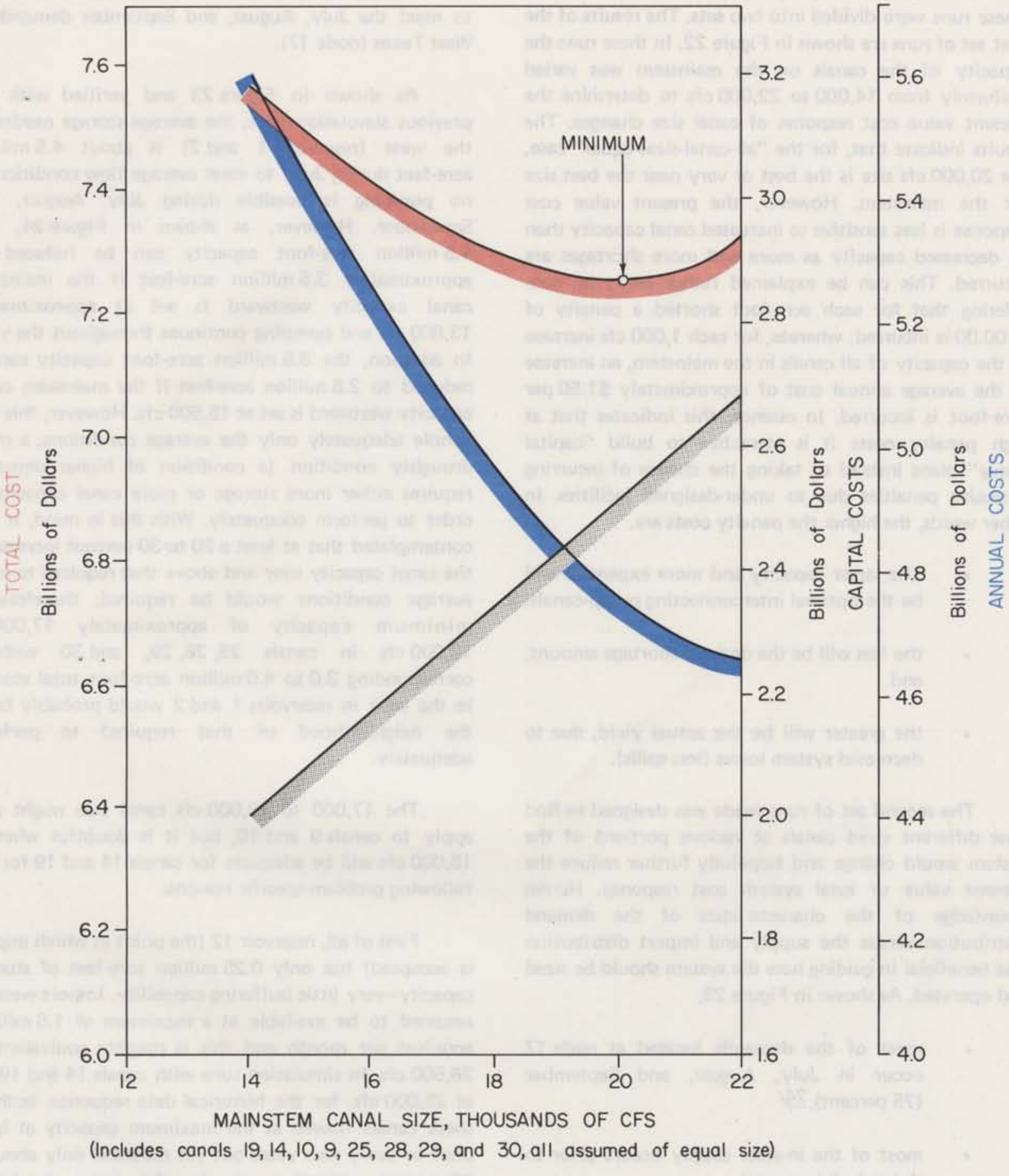



Figure 22

**Present Value Costs Versus Mainstem Canal Size, Initial Estimates for 2020 Demand Level**

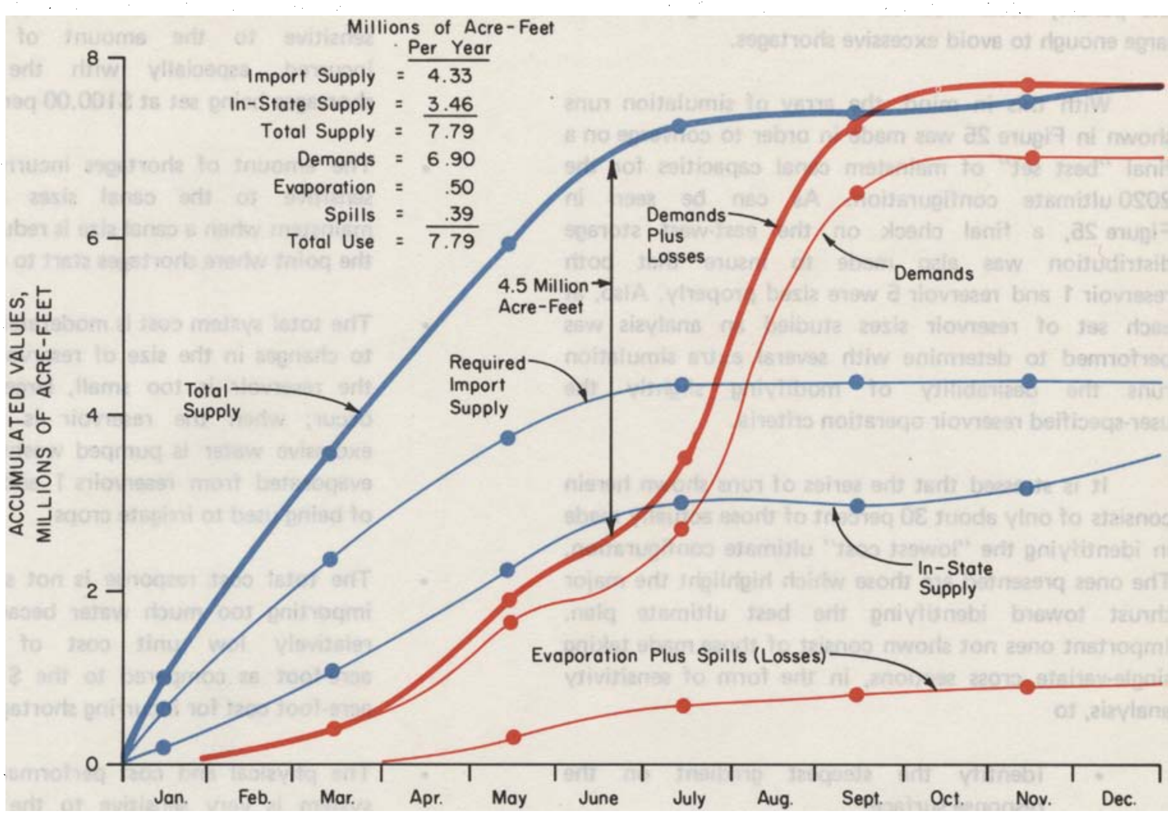



**Figure 23.—Seasonal Distribution of Average Annual Demand, Supply, Evaporation, and Spills**

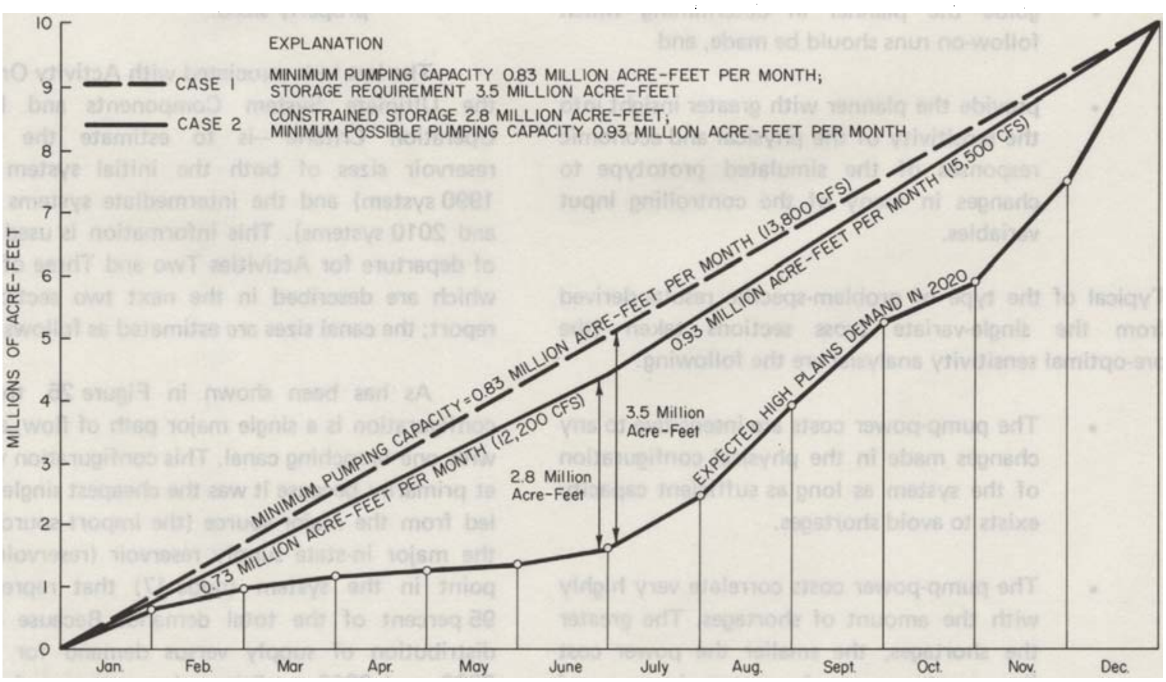



**Figure 24.—Storage Requirement in West Texas (Reservoirs 1 and 2) Versus Mainstem Canal Capacity**

the penalty costs incurred for not building the canals large enough to avoid excessive shortages.

With this in mind, the array of simulation runs shown in Figure 25 was made in order to converge on a final "best set" of mainstem canal capacities for the 2020 ultimate configuration. As can be seen in Figure 25, a final check on the east-west storage distribution was also made to insure that both reservoir 1 and reservoir 5 were sized properly. Also, at each set of reservoir sizes studied an analysis was performed to determine with several extra simulation runs the desirability of modifying slightly the user-specified reservoir operation criteria.

It is stressed that the series of runs shown herein consists of only about 30 percent of those actually made in identifying the "lowest cost" ultimate configuration. The ones presented are those which highlight the major thrust toward identifying the best ultimate plan. Important ones not shown consist of those made taking single-variate cross sections, in the form of sensitivity analysis, to

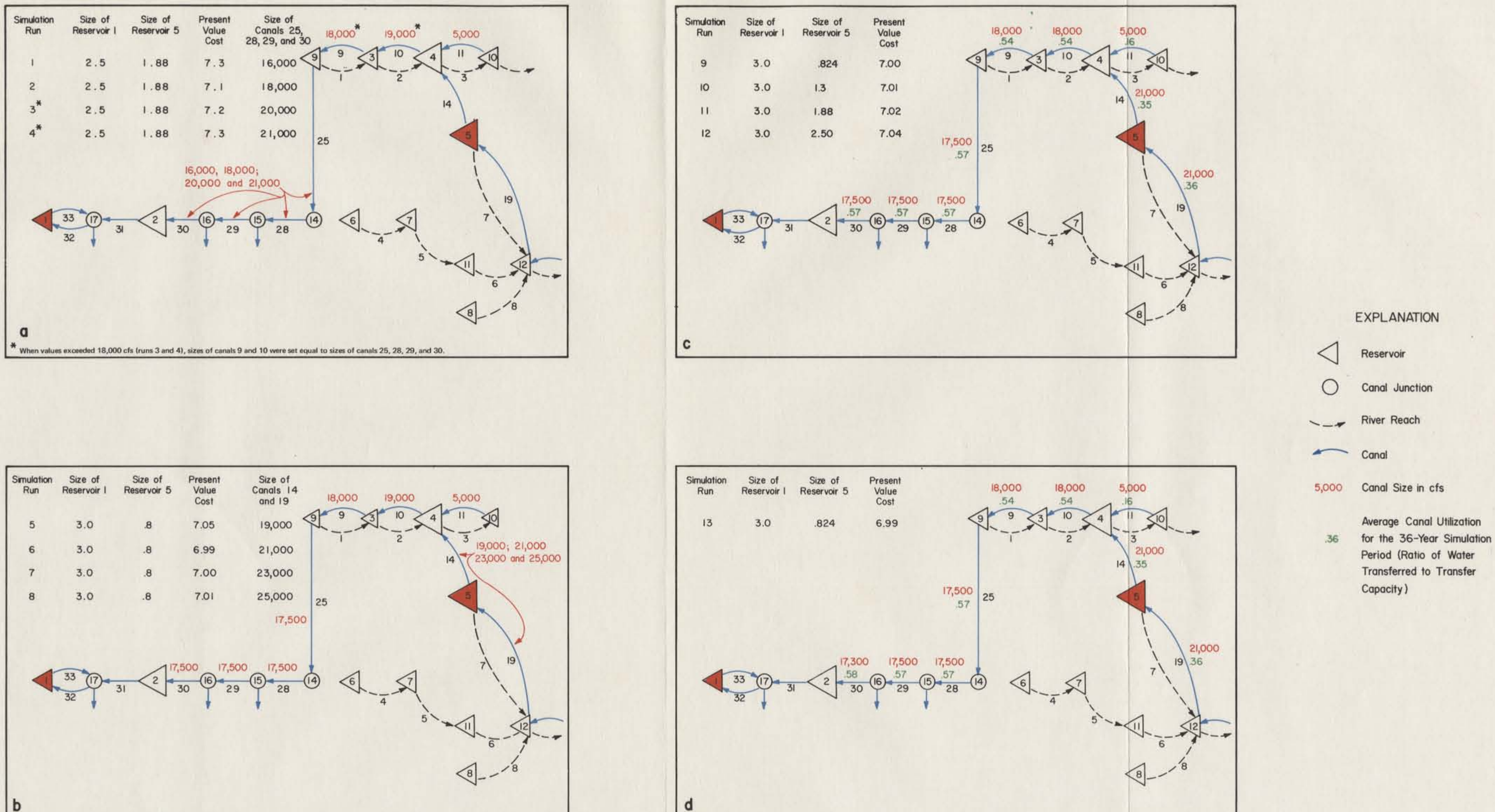
- Identify the steepest gradient on the response surface,
- help insure that the steepest gradient about a point being analyzed is not a local one which will lead the planner to a local minimum instead of a global minimum,
- guide the planner in determining which follow-on runs should be made, and
- provide the planner with greater insight into the sensitivity of the physical and economic responses of the simulated prototype to changes in many of the controlling input variables.

Typical of the type of problem-specific results derived from the single-variate cross sections taken (the pre-optimal sensitivity analysis) are the following:

- The pump-power costs are insensitive to any changes made in the physical configuration of the system as long as sufficient capacity exists to avoid shortages.
- The pump-power costs correlate very highly with the amount of shortages. The greater the shortages, the smaller the power cost (i.e., not as much water is pumped westward).
- The total cost response of the system is very sensitive to the amount of shortages incurred, especially with the cost of shortages being set at $100.00 per acre-foot.
- The amount of shortages incurred is very sensitive to the canal sizes along the mainstem when a canal size is reduced below the point where shortages start to occur.
- The total system cost is moderately sensitive to changes in the size of reservoir 1. When the reservoir is too small, large shortages occur; when the reservoir is too large, excessive water is pumped westward to be evaporated from reservoirs 1 and 2 instead of being used to irrigate crops.
- The total cost response is not sensitive to importing too much water because of the relatively low unit cost of $3.50 per acre-foot as compared to the $100.00 per acre-foot cost for incurring shortages.
- The physical and cost performance of the system is very sensitive to the operation criteria used.
- The cost response of the system is not very sensitive to changes in the size of individual reservoirs as long as the total amount of storage in various portions of the system is properly sized.

The last task associated with Activity One—"Sizing the Ultimate System Components and Developing Operation Criteria"—is to estimate the canal and reservoir sizes of both the initial system (e.g., the 1990 system) and the intermediate systems (the 2000 and 2010 systems). This information is used as a point of departure for Activities Two and Three of Step Four which are described in the next two sections of this report; the canal sizes are estimated as follows.

As has been shown in Figure 25, the ultimate configuration is a single major path of flow, a mainstem with one branching canal. This configuration was arrived at primarily because it was the cheapest single route that led from the major source (the import source) through the major in-state supply reservoir (reservoir 4) to the point in the system (node 17) that represents over 95 percent of the total demand. Because the spatial distribution of supply versus demand for the 1990, 2000, and 2010 conditions does not vary from that of the ultimate 2020 condition, it is safe to assume, as was




**a**

| Simulation Run | Size of Reservoir 1 | Size of Reservoir 5 | Present Value Cost | Size of Canals 25, 28, 29, and 30 |
|---|---|---|---|---|
| 1 | 2.5 | 1.88 | 7.3 | 16,000 |
| 2 | 2.5 | 1.88 | 7.1 | 18,000 |
| 3* | 2.5 | 1.88 | 7.2 | 20,000 |
| 4* | 2.5 | 1.88 | 7.3 | 21,000 |

* When values exceeded 18,000 cfs (runs 3 and 4), sizes of canals 9 and 10 were set equal to sizes of canals 25, 28, 29, and 30.

**b**

| Simulation Run | Size of Reservoir 1 | Size of Reservoir 5 | Present Value Cost | Size of Canals 14 and 19 |
|---|---|---|---|---|
| 5 | 3.0 | .8 | 7.05 | 19,000 |
| 6 | 3.0 | .8 | 6.99 | 21,000 |
| 7 | 3.0 | .8 | 7.00 | 23,000 |
| 8 | 3.0 | .8 | 7.01 | 25,000 |

**c**

| Simulation Run | Size of Reservoir 1 | Size of Reservoir 5 | Present Value Cost |
|---|---|---|---|
| 9 | 3.0 | .824 | 7.00 |
| 10 | 3.0 | 1.3 | 7.01 |
| 11 | 3.0 | 1.88 | 7.02 |
| 12 | 3.0 | 2.50 | 7.04 |

**d**

| Simulation Run | Size of Reservoir 1 | Size of Reservoir 5 | Present Value Cost |
|---|---|---|---|
| 13 | 3.0 | .824 | 6.99 |

NOTES:
1. Reservoir sizes are in millions of acre-feet.
2. Costs are in billions of dollars; these are present value system costs.
3. Shaded reservoirs are those in which size changes are permitted; others are fixed as shown in Table 1.
4. Canals 31, 32, and 33 are sized at 50,000 cfs to avoid physical distribution restrictions; however, they were priced at 10,000 cfs to simulate maximum size of any single distribution canal.

Figure 25

Final Canal Size Adjustments for Step Four, Activity One done, that the same physical set of facilities identified as best for the 2020 condition will likewise be best for the other three conditions. Similarly, because the major change between the 1990, 2000, 2010, and 2020 conditions is the level of demand and required import, as shown in Figure 26, the sizes of both the total storage capacity and the canals on the mainstem are expected to vary nearly linearly with changing average annual demands. Therefore, based on this assumption, Figure 26 shows an estimate of the sizes of reservoir 1 and the canals on the mainstem comprising the "all-canal-sizes-equal" solution discussed previously.

## Present Value Computations for Remaining Activities

Upon preliminarily sizing the ultimate ditch portion of the canal facility, the analysis directs its attention towards finding an optimal system (location, size, and operation criteria) for prespecified points on the demand-buildup curve, starting with the earliest point (the initial configuration) first. At each of the demand points SIM-III is used in the user-oriented iterative manner demonstrated in Activity One. The point of observation for measuring the economic response of the system at each of the demand points (e.g., 1990, 2000, 2010, and 2020) is the beginning of the planning or construction period (e.g., 1985). Again, a 100-year economic life and a 36-year simulation period are used for discussion purposes.

The need to prespecify staging time increments is basic to the analysis procedure. The time increments need not be equal; in fact, their lengths should be based upon an analysis of the shape of the demand-buildup curve, the shape of the present value curve, shortage penalty costs, excess capacity costs, and the greater cost of constructing facilities in stages instead of constructing them to their ultimate size initially. Herein a 20 percent surcharge is added to the cost of construction for each stage (size increase) required. If the demand buildup is extremely delayed as in Figure 27a, the optimal length of the staging increments will tend to be longer at the beginning of the buildup period than at the end. On the other hand, if most of the buildup takes place early in the buildup period as in Figure 27c, the optimal staging

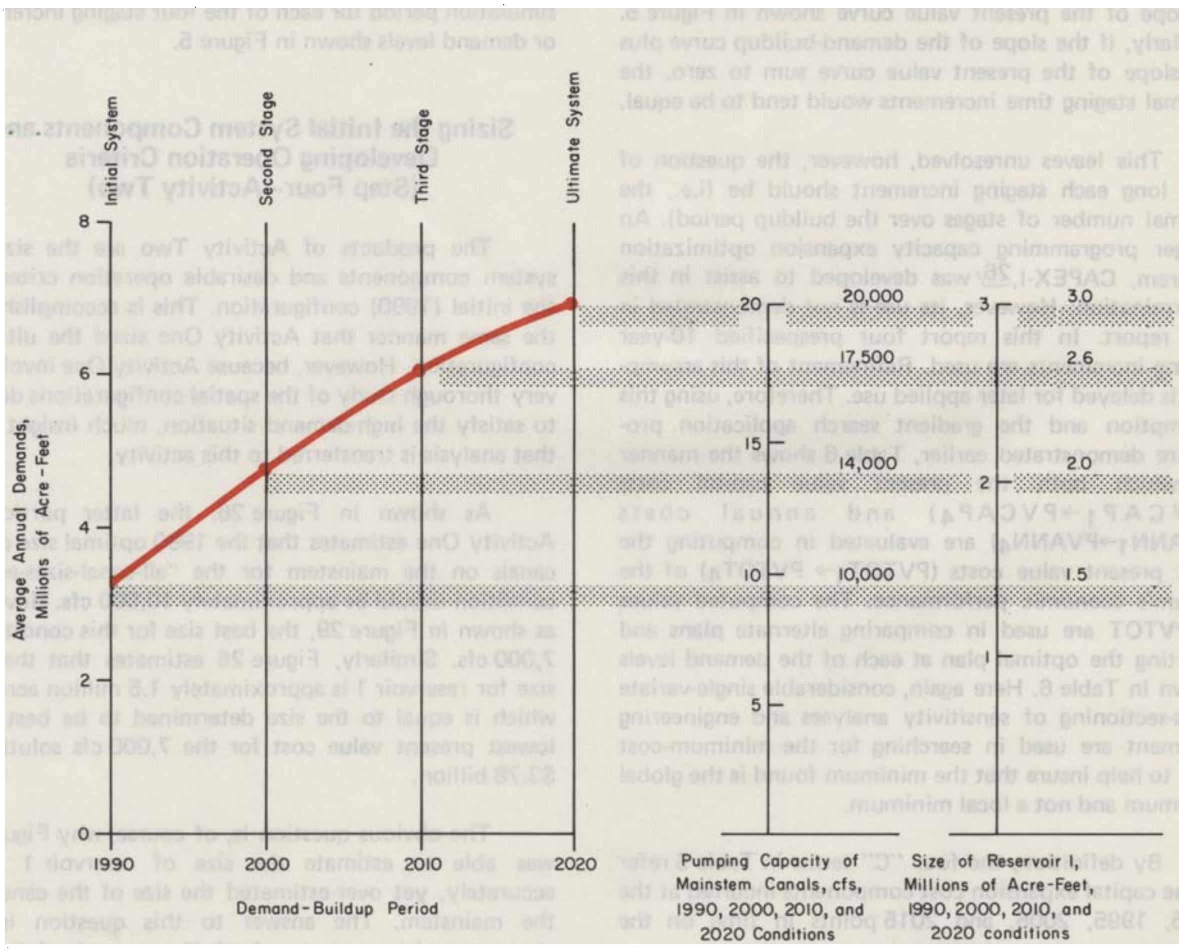



Figure 26.—Developing a Preliminary Staging Plan for Use in Activities Two and Three of Step Four

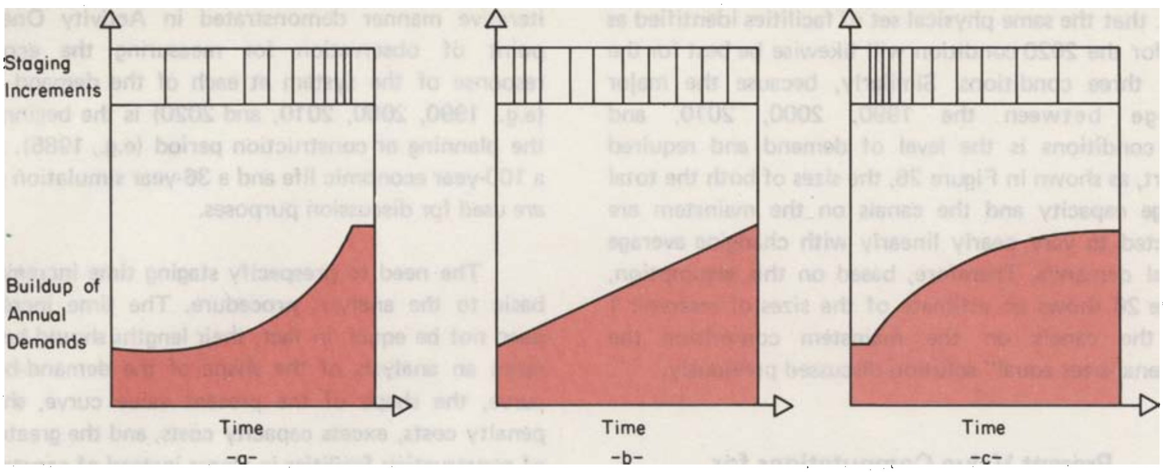



**Figure 27.—Demand Buildup Versus Relative Size of Staging Increments**

increments at the beginning of the buildup period will tend to be shorter than at the end. With a linear buildup curve, as in Figure 27b, the optimal staging increments will also tend to be shorter at the beginning of the buildup period and, with respect to time, will tend to increase in duration roughly proportional to the decrease in slope of the present value curve shown in Figure 5. Similarly, if the slope of the demand-buildup curve plus the slope of the present value curve sum to zero, the optimal staging time increments would tend to be equal.

This leaves unresolved, however, the question of how long each staging increment should be (i.e., the optimal number of stages over the buildup period). An integer programming capacity expansion optimization program, CAPEX-I,[26] was developed to assist in this determination. However, its use is not demonstrated in this report. In this report four prespecified 10-year staging increments are used. Refinement of this assumption is delayed for later applied use. Therefore, using this assumption and the gradient search application procedure demonstrated earlier, Table 6 shows the manner in which both the present value capital costs ($PVCAP_1 \rightarrow PVCAP_4$) and annual costs ($PVANN_1 \rightarrow PVANN_4$) are evaluated in computing the total present value costs ($PVTOT_1 \rightarrow PVTOT_4$) of the system's economic performance. The computed values of PVTOT are used in comparing alternate plans and selecting the optimal plan at each of the demand levels shown in Table 6. Here again, considerable single-variate cross-sectioning of sensitivity analyses and engineering judgment are used in searching for the minimum-cost plan to help insure that the minimum found is the global minimum and not a local minimum.

By definition, the four "C" terms in Table 6 refer to the capital expansion cost components incurred at the 1985, 1995, 2005, and 2015 points in time on the buildup curve shown in Figures 5 and 28. Similarly, the $P_0$, $P_{10}$, $P_{20}$, $P_{30}$ terms are present value factors as defined in Figure 5. $A_1$ through $A_4$ are areas under portions of the present value curve and are also defined in Figure 5. Finally, the four "ANN" terms are averages of the annual cost values incurred over the 36-year simulation period for each of the four staging increments or demand levels shown in Figure 5.

[26] CAPEX-I is described in detail in Volume IIF of this report.

## Sizing the Initial System Components and Developing Operation Criteria (Step Four—Activity Two)

The products of Activity Two are the sizes of system components and desirable operation criteria of the initial (1990) configuration. This is accomplished in the same manner that Activity One sized the ultimate configuration. However, because Activity One involved a very thorough study of the spatial configurations desired to satisfy the high-demand situation, much insight from that analysis is transferred to this activity.

As shown in Figure 26, the latter portion of Activity One estimates that the 1990 optimal size of the canals on the mainstem for the "all-canal-sizes-equal" condition should be approximately 10,000 cfs. However, as shown in Figure 29, the best size for this condition is 7,000 cfs. Similarly, Figure 26 estimates that the best size for reservoir 1 is approximately 1.5 million acre-feet which is equal to the size determined to be best. The lowest present value cost for the 7,000 cfs solution is $3.78 billion.

The obvious question is, of course, why Figure 26 was able to estimate the size of reservoir 1 fairly accurately, yet over-estimated the size of the canals on the mainstem. The answer to this question is not obvious, and is yet unresolved. However, the following plausible reason is given. Even though the size of

**Table 6.—Present Value Algorithm for Evaluating Economic System Response**

| | ACTIVITY TWO | ACTIVITY THREE | | ACTIVITY FOUR |
|---|---|---|---|---|
| | 1990 CONDITION ANALYSIS | 2000 CONDITION ANALYSIS | 2010 CONDITION ANALYSIS | 2020 CONDITION ANALYSIS |
| CAPITAL COST PRESENT VALUE EQUATIONS | $PVCAP_1 =$ $C_0 \times P_0$ | $PVCAP_2 =$ $C_0 \times P_0 +$ $C_{10} \times P_{10}$ | $PVCAP_3 =$ $C_0 \times P_0 +$ $C_{10} \times P_{10} +$ $C_{20} \times P_{20}$ | $PVCAP_4 =$ $C_0 \times P_0 +$ $C_{10} \times P_{10} +$ $C_{20} \times P_{20} +$ $C_{30} \times P_{30}$ |
| ANNUAL COST PRESENT VALUE EQUATIONS | $PVANN_1 =$ $ANN_1 \times (A_1+A_2+A_3+A_4)$ | $PVANN_2 =$ $ANN_1 \times A_1 +$ $ANN_2 \times (A_2+A_3+A_4)$ | $PVANN_3 =$ $ANN_1 \times A_1 +$ $ANN_2 \times A_2 +$ $ANN_3 \times (A_3+A_4)$ | $PVANN_4 =$ $ANN_1 \times A_1 +$ $ANN_2 \times A_2 +$ $ANN_3 \times A_3 +$ $ANN_4 \times A_4$ |
| PRESENT VALUE TOTAL COST | $PVTOT_1 =$ $PVCAP_1 + PVANN_1$ | $PVTOT_2 =$ $PVCAP_2 + PVANN_2$ | $PVTOT_3 =$ $PVACP_3 + PVANN_3$ | $PVTOT_4 =$ $PVCAP_4 + PVANN_4$ |

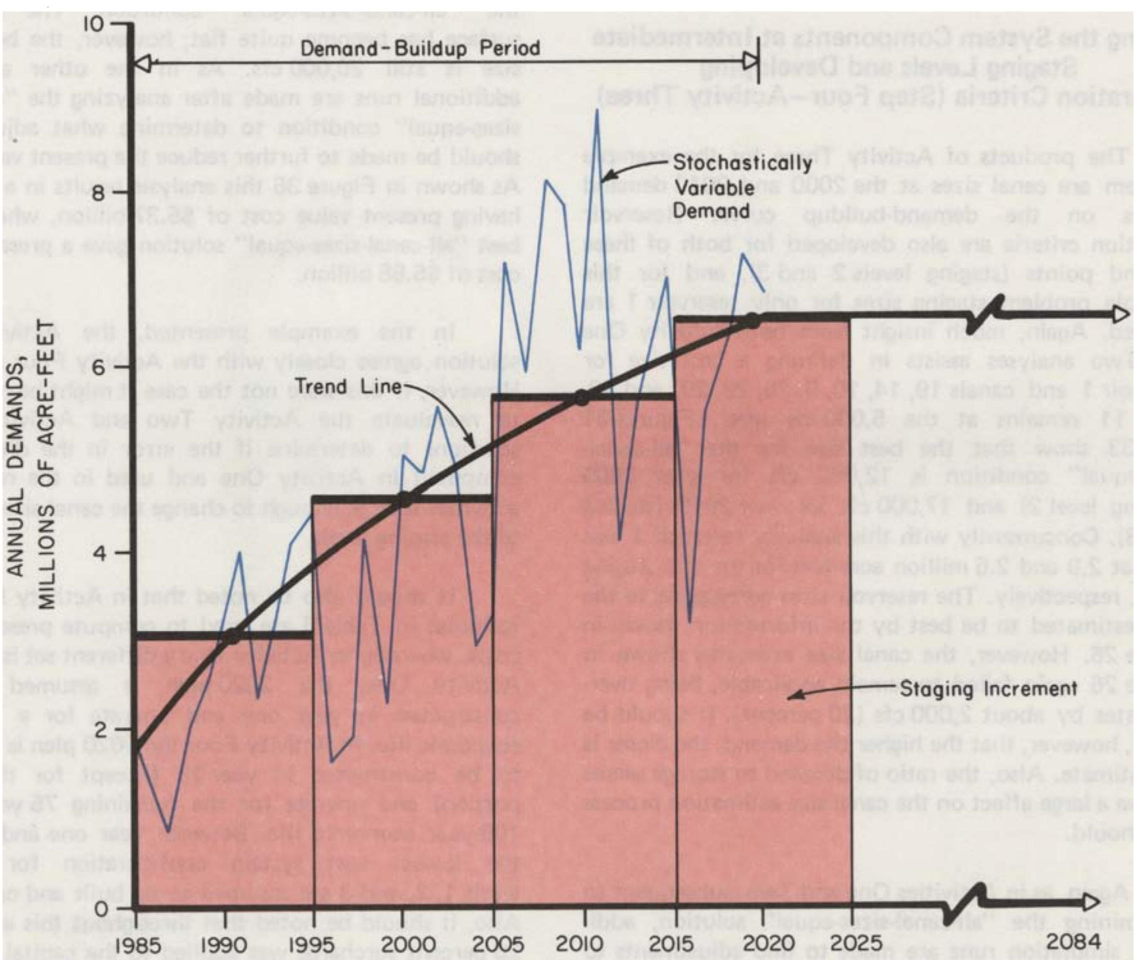



Figure 28.—Demand Buildup and Prespecified Staging Increments

reservoir 1 decreased by 50 percent with a 50 percent decrease in demands, the total terminal storage in West Texas (reservoirs 1 and 2) decreased by only 30 percent. Conversely, the canal sizes transferring water to the west can be expected to be reduced by more than 50 percent. Also, the ratio of demands to total available system storage (East Texas plus West Texas storage) is less than in the 2020 condition by almost a factor of two; therefore, smaller canal sizes are likely.

Subsequent to finding the best size for the "all-canal-sizes-equal" solution, several additional runs are made to find how the sizes of canals 19, 14, 10, 9, 25, 28, 29, and 30 should be varied to further reduce the present value cost. The results of these runs are shown in Figure 30 and indicate that canals 14 and 19 should remain the same size and canals 9, 10, 25, 28, 29, and 30 reduced in size as compared to the "all-canal-sizes-equal" solution. The resulting present value of this improved solution is \$3.68 billion as compared to \$3.78 billion for the "all-canal-sizes-equal" solution. The present value costs computed in this activity and all following activities in this chapter are performed according to the formulas shown in Table 6.

## Sizing the System Components at Intermediate Staging Levels and Developing Operation Criteria (Step Four—Activity Three)

The products of Activity Three for the example problem are canal sizes at the 2000 and 2010 demand points on the demand-buildup curve. Reservoir operation criteria are also developed for both of these demand points (staging levels 2 and 3), and for this example problem staging sizes for only reservoir 1 are selected. Again, much insight from both Activity One and Two analyses assists in defining a best size for reservoir 1 and canals 19, 14, 10, 9, 25, 28, 29, and 30. Canal 11 remains at the 5,000 cfs level. Figures 31 and 33 show that the best size for the "all-canal-sizes-equal" condition is 12,000 cfs for year 2000 (staging level 2) and 17,000 cfs for year 2010 (staging level 3). Concurrently with this analysis, reservoir 1 was sized at 2.0 and 2.6 million acre-feet for the two staging levels, respectively. The reservoir sizes correspond to the sizes estimated to be best by the information shown in Figure 26. However, the canal size estimates shown in Figure 26 again failed to remain applicable, being over-estimates by about 2,000 cfs (20 percent). It should be noted, however, that the higher the demand, the closer is the estimate. Also, the ratio of demand to storage seems to have a large affect on the canal size estimation process as it should.

Again, as in Activities One and Two, subsequent to determining the "all-canal-sizes-equal" solution, additional simulation runs are made to find adjustments to the canal sizes that hopefully will define a lower-cost solution. In this case, lower-cost solutions were found for both the 2000 and 2010 staging levels. Where the "all-canal-sizes-equal" solutions had present value costs of \$4.68 billion and \$5.30 billion, respectively, the subsequent canal size adjustment simulation runs provided present value costs of \$4.55 billion and \$5.10 billion. The canal size adjustment analyses are shown in Figures 32 and 34. Here again, the present value cost computations are performed according to the formulas shown in Table 6.

## Final Sizing of the Ultimate System Components and Developing Operation Criteria (Step Four—Activity Four)

Activity Four includes a resizing of the ultimate system components, if necessary, from those sized in Activity One. As the reader may recall, the primary purpose of Activity One is to size the ultimate configuration in order to obtain an accurate estimate of the cost of the ditch portion of the canal facility (the ditch portion is not staged). Therefore, much of the analysis performed in Activity One is directly applicable to the reevaluation here in Activity Four.

Figure 35 shows the results of sizing the canals for the "all-canal-sizes-equal" condition. The response surface has become quite flat; however, the best canal size is still 20,000 cfs. As in the other activities, additional runs are made after analyzing the "all-canal-sizes-equal" condition to determine what adjustments should be made to further reduce the present value cost. As shown in Figure 36 this analysis results in a solution having present value cost of \$5.37 billion, whereas the best "all-canal-sizes-equal" solution gave a present value cost of \$5.58 billion.

In the example presented, the Activity One solution agrees closely with the Activity Four solution. However, if this were not the case it might be necessary to reevaluate the Activity Two and Activity Three solutions to determine if the error in the ditch costs computed in Activity One and used in the remaining activities is large enough to change the canal sizes at each of the staging levels.

It should also be noted that in Activity Four the formulas in Table 6 are used to compute present value costs, whereas, in Activity One a different set is used. In Activity One, the 2020 plan is assumed to be constructed in year one and operate for a 100-year economic life. In Activity Four the 2020 plan is assumed to be constructed in year 25 (except for the ditch portion) and operate for the remaining 75 years of a 100-year economic life. Between year one and year 25 the lowest cost system configuration for staging levels 1, 2, and 3 are assumed to be built and operating. Also, it should be noted that throughout this analysis a 20 percent surcharge was applied to the capital costs of all staging operations. This is done to penalize for the additional cost of a staged construction plan. If no

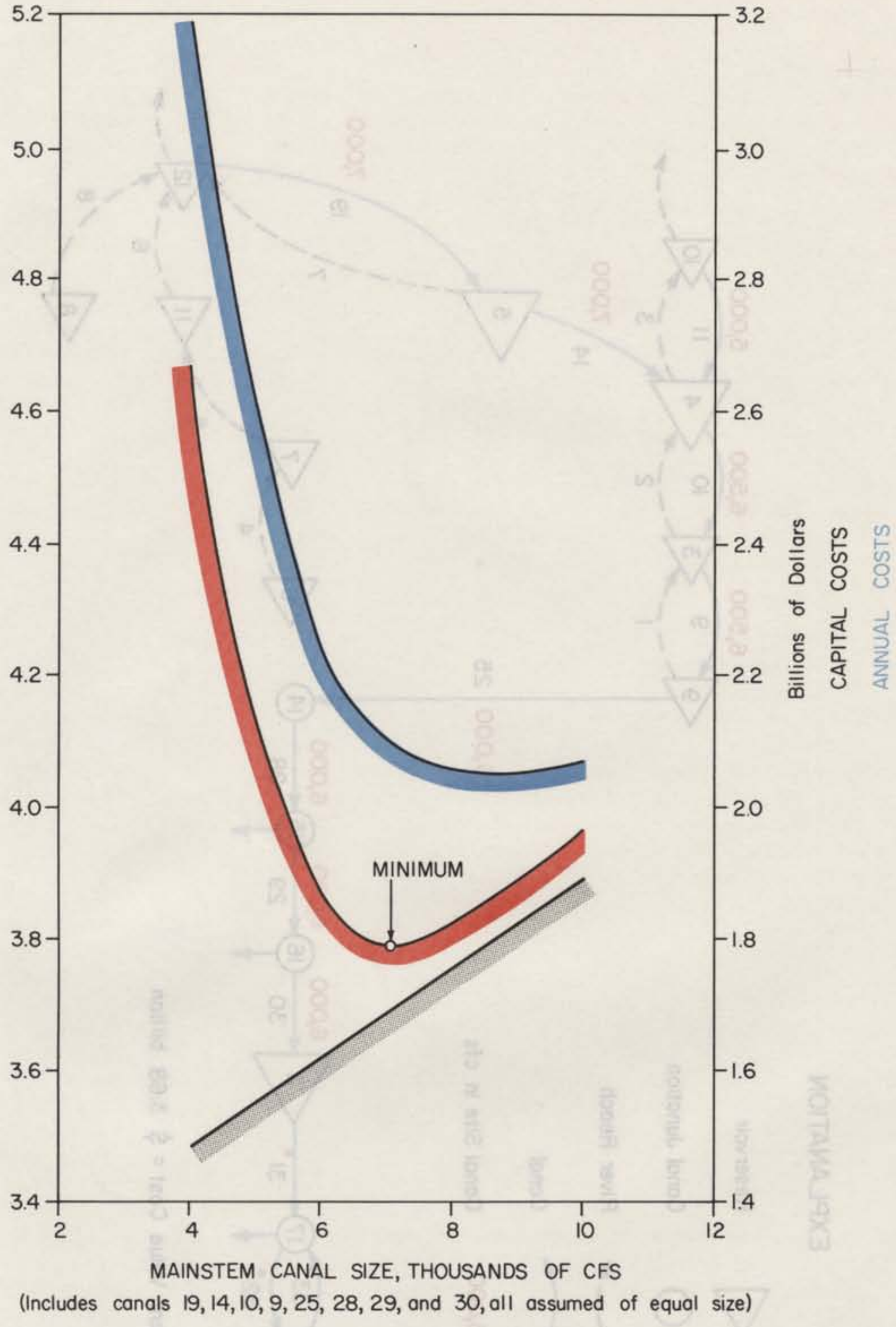



Figure 29

**Present Value Costs Versus Mainstem Canal Size, Analysis for 1990 Demand Level**

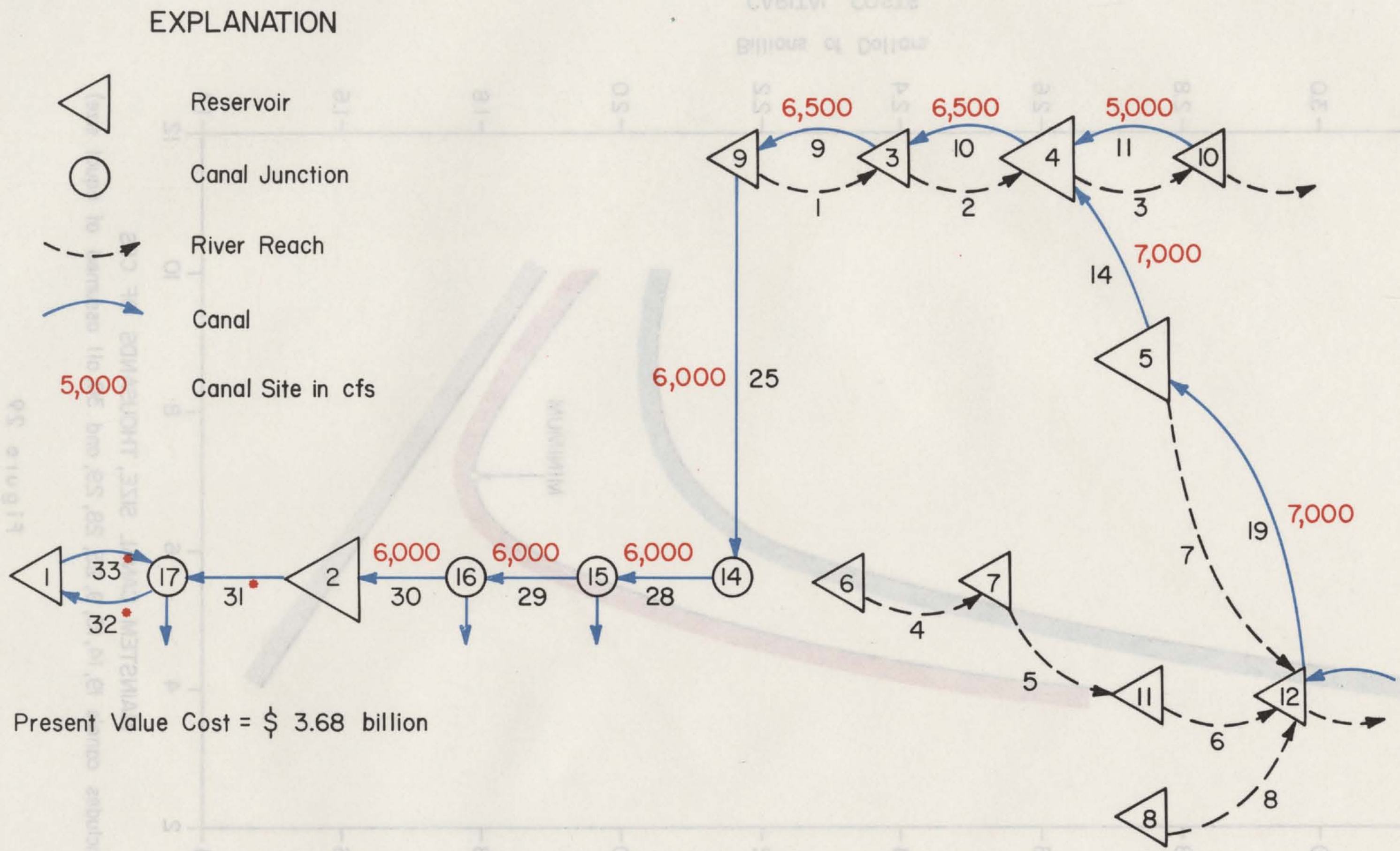



Figure 30

Best Canal Sizes, 1990 Demand Level

* Canals 31, 32, and 33 are sized at 50,000 cfs, as are river reaches.

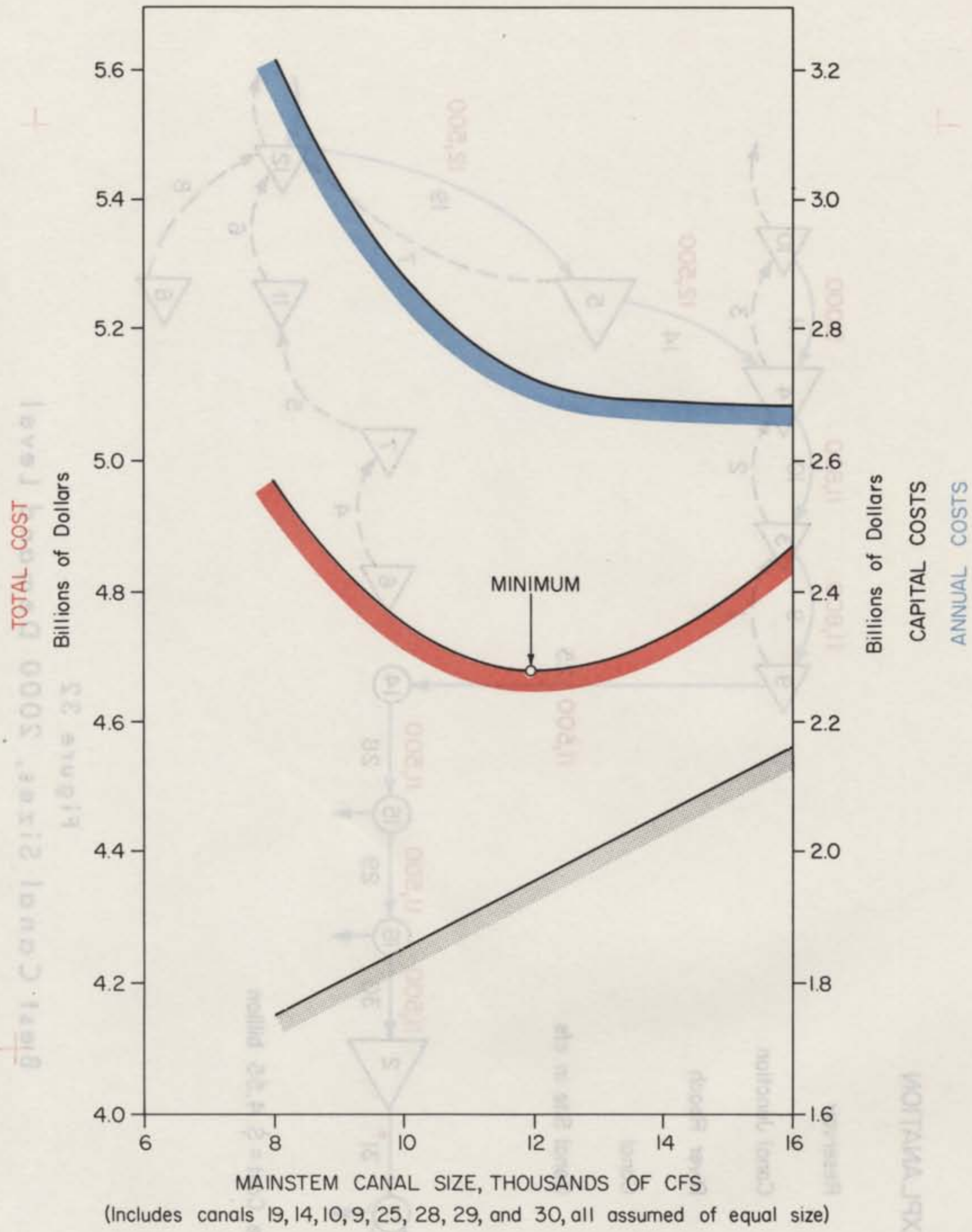



Figure 31

**Present Value Costs Versus Mainstem Canal Size, Analysis for 2000 Demand Level**

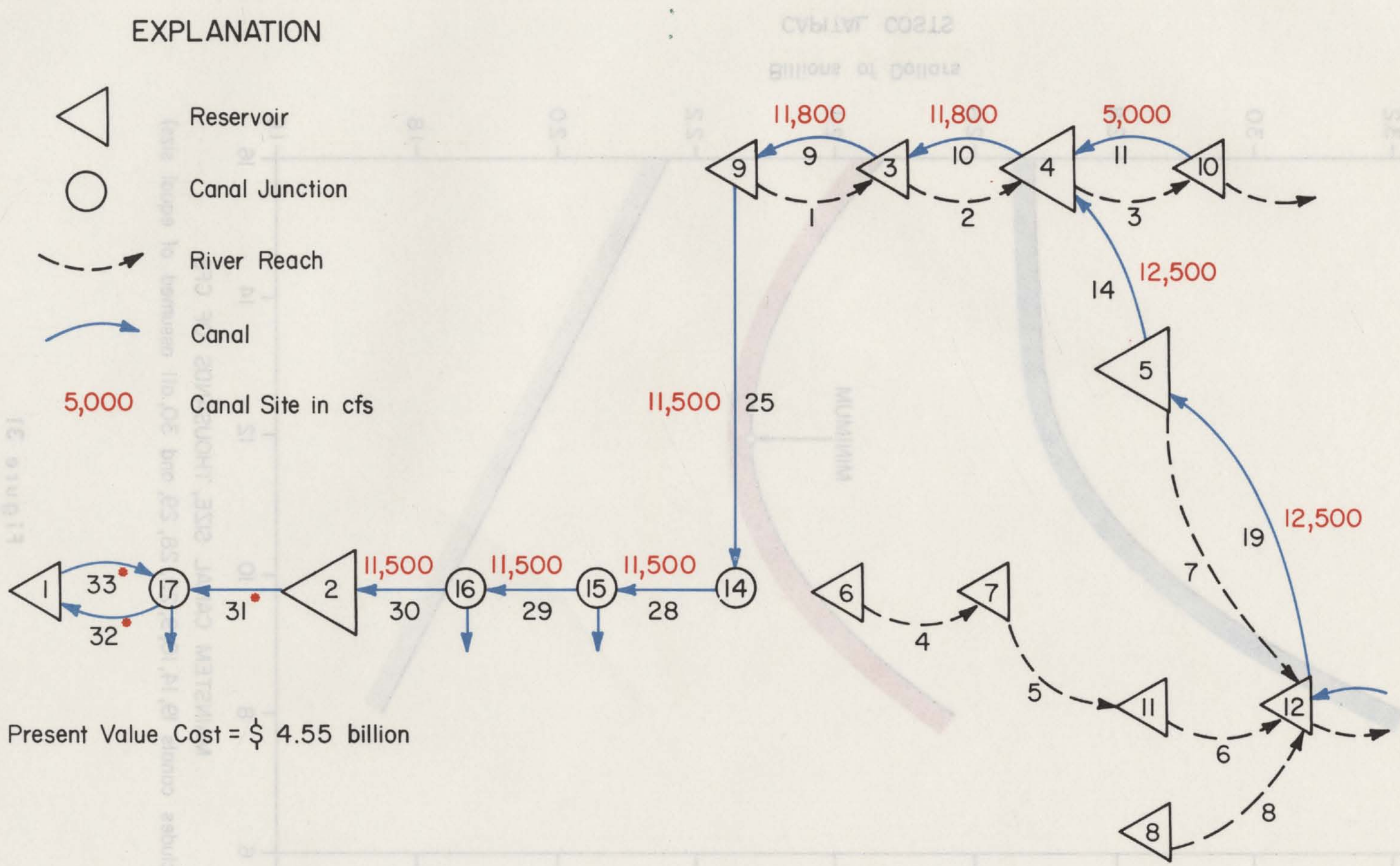



Figure 32

Best Canal Sizes, 2000 Demand Level

• Canals 31, 32, and 33 are sized at 50,000 cfs, as are river reaches.

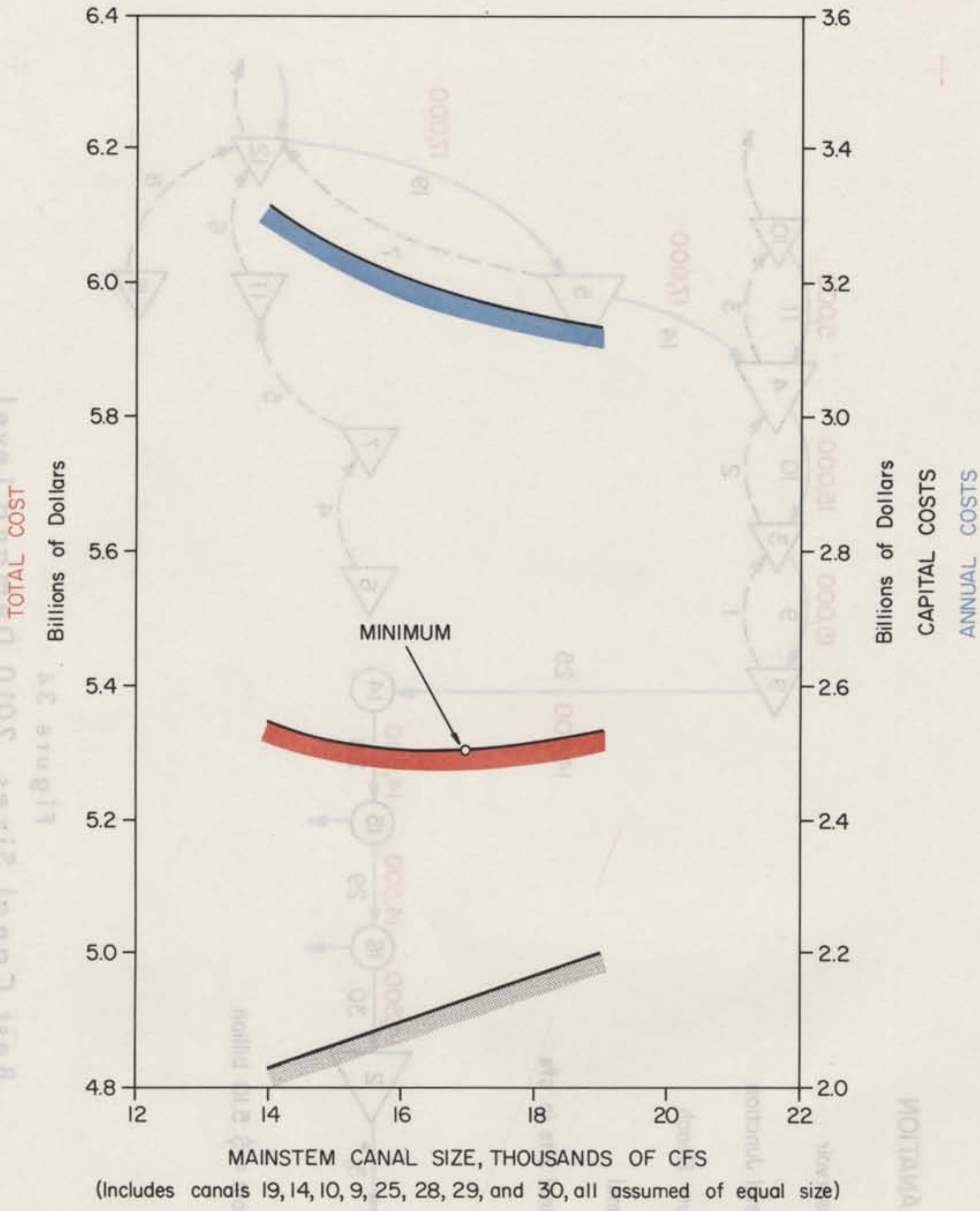



Figure 33

**Present Value Costs Versus Mainstem Canal Size, Analysis for 2010 Demand Level**

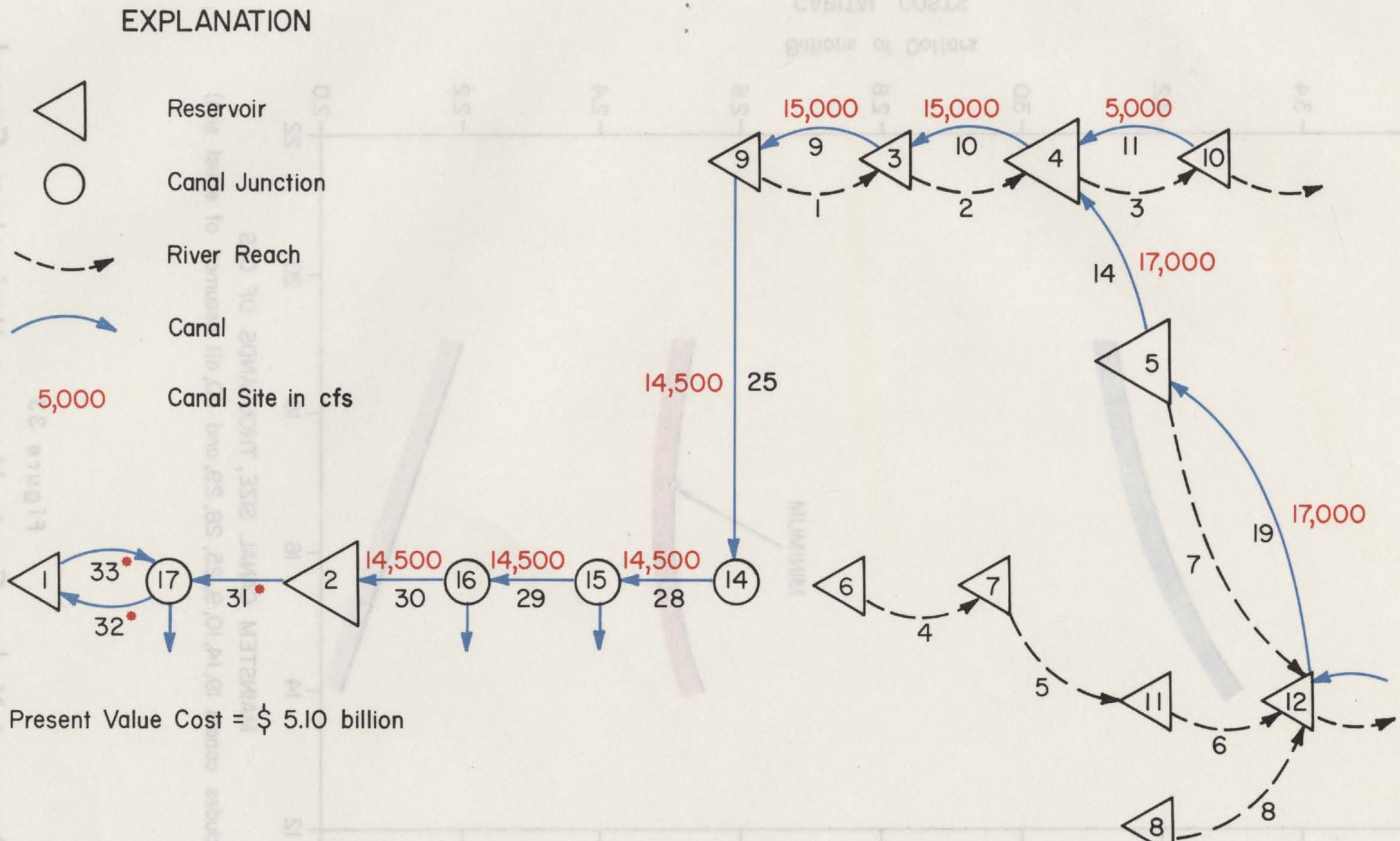



Figure 34

Best Canal Sizes, 2010 Demand Level

* Canals 31, 32, and 33 are sized at 50,000 cfs, as are river reaches.

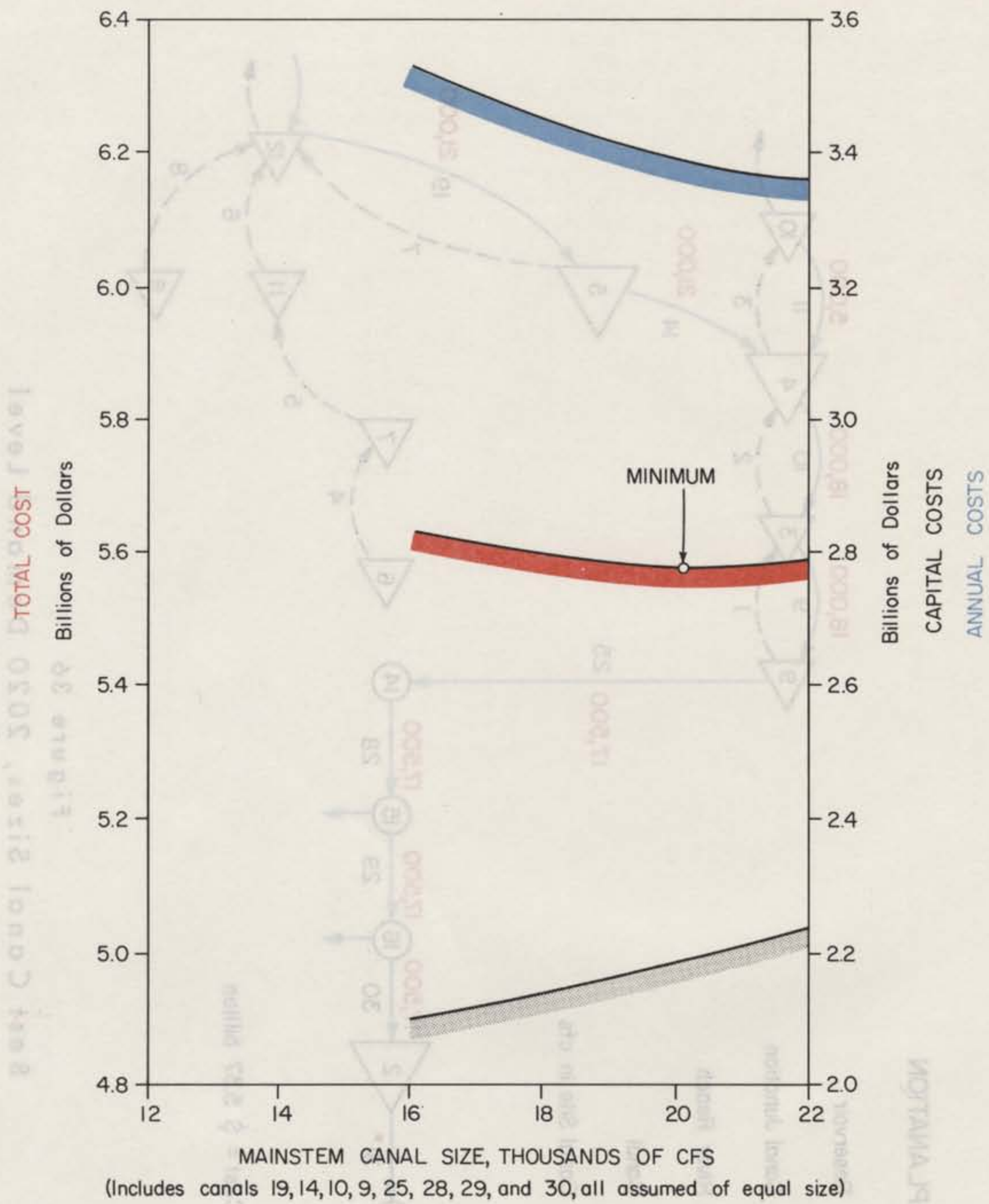



Figure 35

**Present Value Costs Versus Mainstem Canal Size, Analysis for 2020 Demand Level**

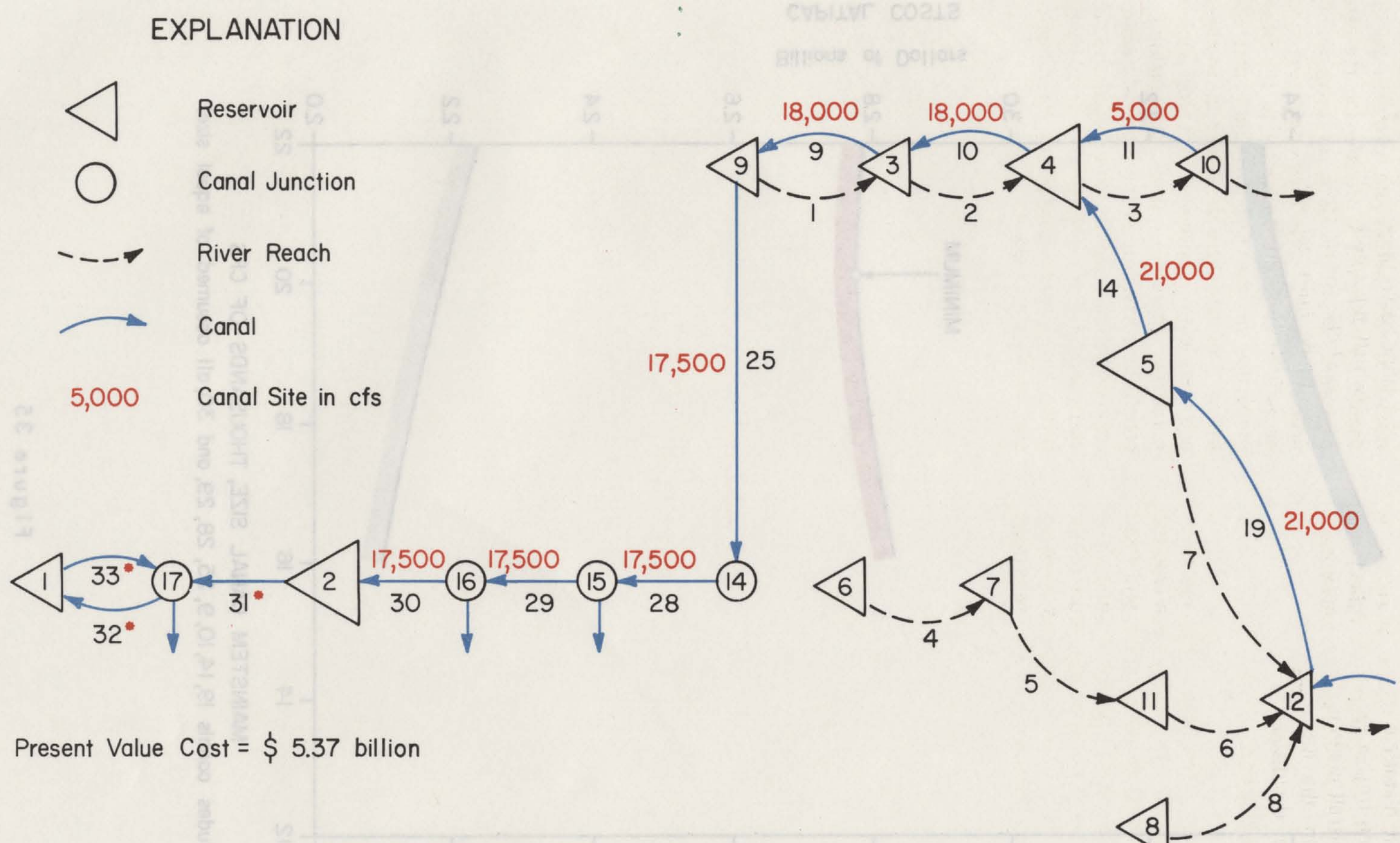



Figure 36

Best Canal Sizes, 2020 Demand Level

* Canals 31, 32, and 33 are sized at 50,000 cfs, as are river reaches.

staging penalty is applied, the cheapest implementation plan is to increase canal capacity exactly as it is needed (i.e., every year). This would hardly be practical, pump sizes that small would be unrealistic and inefficient. The method used seems to give a realistic approach to quantifying the cost of staging, and worked well in the example problem. The 20 percent figure, of course, is problem-specific and can be changed to reflect the proper cost of staging (it is a modeling input parameter).

## Retrospect

With the sizing of the canals and reservoirs completed for the four staging levels, this concludes the work which uses only the historical sequence and fixed demand levels upon which to size system facilities. The remaining steps in the overall planning procedure (Steps Five and Six) use both the historical and selected stochastic sequences as discussed in the remaining chapters.

Also, the remaining analyses use the demand-buildup data instead of the constant-level data used in this and previous chapters. Therefore, the present value information presented in the following chapters cannot be compared directly with that shown in this chapter. It is stressed, as was mentioned in Chapter I, that the results shown herein are problem-specific and care should be exercised in transferring the results to other problems. Also, the data set used for this example problem is an unverified data set and does not necessarily correspond directly to the exact hydrologic properties of the Cypress Creek and Sulphur River basins. The data were developed for model development and verification purposes, not for engineering study purposes; therefore, prior to making any direct conclusions based upon the material herein the data set upon which the results were based should be checked very carefully for errors. The data set did, however, prove to be quite satisfactory for model development and verification purposes.

# VI. STOCHASTIC SEQUENCE SELECTION

This chapter discusses a procedure for analyzing and selecting comparative stochastic sequences of runoff and demand data for use with the historical data in optimizing the implementation plan. The data sequence selection procedure presented is the first part of Step Five of the overall plan optimization procedure disucssed in Chapter II. It is also a necessary prelude to the second portion of Step Five, that of final optimization of an implementation plan, as discussed in Chapter VII.

Using stochastic sequences in addition to historical data, in connection with complex modeling techniques, can significantly improve the reliability of the answers derived; however, tremendous additional amounts of computation are required. To adequately span the population of possible ways that various future system inputs might occur, 20 to 30 randomly selected sequences are considered barely adequate. On the other hand, if the variation in system response caused by sequence differences is large, a much greater number of sequences (e.g., 50 to 100) may be required.

If the data generation technique has been well conceived and the number of sequences is adequate for statistical reliability, then a planner might feel justified in drawing conclusions concerning the effect of hydrology on the choice of planning alternatives. To do this, he would have only to

- generate the desired number of hydrologic sequences (e.g., 50 to 100),
- run each of the system configurations with these sequences through each of the appropriate simulation models,
- measure the responses, and
- make his assessments.

As straightforward as it may seem, this approach is not practical as it is too costly and could even be impossible from a computational standpoint.

Thus, any steps which can be taken to reduce the cost, while preserving the reliability of the model results and significance of the decisions based on model results, are highly desirable. This issue raises three questions that must be resolved to define a practical and cost-effective solution methodology. They are:

- What is the minimum number of stochastic sequences necessary to adequately evaluate the range of system performance?
- What is a good criterion for selecting those sequences required for analysis?
- How can the selected sequences be used efficiently in evaluating system performance?

The answers to these questions lie partly in the rigor of statistical analysis of samples of varying size. For example, a sample of 100 sequences is considered large and generally adequate for evaluation purposes. Conversely, a sample of 6 sequences is clearly too small to produce reliable estimates of expected system performance. Given that nothing more than the statistical laws of randomly selected samples is known, a sample of 30 might be considered barely adequate. But, even this large a sampling might be too costly for routine planning purposes.

Yet, if the planner has special knowledge of (1) the kind of stochastic process involved and (2) the physical system with which he is dealing, it should permit him to use a smaller sample. If it is possible to utilize this knowledge and exercise good judgment in preselecting samples that in the aggregate adequately describe the larger group, then it is possible to reduce costs substantially.

Suppose now that the stochastic data generation procedures described in Chapter III indicate the desirability of generating approximately 3,600 years of data. If, from the analysis of the problem being treated (as discussed in Chapter V) sequences 36 years long are desired, a total of 100 sequences or so should be generated to properly span the significant combinations of possible occurrences. Because using the entire set of 100 sequences as input to detailed simulation models is computationally prohibitive, selection of a representative few of these 100 sequences (in a manner to reduce small-sample bias) is necessary prior to their use with detailed simulation and optimization modeling procedures.

## Drought Characteristics

In order to select a small representative number of sequences from a large number, it is desirable to determine characteristics of the sequences that substantially influence the performance of the system. If there is more than one important independent characteristic, it is necessary to classify the sequences on the basis of each characteristic. As mentioned in Chapter III, two of the most important characteristics of a drought are the *magnitude of the most critical drought* within a sequence and the *duration of the drought*.

The magnitude of the drought relates directly to the demand imposed on the physical system. If there is no demand, then there really is no deficit. Hence the planner tends to relate his choices of droughts for study to the schedule of demands he contemplates. The severity of the most critical drought influences the ultimate system plan by regulating the size and number of facilities required to give optimal performance. In general, the more severe the most critical drought is, the larger or more numerous the required facilities must be to give adequate performance. Also, all droughts which require the development of new storage capacity to ensure uninterrupted deliveries or which require importation from other sources will be of particular interest to the planner.

Another important characteristic is the *location of the drought* within the sequence, if, over time, the staging of facilities to meet an increasing demand for water is to be analyzed. For example, if only observed historical data are used and the critical drought occurs at the beginning of the demand-buildup period, the drought may force premature construction of storage and transfer facilities necessary for the importation of water, shorten the construction period, and raise the present value of the capital costs for major system components. Placed near the end of the sequence, it may have the opposite effect, delaying the critical date at which importation must commence. However, by using equally likely stochastic sequences in the analysis procedure, with critical droughts occurring early in some sequences and late in others (each with varying severities), a more representative expected condition forms the basis on which to select an optimal implementation plan.

The severity of the drought expressed in terms of its duration likewise has a strong effect on the scheduling of system elements. Of special importance is the relation between drought duration and the physical capability of the system to meet demands over the drought period from accumulated storage. A deficit condition with a duration that is equal to or longer than the time required to fully drain the system from a full condition must be regarded in a different light than one of shorter duration. Although there is a direct relation between drought duration and magnitude, those droughts with long durations tend to be more severe on a given project than those of short duration, given a constant deficit amount.

## Devising Sequence Selection Categories Based on Drought Characteristics

Upon determining for a given problem whether a one, two, or perhaps even a three dimensional selection methodology is to be employed, the general procedure shown in Figure 37 is suggested. For the two-dimensional hydrologic case, both the number of selection categories and the scale quantities to be placed on the ordinate and the abscissa of the sequence selection chart must be determined. To do this (as indicated in Figure 37), all sequences are first arranged in order of increasing drought severity (system net flow).[27/] Secondly, they are arranged according to location of drought. A check is next made on the 100 sequences to see (1) how normally they are distributed along the ordinate (see Figure 40), and (2) how uniformly they are distributed along the abscissa.[28/] Conformance of the data to the appropriate distribution gives partial assurance that the 100 sequences do adequately represent the problem.

The number of categories for any particular problem should be just sufficient to adequately define the response of the system. That is, if (1) it is suspected that 10-year staging increments are to be used in planning and implementing the prototype, and (2) the demand-buildup period is expected to be approximately 30 years long, then three periods, each 10 years long, could be used to define the number of drought-location selection categories (the abscissa). On the other hand, an estimate of the number of sequences required to properly span the system performance due to drought (net flow) variability can be obtained from the preliminary simulation analysis that is designed to determine the number of sequences that should be generated. With this information, the number of net-flow categories (the ordinate) can be determined considering that with the previously defined 10-year staging increment, three sequences will be selected for each net-flow category defined. Therefore, if 12 sequences are desired, four levels of net-flow categories are required. Similarly, if nine sequences are desired, three levels of net-flow categories are required. Although the selection of the number of categories and their arrangement may seem somewhat arbitrary, it should be kept in mind that the purpose of this selection process is to obtain a sample from the total 100 sequences which in the aggregate

- uniformly spans the population of possible occurrences according to the underlying statistical distributions,
- possesses virtually the same range of characteristics as the total set of sequences, and
- gives an estimate of the system performance within the accuracy required by the user.

Therefore, by definition, each selection category should be designed so that an equal number of the original 100 sequences is contained within each category. If, for example, the two selection characteristics are each divided into three subsets as shown in Figure 38, there will be an array of nine

27/ "Drought severity", as used here, is equated with system net flow, and is the total system supply (total inflows to system storage) minus the total system demand averaged over various durations.

28/ Statistical inference indicates that a drought of a given duration should have an equally likely chance of occurring at any point in each sequence; whereas, the critical period magnitudes of all sequences should be normally distributed.

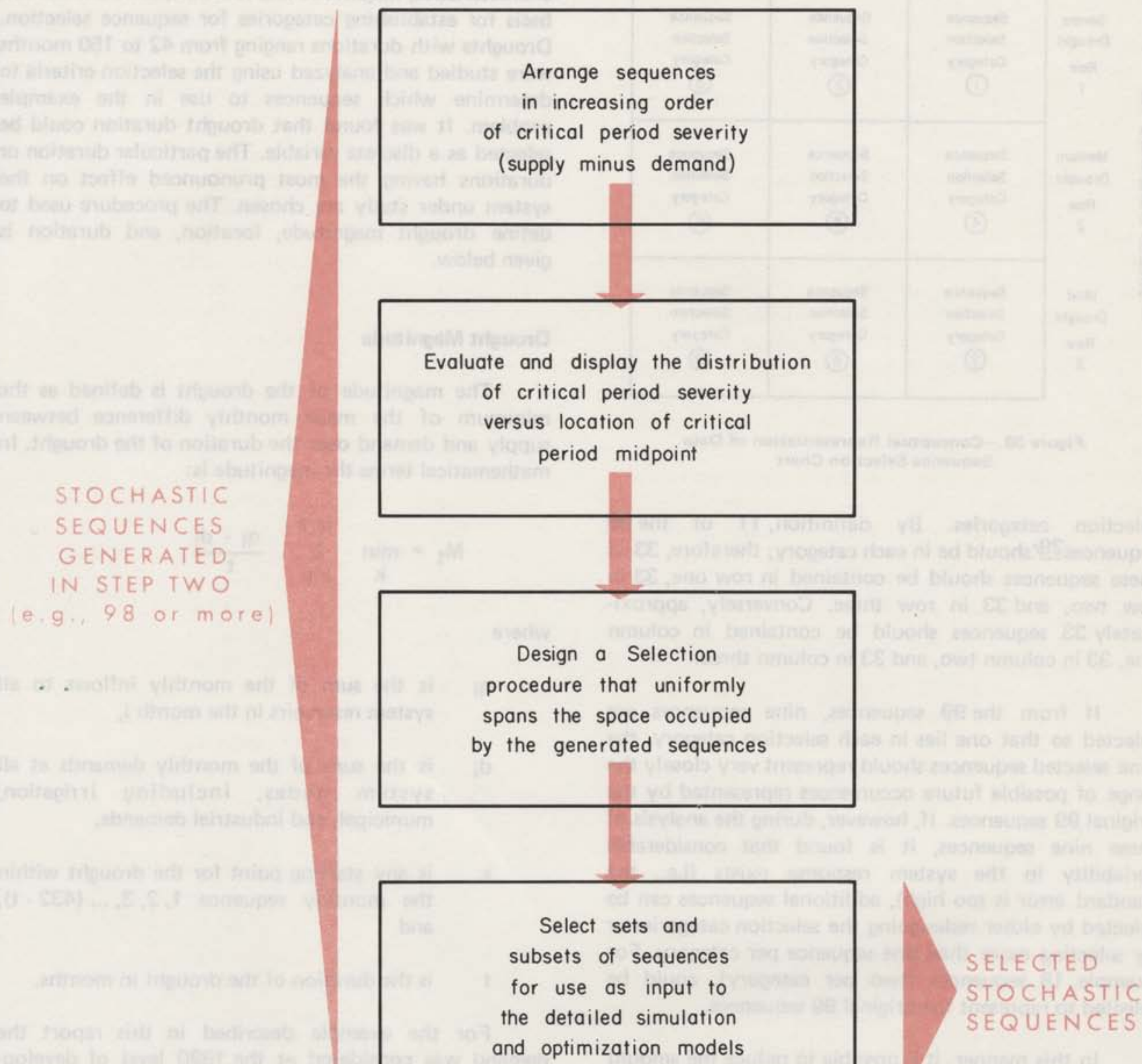



Figure 37

Sequence Selection Steps

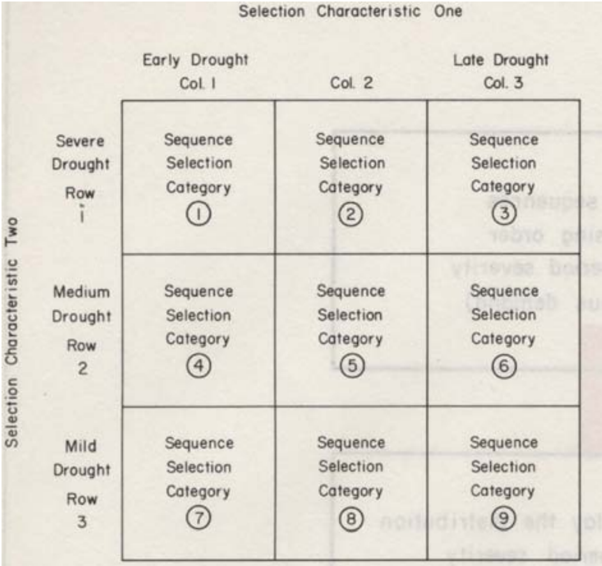



Figure 38.—Conceptual Representation of Data Sequence Selection Chart

selection categories. By definition, 11 of the 99 sequences[29] should be in each category; therefore, 33 of these sequences should be contained in row one, 33 in row two, and 33 in row three. Conversely, approximately 33 sequences should be contained in column one, 33 in column two, and 33 in column three.

If from the 99 sequences, nine sequences are selected so that one lies in each selection category, the nine selected sequences should represent very closely the range of possible future occurrences represented by the original 99 sequences. If, however, during the analysis of these nine sequences, it is found that considerable variability in the system response exists (i.e., the standard error is too high), additional sequences can be selected by either redesigning the selection categories or by selecting more than one sequence per category. For example, 18 sequences (two per category), could be selected to represent the original 99 sequences.

In this manner, it is possible to reduce the amount of required computation in a highly complex problem by an order of magnitude or more while at the same time maintaining the reliability of the results very close to that which can be obtained by employing all 100 sequences in the analysis process.[30]

[29] In this case, 99 sequences are used to permit an even division of the sequences per selection category. One of these 99 sequences is the historical sequence and the remaining 98 are stochastically generated.

[30] Additional reductions in computational requirement can be realized by using the procedures described in the section "Computation Reduction Procedures" in Chapter VII.

## Sequence Selection Criteria

In this investigation 99 sequences were generated, each having a duration of 36 years. Two drought characteristics, *magnitude* and *location*, were chosen as a basis for establishing categories for sequence selection. Droughts with durations ranging from 42 to 150 months were studied and analyzed using the selection criteria to determine which sequences to use in the example problem. It was found that drought duration could be selected as a discrete variable. The particular duration or durations having the most pronounced effect on the system under study are chosen. The procedure used to define drought magnitude, location, and duration is given below.

### Drought Magnitude

The magnitude of the drought is defined as the minimum of the mean monthly difference between supply and demand over the duration of the drought. In mathematical terms the magnitude is:

$$M_t = \min_k \sum_{i=k}^{k+t} \frac{q_i - d_i}{t}$$

where

$q_i$ is the sum of the monthly inflows to all system reservoirs in the month i,

$d_i$ is the sum of the monthly demands at all system nodes, including irrigation, municipal, and industrial demands,

k is any starting point for the drought within the monthly sequence 1, 2, 3, ... (432 - t), and

t is the duration of the drought in months.

For the example described in this report the demand was considered at the 1990 level of development. The annual and seasonal variations in system supply, demand, and net flow (supply minus demand) for this level of development are depicted for the historical sequence in Figure 39.

### Drought Location

The location of the drought within a sequence is defined by the position of its midpoint. For a drought with duration t and a starting time k, the location is

$$P_t = k + \frac{t}{2}$$

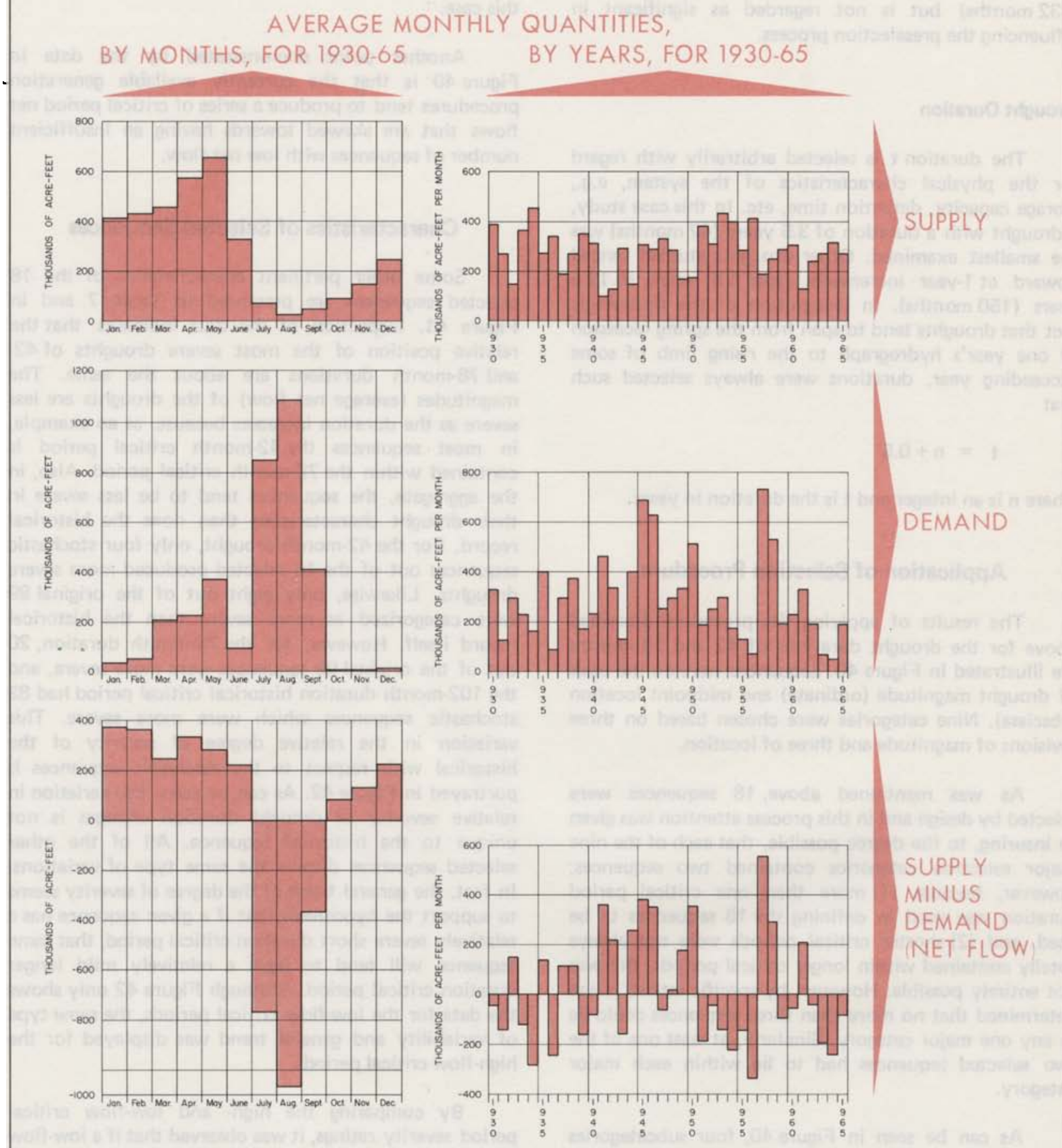



Figure 39
Example of System Demand-Supply Relationships,
Based on Demand at 1990 Staging Level and the
Historical Data Sequence

For a given duration, t, the drought location can be no closer to the beginning or end of the sequence than t/2. This precludes a statistically uniform distribution of drought location over the full span of the sequences (432 months) but is not regarded as significant in influencing the preselection process.

**Drought Duration**

The duration t is selected arbitrarily with regard for the physical characteristics of the system, e.g., storage capacity, detention time, etc. In this case study, a drought with a duration of 3.5 years (42 months) was the smallest examined. Other droughts studied ranged upward at 1-year increments from 3.5 years to 12.5 years (150 months). In recognition of the hydrologic fact that droughts tend to span from the spring recession of one year's hydrograph to the rising limb of some succeeding year, durations were always selected such that

$$t = n + 0.5$$

where n is an integer and t is the duration in years.

## Application of Selection Procedure

The results of applying the procedure described above for the drought durations of 42 and 78 months are illustrated in Figure 40. Sequences were on the basis of drought magnitude (ordinate) and midpoint location (abscissa). Nine categories were chosen based on three divisions of magnitude and three of location.

As was mentioned above, 18 sequences were selected by design and in this process attention was given to insuring, to the degree possible, that each of the nine major selection categories contained two sequences; however, because (1) more than one critical period duration was used in defining the 18 sequences to be used, and (2) shorter critical periods were not always totally contained within longer critical periods, this was not entirely possible. However, by specific intent, it was determined that no more than three sequences could lie in any one major category. Similarly, at least one of the two selected sequences had to lie within each major category.

As can be seen in Figure 40, four subcategories were defined in each major category. The purpose of this was to strive to insure that when selecting 18 sequences no two sequences would lie in any given subcategory. For the 78-month duration selection chart this was accomplished in all cases except for the "row three—column three" category.

Selection charts of the other critical periods analyzed had placement successes similar to the ones shown herein; however, by design, sequences were selected on the basic assumption that shorter critical periods should be contained within longer critical periods. It was desirable to make this assumption in order to apply a two-dimensional selection strategy to this case.

Another point demonstrated by the data in Figure 40 is that the currently available generation procedures tend to produce a series of critical period net flows that are skewed towards having an insufficient number of sequences with low net flow.

## Characteristics of Selected Sequences

Some other pertinent characteristics of the 18 selected sequences are presented in Table 7 and in Figure 41. Inspection of the data indicates that the relative position of the most severe droughts of 42- and 78-month durations are about the same. The magnitudes (average net flow) of the droughts are less severe as the duration increases because, as an example, in most sequences the 42-month critical period is contained within the 78-month critical period. Also, in the aggregate, the sequences tend to be less severe in their drought characteristics than does the historical record. For the 42-month drought, only four stochastic sequences out of the 18 selected produced more severe droughts. Likewise, only eight out of the original 99 were categorized as more severe than the historical record itself. However, for the 78-month duration, 20 out of the original 99 sequences were more severe, and the 102-month duration historical critical period had 83 stochastic sequences which were more severe. This variation in the relative degree of severity of the historical with respect to the stochastic sequences is portrayed in Figure 42. As can be seen, this variation in relative severity as drought duration changes is not unique to the historical sequence. All of the other selected sequences display the same type of variations. In fact, the general trend of the degree of severity seems to support the hypothesis that if a given sequence has a relatively severe short duration critical period, that same sequence will tend to have a relatively mild longer duration critical period. Although Figure 42 only shows the data for the low-flow critical periods, the same type of variability and general trend was displayed for the high-flow critical periods.

By comparing the high- and low-flow critical period severity ratings, it was observed that if a low-flow critical period drought of a given duration is quite severe with respect to the other stochastic sequences, the high-flow critical period is also quite severe from a flooding viewpoint.

From these observations it seems as though the generation process, designed to preserve the mean and variability terms of the data sets, tends to compensate for generating a severe low-flow drought by subsequently generating a relatively severe high-flow

EXPLANATION

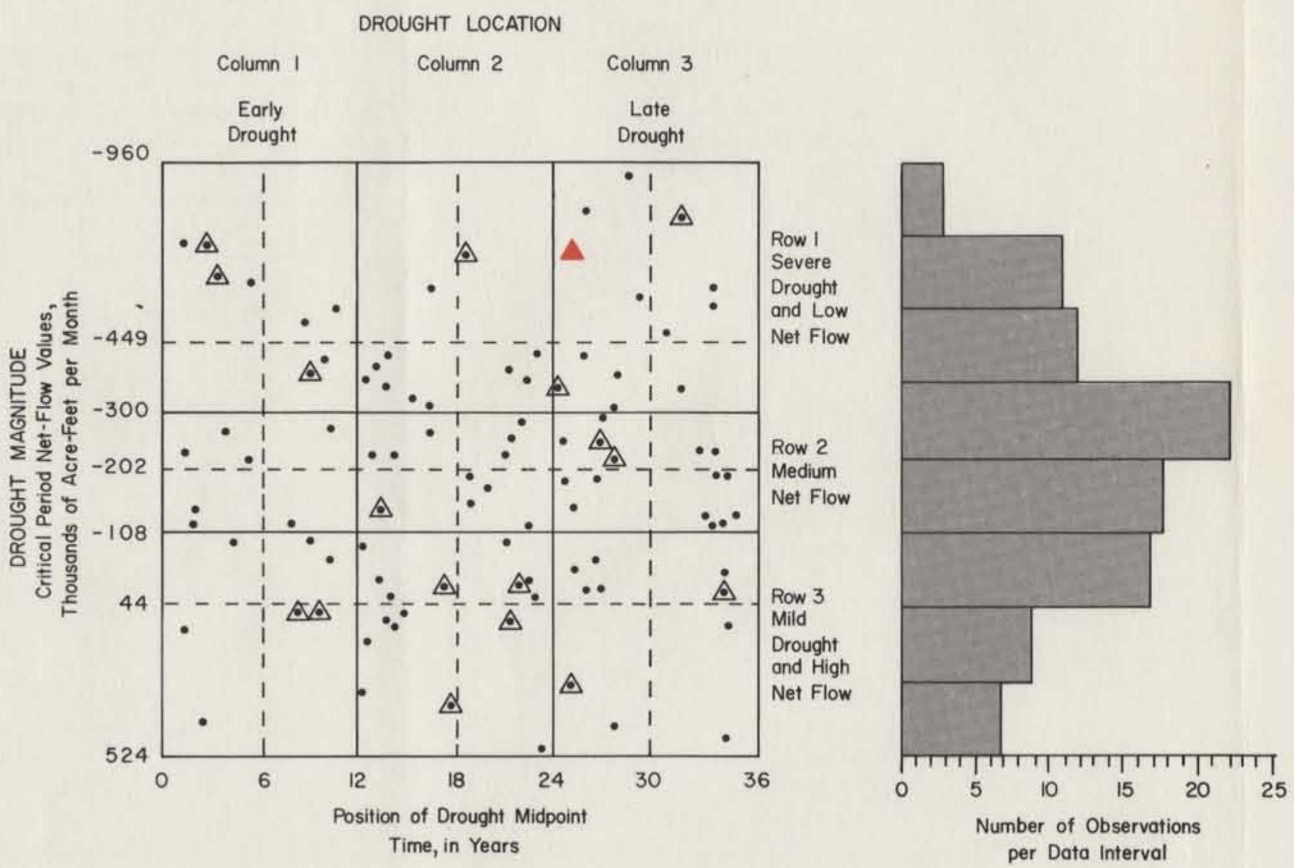
One of the 99 Data Sequences

The Historical Sequence

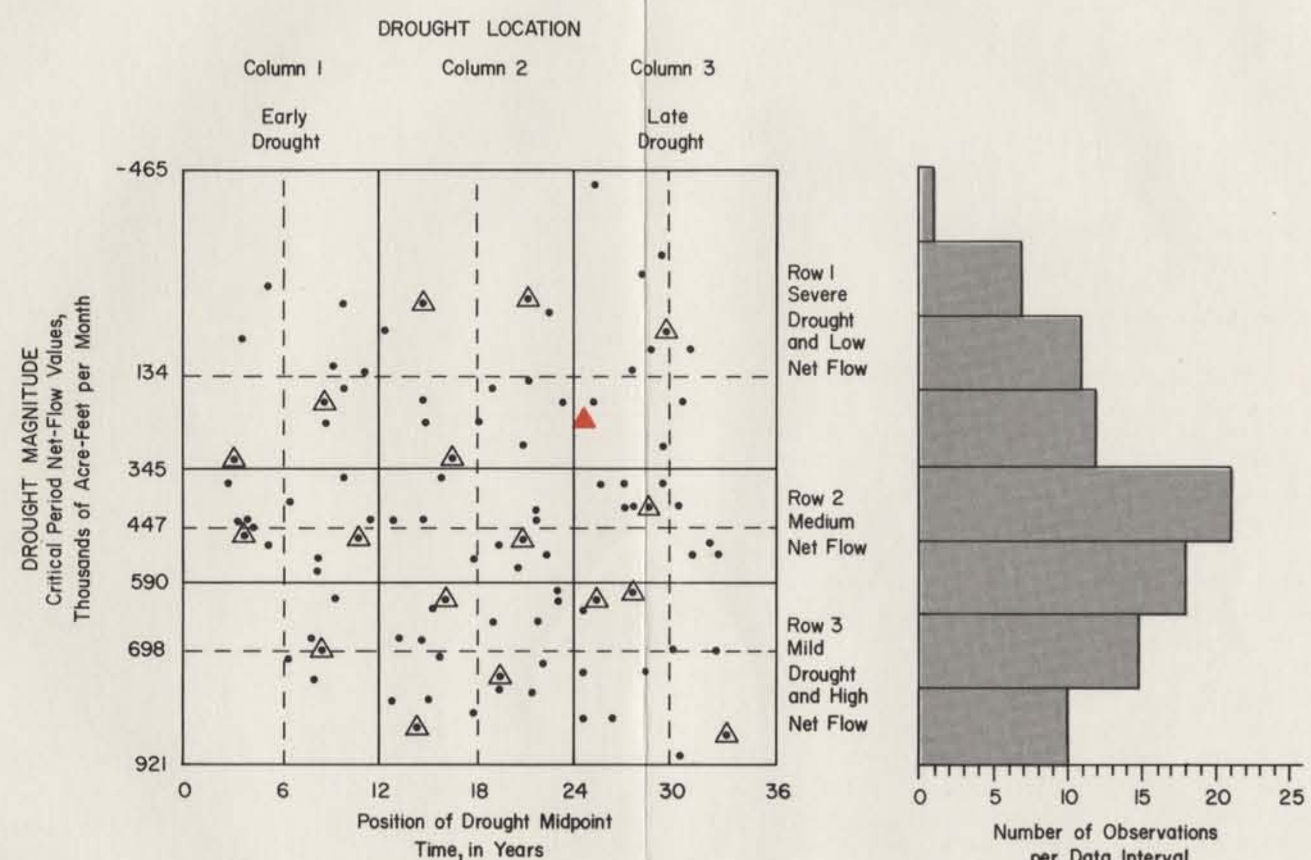
One of the Selected 18 Sequences

Line Dividing Selection Categories

Line Defining Selection Subcategories




42-Month Duration Categorization and Selection Chart




78-Month Duration Categorization and Selection Chart

Figure 40

Data Sequence Categorization and Selection Charts

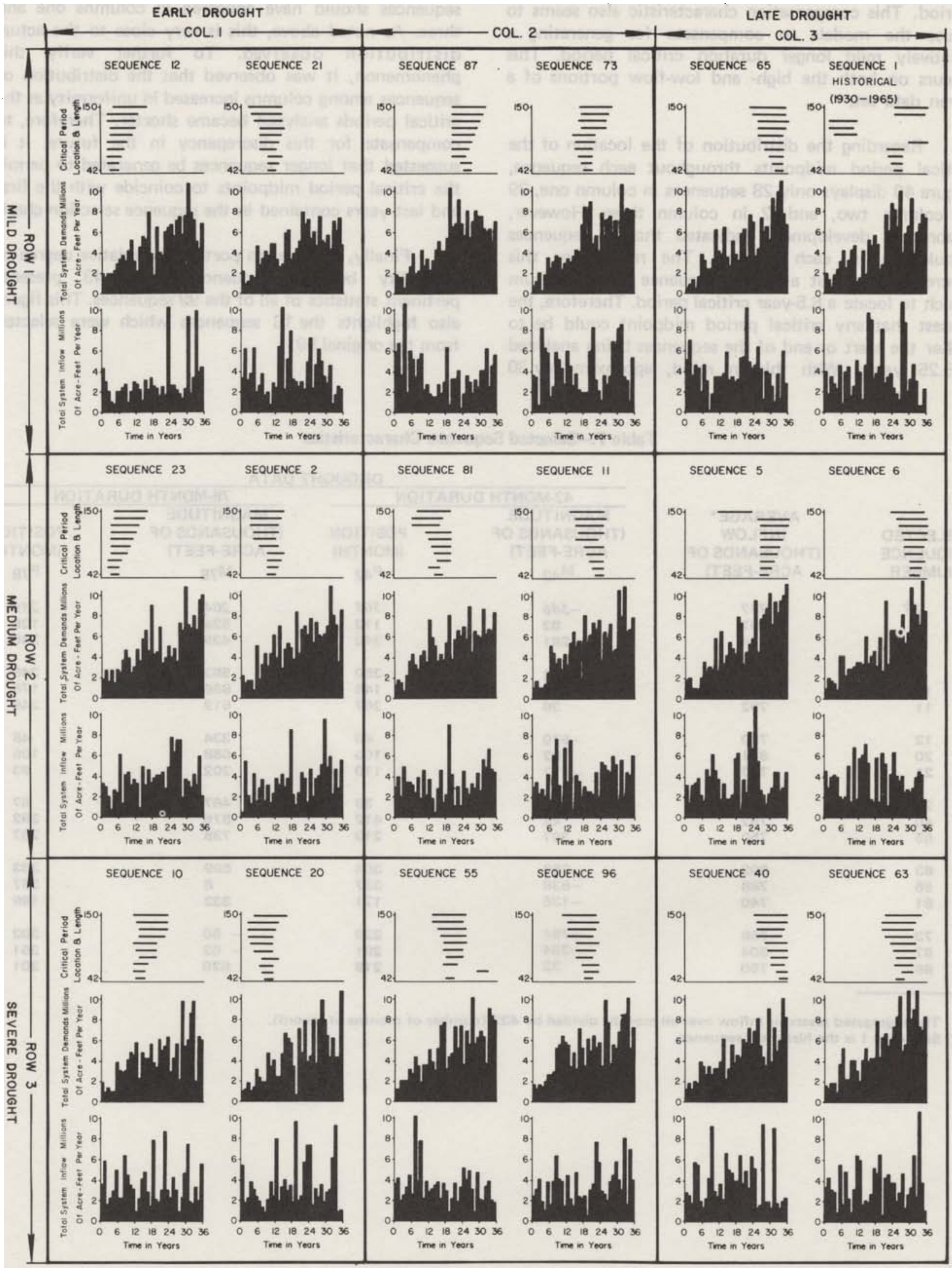



**Figure 41**

**Variability and Location of Droughts in 18 Selected Demand and Supply Sequences**

period. This compensation characteristic also seems to cause the models to compensate for generating a relatively mild longer duration critical period. This occurs on both the high- and low-flow portions of a given data set.

Regarding the distribution of the location of the critical period midpoints throughout each sequence, Figure 40 displays only 28 sequences in column one, 39 in column two, and 32 in column three. However, theoretical development indicated that 33 sequences should be in each column. The reason for this discrepancy is that a 36-year sequence was used from which to locate a 6.5-year critical period. Therefore, the closest that any critical period midpoint could be to either the start or end of the sequences being analyzed is 3.25 years. With this in mind, approximately 30 sequences should have appeared in columns one and three. As noted above, this is very close to the actual distribution observed. To further verify this phenomenon, it was observed that the distribution of sequences among columns increased in uniformity as the critical periods analyzed became shorter. Therefore, to compensate for this discrepancy in the future, it is suggested that longer sequences be generated to permit the critical period midpoints to coincide with the first and last years contained in the sequence selection chart.

Finally, in order to portray the relative degree of variability between sequences, Figure 43 presents pertinent statistics of all of the 99 sequences. This figure also highlights the 18 sequences which were selected from the original 99.

**Table 7.—Selected Sequence Characteristics**

| | | DROUGHT DATA | | | |
|---|---|---|---|---|---|
| | | 42-MONTH DURATION | | 78-MONTH DURATION | |
| SELECTED SEQUENCE NUMBER | AVERAGE* INFLOW (THOUSANDS OF ACRE-FEET) | MAGNITUDE (THOUSANDS OF ACRE-FEET) $M_{42}$ | POSITION (MONTH) $P_{42}$ | MAGNITUDE (THOUSANDS OF ACRE-FEET) $M_{78}$ | POSITION (MONTH) $P_{78}$ |
| *1*† | *787* | *−586* | *303* | *204* | *289* |
| 2 | 787 | 82 | 112 | 524 | 128 |
| 5 | 784 | −261 | 340 | 429 | 345 |
| 6 | 801 | −202 | 350 | 593 | 346 |
| 10 | 788 | 243 | 148 | 866 | 176 |
| 11 | 792 | 39 | 267 | 512 | 249 |
| 12 | 750 | −670 | 40 | 334 | 45 |
| 20 | 821 | 82 | 100 | 688 | 105 |
| 21 | 783 | −377 | 110 | 202 | 93 |
| 23 | 768 | −592 | 39 | 487 | 57 |
| 40 | 792 | 44 | 412 | 875 | 392 |
| 55 | 750 | 307 | 219 | 738 | 237 |
| 63 | 802 | 258 | 304 | 599 | 333 |
| 65 | 788 | −838 | 337 | 8 | 367 |
| 81 | 740 | −136 | 171 | 332 | 189 |
| 73 | 766 | −784 | 220 | − 50 | 202 |
| 87 | 804 | −334 | 291 | − 62 | 261 |
| 96 | 750 | 33 | 219 | 620 | 201 |

* The aggregated reservoir inflow over all months divided by 432 (number of months of record).
† Sequence 1 is the historical sequence.

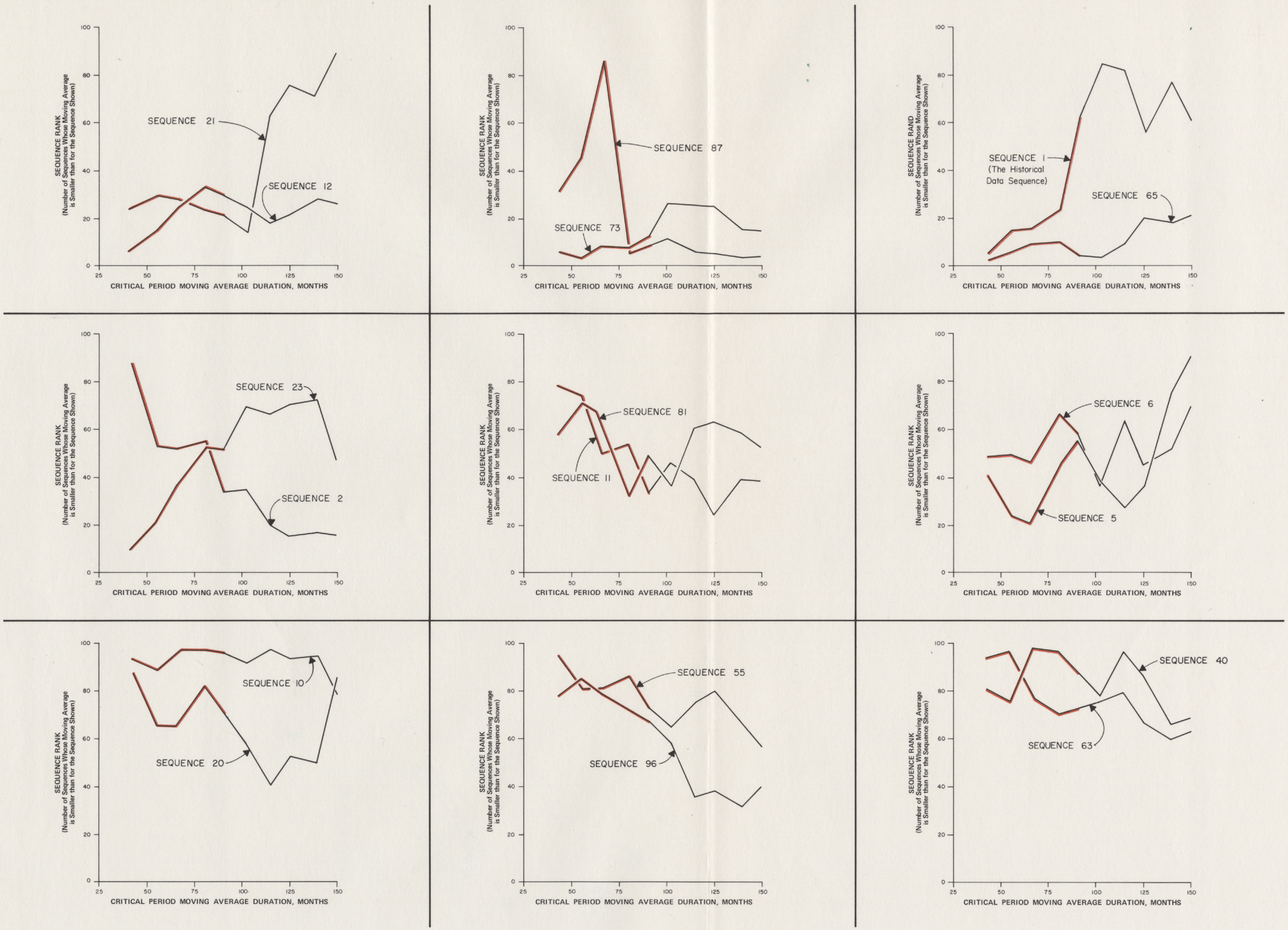



Figure 42
Rank of Low-Flow Critical Period Moving Averages Versus Length of Critical Period

*NOTE: Red shading denotes the range of moving average durations (durations of 42, 54, 66, 78, and 90 months used to select the 18 representative sequences.*

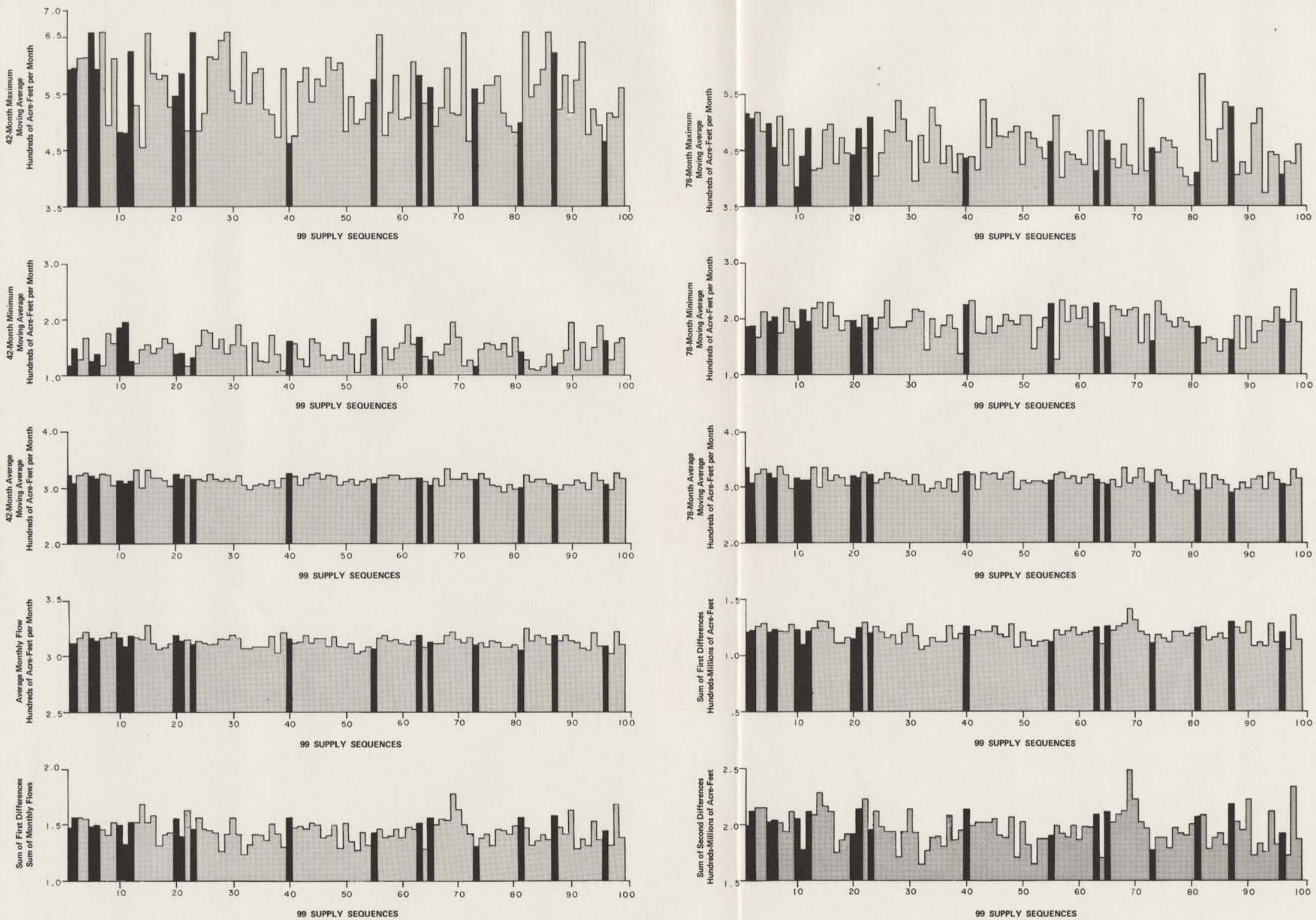



Figure 43
Selected Characteristics of the 99 Sequences of Total System Supply

Note: The black columns are the 18 selected sequences; sequence 1 is the historical sequence.

# VII. PLAN OPTIMIZATION

This chapter describes the procedure comprising the second portion of Step Five in the overall plan optimization procedure discussed in Chapter II. The procedure is designed to help the planner find the minimum-expected-cost implementation plan defined as the optimal or most reasonable implementation plan.

The procedure uses SIM-III in conjunction with both the historical sequence and other stochastic data sequence sets as selected in the first portion of Step Five and as described in Chapter VI. The procedure is described by discussing the results of applying it to the example problem. Variations in the procedure which appear to be useful in improving the computational efficiency are also discussed.

## Basic Approach

It is assumed that the work described in Chapters IV, V, and VI outlining plan development, plan improvement, and sequence selection has been completed. Thus, a set of good basic plans at each prespecified point on the demand-buildup curve has been developed. Also, a set of stochastic sequences has been selected to support plan optimization.

The importance of good initial plans cannot be overemphasized. Once an initial staging plan has been selected, the process used in improving it so as to further minimize expected costs is essentially one of direct search of a response surface. As with all search procedures which iteratively move from a starting point toward an optimum in a finite number of steps, the efficiency of the technique is directly related to the goodness of the initial starting point.

The starting point used is comprised of "all-canal-sizes-equal" solutions of the mainstem (canals 19, 14, 10, 9, 25, 28, 29, and 30) for each of the staging increments analyzed in Step Four. The canal sizes used along with their assumed installation times are shown in Table 8. Reservoir 1 was the only storage facility staged, and its desired capacity expansion is also shown in Table 8. The canal sizes shown in Table 8 were obtained from Figures 29, 31, 33, and 35 contained in Chapter V. The duration of the first three staging levels is prespecified at 10 years. The fourth and last level has a duration of 70 years which is equal to the remaining years of an assumed 100-year economic life.

**Table 8.—Staging Plan Sizes**

| STAGING LEVEL | (YEAR) DEMAND POINT | STAGING PERIOD | STAGING INCREMENT LENGTH (YEARS) | MAINSTEM* CANAL SIZE (CFS) | RESERVOIR 1 SIZE (MILLIONS OF ACRE-FEET) |
|---|---|---|---|---|---|
| 1 | 1990 | 1985-1994 | 10 | 7,000 | 1.4 |
| | | *1985-1999*† | *15* | *6,000* | *1.4* |
| 2 | 2000 | 1995-2004 | 10 | 12,000 | 2.0 |
| | | *2000-2009* | *10* | *11,000* | *2.0* |
| 3 | 2010 | 2005-2014 | 10 | 17,000 | 2.6 |
| | | *2010-2019* | *10* | *16,000* | *2.6* |
| 4 | 2020 | 2015-2084 | 70 | 20,000 | 3.0 |
| | | *2020-2084* | *65* | *19,000* | *3.0* |

* Canals 19, 14, 10, 9, 25, 28, 29, and 30. Canal 11 was sized at 5,000 cfs for all staging levels. Canals 31, 32, and 33 were sized at 50,000 cfs but priced at 10,000 cfs.

† Numbers in italic correspond to the minimum expected cost "all-canal-sizes-equal" solution of Figure 46. Numbers in letter gothic correspond to the initial plan derived in Chapter V and displayed in Figures 29, 31, 33, and 35.

## A Staging Plan to Minimize Expected Costs

The staging plan shown in Table 8 and graphically portrayed in Figure 44 is termed the central case and is optimized in this step of the procedure. All computations performed are related to the central case through the notation shown in Figure 44. This convention describes the perturbation of the central case in the search for the minimum-expected-cost implementation program. Two parameters, representing changes in canal capacity (maximum rate of flow) and time at which canal capacity is increased, describe this movement. These parameters are designated by $\Delta C$ and $\Delta T$, respectively. The change in the capacity of mainstem canals is represented by $\Delta C$ and is applied to the capacities at all four levels of the staging program. The shift in time of the occurrences of staging is defined by $\Delta T$. For example, a $\Delta C$ of +1,000 cfs and a $\Delta T$ of +1 year defines a program that would expand capacities from 8,000 cfs in 1986 to 13,000, 18,000, and 21,000 cfs in 1996, 2006, and 2016, respectively.

Improved staging programs were found by selecting values for $\Delta C$ and $\Delta T$ and subjecting them to the 18 selected demand and reservoir inflow sequences. SIM-III in conjunction with all 18 of the selected data sequences is used to evaluate the economic response of each implementation plan. The present value expected cost for this program is computed as the mean of the 18 values. Based on this result, the staging program is modified to drive the expected value to a smaller amount.[31]

The method used for finding a minimum-expected-cost plan is a gradient search procedure (Wilde and Beightler, 1967). In this procedure, discrete steps are made along the two-dimensional response surface defined by canal capacity and time at which staging occurs. Two step sizes were used as shown in Figures 45 and 46. First of all, 2,000 cfs $\Delta C$ and 10 year $\Delta T$ step sizes were used. Using this step size, a 10 percent lower expected cost plan was found—$4.36 billion versus $4.81 billion. Upon defining an area where the minimum is expected to lie, the step size was reduced to 1,000 cfs $\Delta C$ and 5 year $\Delta T$ to permit closer inspection of the region near the minimum. Upon exploring this region with the second set of step sizes, no further step size reductions were made. This is because an improvement of less than 0.5 percent was realized with the second set of step sizes.

Figure 45 illustrates the results of the perturbations leading to the location of a minimum-expected-cost plan in this example. It shows that the present value expected cost was reduced from $4.81 billion for the central case ($\Delta C$=0, $\Delta T$=0) to $4.34 billion at the minimum ($\Delta C$=−1,000, $\Delta T$=10). The actual canal and reservoir sizes corresponding to the solution of lowest expected cost shown in Figure 45 are represented by the italic numbers in Table 8 and are further displayed in Figure 46. Figure 46 also shows the order in which the canal size and staging perturbations were made in identifying the low-cost $4.34 billion staging plan. While this staging plan is the one of lowest expected cost found to this point in the analysis, it does not represent the minimum-expected-cost staging plan. Experience in analyzing prespecified points on the demand-buildup curves, in the manner described in Chapter V, indicates that an "all-canal-sizes-equal" solution is not the lowest cost solution possible for each of the 1990, 2000, 2010, and 2020 plan improvement solutions. By comparing Figures 29 through 36 contained in Chapter V and summarized in Table 9, a present value cost reduction of approximately 5 percent of the $4.34 billion can be saved by not using an "all-canal-sizes-equal" staging plan for canals 19, 4, 10, 9, 25, 28, 29, and 30. The columns in Table 9 titled Stochastic Data—Solution 1 represent those canal sizes for the "all-canal-sizes-equal" $4.34 billion staging plan; whereas, the columns titled Stochastic Data—Solution 2 represent the minimum-expected-cost staging plan of $4.15 billion.

It is recognized that further perturbation of the individual canal sizes and staging times either parallel to the coordinate axes or at some angle to them could find slightly lower costs, especially if smaller search step sizes are used; however, the additional reduction in the expected cost response will, in all likelihood, be less than another 0.5 percent. Therefore, no additional search is considered to be warranted, and the solution shown in Table 9 is considered to be the optimal (most reasonable) implementation plan.

## Experience Gained

It is interesting to note that, while each step in the plan development, plan improvement, and plan optimization process did not individually yield drastic reductions in the cost response of the system, collectively they reduced the present value cost of the implementation plan about 35 percent. Therefore, with this in mind, the procedure used to find the minimum-expected-cost implementation plan, while not mathematically rigorous, appears to be reasonable, meets with the test of practicality, and demonstrates planning concepts in sufficient detail to guide others in their selection of approaches for analyzing and solving related problems.

Separate analysis not discussed herein showed that a system other than the single mainstem of canals might be attractive from a flexibility standpoint. A more complicated canal network would offer more flexibility

[31] The cost of computation ranges from $0.15 to $0.25 per year of simulation for the example problem; therefore, the cost to compute an expected value using 18 stochastic sequences, each 36 years long, ranges from $97.20 to $162.00, depending on the speed of the computer used and its cost of operation.

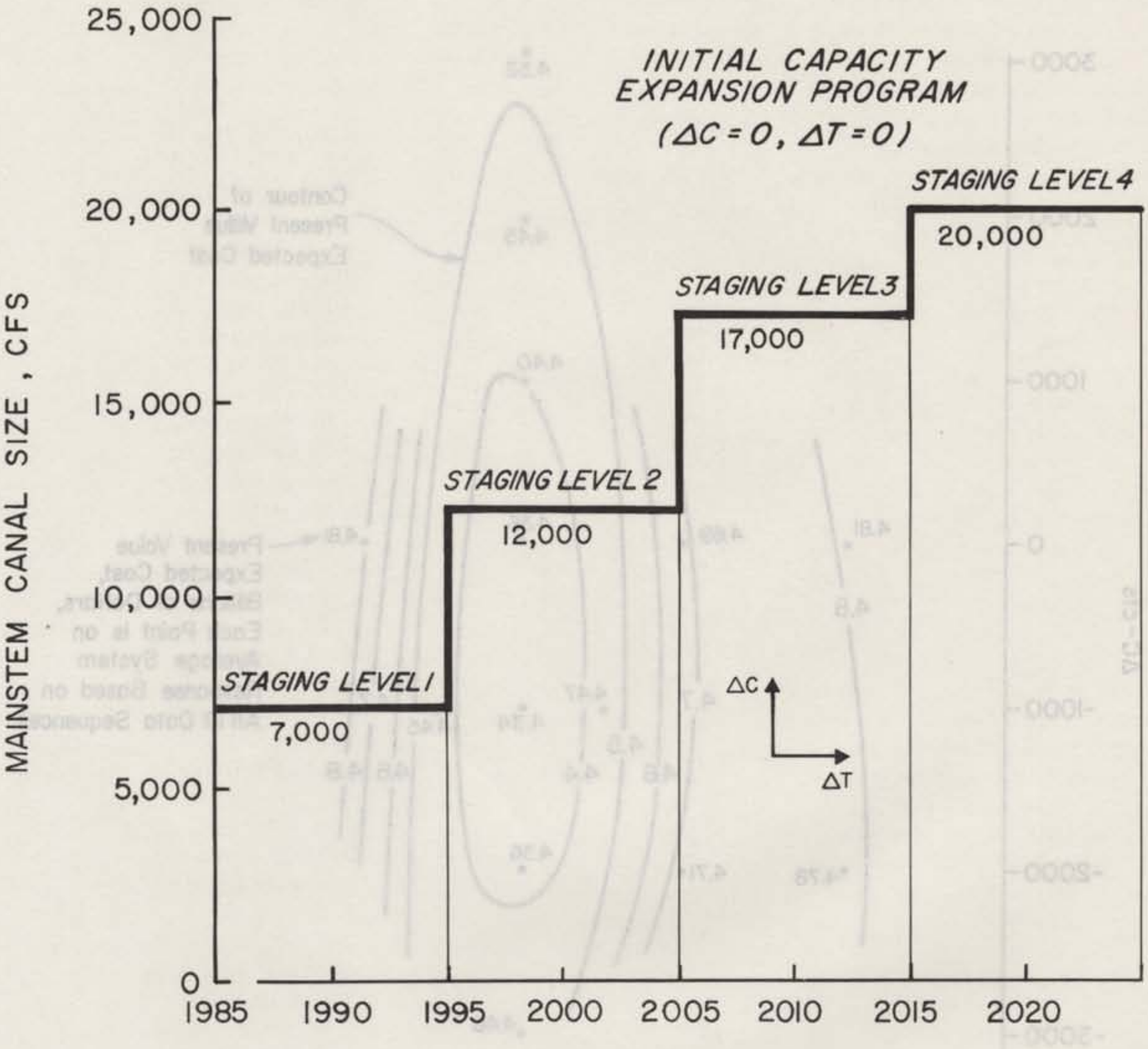



Figure 44

The Initial Canal Capacity Expansion Program, and Notation Used in Subsequent Exploration of Cost Response Surfaces

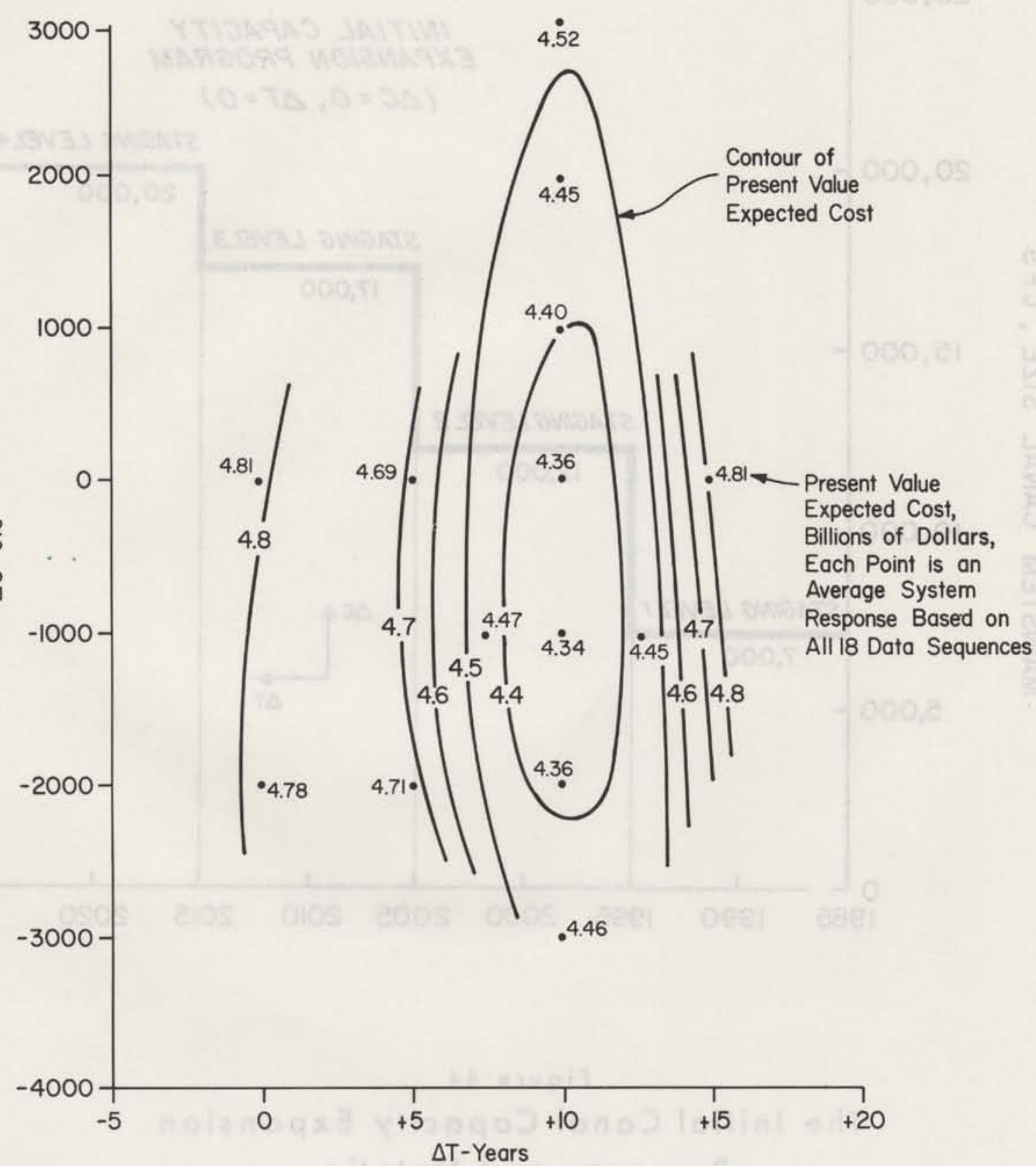



Figure 45

Present Value Cost Response Surface

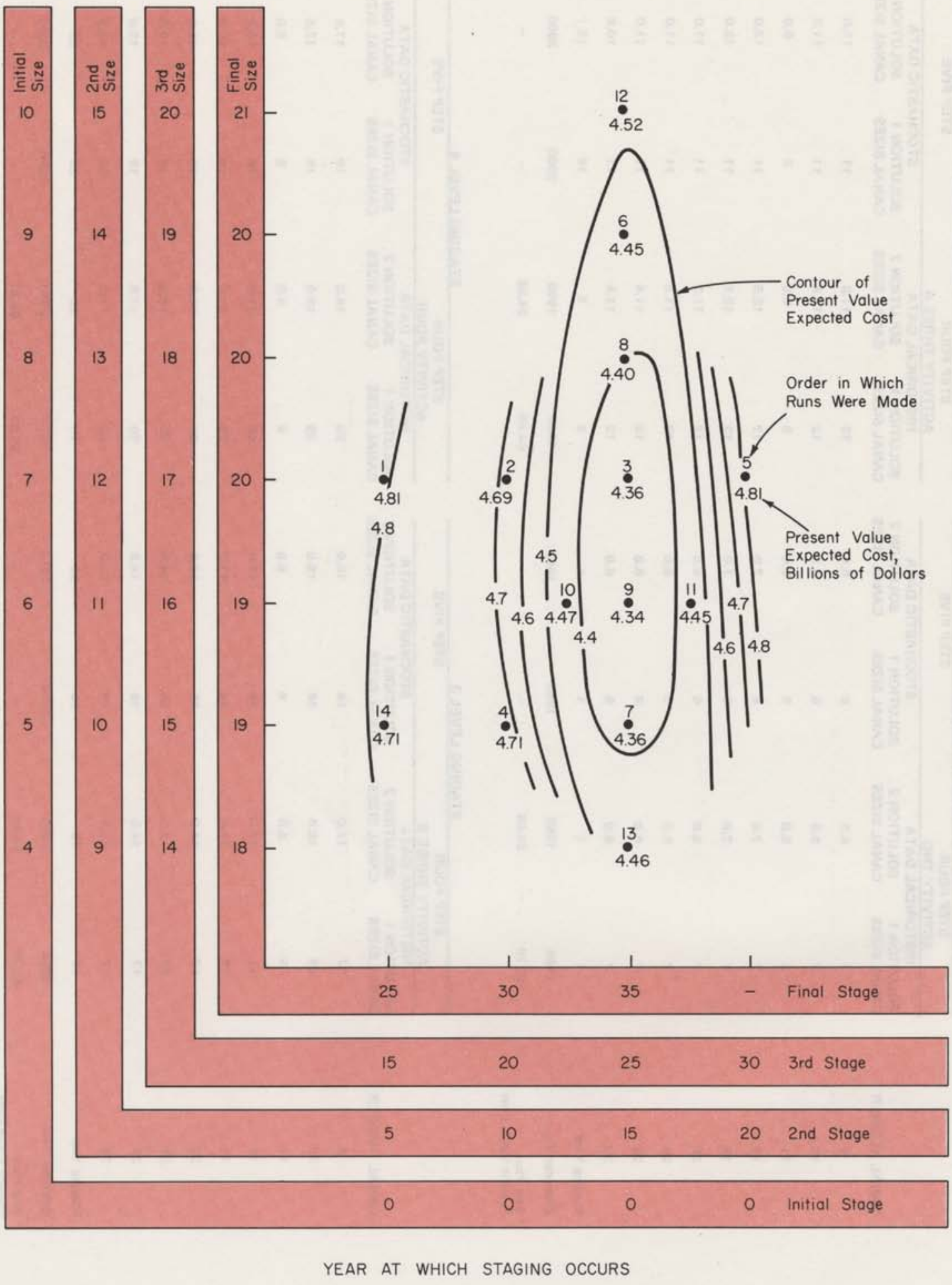



Figure 46

**Present Value Cost Response Surface Showing Canal Sizes**

## Table 9.–Summary of Staging Plan Optimization

| | STAGING LEVEL 1 | | | | STAGING LEVEL 2 | | | |
|---|---|---|---|---|---|---|---|---|
| | STEP FOUR ACTIVITY TWO | | STEP FIVE | | STEP FOUR ACTIVITY THREE A | | STEP FIVE | |
| | HISTORICAL DATA | | STOCHASTIC DATA | | HISTORICAL DATA | | STOCHASTIC DATA | |
| CANAL NUMBER | SOLUTION 1 CANAL SIZES | SOLUTION 2 CANAL SIZES | SOLUTION 1 CANAL SIZES | SOLUTION 2 CANAL SIZES | SOLUTION 1 CANAL SIZES | SOLUTION 2 CANAL SIZES | SOLUTION 1 CANAL SIZES | SOLUTION 2 CANAL SIZES |
| 9 | 7 | 6.5 | 6 | 6.5 | 12 | 11.8 | 11 | 11.0 |
| 10 | 7 | 6.5 | 6 | 6.5 | 12 | 11.8 | 11 | 11.0 |
| 11 | 5 | 5.0 | 5 | 5.0 | 5 | 5.0 | 5 | 5.0 |
| 14 | 7 | 7.0 | 6 | 7.0 | 12 | 12.5 | 11 | 12.0 |
| 19 | 7 | 7.0 | 6 | 7.0 | 12 | 12.5 | 11 | 12.0 |
| 25 | 7 | 6.0 | 6 | 6.0 | 12 | 11.5 | 11 | 11.0 |
| 28 | 7 | 6.0 | 6 | 6.0 | 12 | 11.5 | 11 | 11.0 |
| 29 | 7 | 6.0 | 6 | 6.0 | 12 | 11.5 | 11 | 11.0 |
| 30 | 7 | 6.0 | 6 | 6.0 | 12 | 11.5 | 11 | 10.8 |
| Staging Year | 1 | 1 | 1 | 1 | 5 | 5 | 15 | 15 |
| Calendar Year | 1985 | 1985 | 1985 | 1985 | 1990 | 1990 | 2000 | 2000 |
| Best Plan Present Value Cost | $3.78 | $3.68 | – | – | $4.68 | $4.55 | – | – |

| | STAGING LEVEL 3 | | | | STAGING LEVEL 4 | | | |
|---|---|---|---|---|---|---|---|---|
| | STEP FOUR ACTIVITY THREE B | | STEP FIVE | | STEP FOUR ACTIVITY FOUR | | STEP FIVE | |
| | HISTORICAL DATA | | STOCHASTIC DATA | | HISTORICAL DATA | | STOCHASTIC DATA | |
| CANAL NUMBER | SOLUTION 1 CANAL SIZES | SOLUTION 2 CANAL SIZES | SOLUTION 1 CANAL SIZES | SOLUTION 2 CANAL SIZES | SOLUTION 1 CANAL SIZES | SOLUTION 2 CANAL SIZES | SOLUTION 1 CANAL SIZES | SOLUTION 2 CANAL SIZES |
| 9 | 17 | 15.0 | 16 | 15.0 | 20 | 18.0 | 19 | 17.5 |
| 10 | 17 | 15.0 | 16 | 15.0 | 20 | 18.0 | 19 | 17.5 |
| 11 | 5 | 5.0 | 5 | 5.0 | 5 | 5.0 | 5 | 5.0 |
| 14 | 17 | 17.0 | 16 | 17.0 | 20 | 21.0 | 19 | 20.0 |
| 19 | 17 | 17.0 | 16 | 17.0 | 20 | 21.0 | 19 | 20.0 |
| 25 | 17 | 14.5 | 16 | 14.5 | 20 | 17.5 | 19 | 17.0 |
| 28 | 17 | 14.5 | 16 | 14.5 | 20 | 17.5 | 19 | 17.0 |
| 29 | 17 | 14.5 | 16 | 14.5 | 20 | 17.5 | 19 | 17.0 |
| 30 | 17 | 14.5 | 16 | 14.3 | 20 | 17.5 | 19 | 16.8 |
| Staging Year | 15 | 15 | 25 | 25 | 25 | 25 | 35 | 35 |
| Calendar Year | 2000 | 2000 | 2010 | 2010 | 2010 | 2010 | 2020 | 2020 |
| Best Plan Present Value Cost | $5.30 | $5.10 | – | – | $5.58 | $5.37 | – | – |

NOTES: Canal sizes are in thousands of cfs.
Solutions 1 assume all mainstem canals of equal size; solutions 2 vary individual canal sizes along the mainstem striving to reduce present value costs.
All results are based upon a penalty cost of $100.00 per acre-foot of shortage.
Canals 31, 32, and 33 were unconstrained at 50,000 cfs but priced at 10,000 cfs.

in the day-to-day operation of the system. If this flexibility is desired, it could be accomplished at about a 10 to 20 percent increase in cost. With this trade-off quantified, the planner and the financing agent can be fully aware of what various sub-optimal decisions might cost them and decide whether the extra flexibility is worth the cost.

Another interesting point is that these other plans produced roughly the same distribution of costs as did the minimum-expected-cost plan. This suggests that major trade-offs between cost components probably will not reveal themselves in the course of sensitivity analysis. It further suggests that conclusions regarding the impact of hydrologic variability on project planning economics will be relatively insensitive to variations in the technique of plan development, provided that the overall guiding concepts are invariant.

Considering the search process used, the reader will note that the minimum-expected-cost plan was found by reducing the canal capacities and shifting the location of the capacity staging points forward in time. This illustrates that the plan developed using historical data is conservative. This is because the canal sizes at each staging level were determined with the assumption that the historical critical period would occur at that level of development. With this is mind, the planner should recognize that by design a minimum-expected-cost plan will be one with lower capacities and/or later staging times than the one developed in Chapter V.

One operational question that must be answered and which is considered in the next chapter is how many stochastic records are necessary to derive a reliable estimate of expected cost. The reduction of the number of records would significantly reduce the computer time requirements; for example, if nine records would suffice, the computer time requirement (being directly proportional to the number of records) would be reduced by a factor of two. Reduction in computer time could lead to the consideration of more variables in the plan development procedure. (Recall that only two items, $\Delta C$ and $\Delta T$, were varied.) The more items one can afford to vary in the procedure, the more confidence one has in the nearness of improved plans to the global optimum.

## Computation Reduction Procedures

The analysis procedure simulating each alternative against 18 data sequences, each approximately 30 to 50 years in length, is computationally burdensome. Anything which could be done to eliminate some of the computational burden would greatly accelerate the search for an optimal solution. This is a fruitful area for additional study. Possible approaches aimed at speeding computation were examined and are suggested in this section.

Considering the use of stochastic and historical data, three possible approaches designed to reduce required computational effort are to:

- use portions or segments of each of the historical or stochastically generated sequences,
- use subsets composed of two or more of the entire n-year sequences drawn from the k previously selected sequences (in this example n=36 and k=18), or
- use a combination of the above two approaches.

The methodology described here essentially uses the third approach and is designed to help guide users in determining when to use which sequences and what portions of them in finding and verifying the optimal. Based on the facts that

- many planners have more confidence in the characteristics contained in the historical data than those contained in any given sequence of stochastically generated data,
- it is desirable, from a computational viewpoint, to work with as few years of data as practicable, and
- the primary purpose of using generated stochastic sequences is to be able to evaluate the impact that other equally likely streamflow and demand sequences have on the system response,

considerable dependence is placed upon the use of the available historical data sequence and portions of it (e.g., the critical periods) prior to expending much effort analyzing the impact caused by stochastically generated sequences.

Even though only the historical data are used, the question of how much of that available data needs to be used becomes important. If, for example, firm-yield computations are being made, considerable computer time can be saved by analyzing, in many of the computer runs, only the more droughty periods. On the other hand, if an accurate estimate of the average pumping and shortage[32]/costs is desired, it is probably wise to use, at least at the beginning of the analysis, all of the historical data available. Only upon gaining

[32]/ "Shortage" as used here is the difference between a prespecified demand for water and the amount of water delivered. The simulation and optimization models referred to herein have the capability to assign a penalty cost for shortages incurred; thus, they can be used to trade off the cost of shortages incurred against reservoir or canal capacity costs and solve for the optimal shortage level.

considerable insight into the performance of a given system against a given prespecified data set should the entire-record simulation process be shortened.

#### Selective Use of Only a Portion of a Sequence

Assuming that a user finds it desirable to use only a portion of the historical record to compute average pumping costs and average shortage costs, it would be foolish, of course, to use only the more droughty periods in this analysis because a higher than average shortage cost would probably result. Rather, what is desired is as short a duration as possible that, on the average, contains most of the characteristics of the entire record. This degree of conformity should be measured in terms of the system response—not necessarily the hydrologic variability.

The portion of the record that best exhibits these characteristics can be found by computing and analyzing, throughout the entire historical record, simulated moving average costs and physical system responses for various durations. Table 10 presents the results of this type of analysis on a 36-year simulation run. It shows that several shorter periods in the entire 36-year record can be used to accurately estimate, within plus or minus 2 percent, the pumping costs plus the shortage costs incurred. Perhaps the best shorter period for this example is the one 16 years long that begins in year 14.

By using carefully selected shorter periods of record intermittently with the entire 36-year historical period, a 40 to 50 percent savings in computation time can be realized with negligible reduction in the accuracy of the answer generated. On even a modest sized problem this savings in computer time can represent several thousand dollars.

#### Selective Use of Selected Sequences

Assuming that 18 sequences were selected for analysis, as shown in Figure 38, the question arises of how best to use these sequences in finding the minimum-expected-cost plan. The first step is comprised of evaluating the plan derived in Step Four by simulating its performance against all 18 sequences. The average of the resulting costs (the expected performance) and the range and distribution of these costs then form the basis for further sequence usage.

For example, subsets of

- 9 sequences—one per each category,
- 8 sequences—two per each corner category,
- 4 sequences—one per each corner category,

or any other combination can be selected and evaluated to determine how closely their expected performances approximate that of the entire 18 sequences.

The material contained in Table 11 demonstrates how, for the example hydrologic system, the expected performances vary according to usage of different numbers of sequences and their spatial distribution on the selection chart.

Based on this type of analysis, the subset that most closely approximates the expected performance of the 18 sequences can be used intermittently with the entire set of 18 sequences when striving to find the minimum-expected-cost plan. It is stressed that selected sequence subsets should be used only if it can be demonstrated that the fewer sequences give reasonable results in estimating the portion of the costs that have the greatest variability and thus affect the expected performance the most.

In addition to using various sequence subsets, additional savings in computer time can be realized, without significantly reducing the accuracy of the answers generated, by using only portions of each stochastic sequence. This can be accomplished in the same manner used to analyze the historical data sequence. Only the portion of the stochastic sequence which most closely approximates the response generated by the entire sequence need be used in computing estimates of expected system performance.

By combining these two shortening processes, the amount of computer time required to compare alternatives and thus find the minimum-expected-cost plan can be reduced by at least 50 percent.

## The No Import Case

To this point considerable attention has been given to developing procedures for finding minimum-expected-cost staging plans. The procedures developed have been primarily directed toward the case where an abundance of import water is available. Therefore, the procedures, in essence, help the planner solve for the amount of import water required. In the example problem 10 million acre-feet per year was assumed to be available. As is shown in Figure 20 contained in Chapter V, the average import used was only 4.33 million acre-feet per year at the 2020 demand level, with the import used in any year never exceeding 8 million acre-feet. Since the import water did not constrain the supply to the point where demands could not be met, the problem of sizing demands to a limited supply was not a part of the example problem.

This section poses the problem of finding the optimal level of demands (containing a stochastic component) that can be met with a limited and variable supply. The questions posed are:

**Table 10.—Percent Deviation From 36-Year Average Annual Cost Response**

| YEAR AT WHICH MOVING AVERAGE BEGINS | DURATION OF THE MOVING AVERAGE (YEARS) | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 4 | 6 | 8 | 10 | 12 | 14 | 16 | 18 | 20 | 24 | 28 | 32 | 36 |
| 1 | 11.2 | 22.8 | 25.2 | 17.5 | 12.9 | 4.2 | *1.0* | *− .5* | *− .5* | *−1.7* | 3.3 | *1.3* | *0.0* |
| 2 | 9.4 | 28.1 | 12.7 | 11.9 | 2.8 | *− 1.4* | − 3.1 | − 2.3 | −5.9 | −2.2 | *1.1* | *−1.2* | |
| 4 | 40.5 | 15.8 | 14.0 | 2.7 | − 2.2 | − 4.0 | − 3.0 | − 6.9 | −6.0 | *.1* | *0.0* | *−2.0* | |
| 6 | 16.0 | 13.5 | *− .5* | − 5.7 | − 7.3 | − 5.6 | − 9.7 | − 8.4 | −4.6 | *− .3* | −2.7 | | |
| 8 | −12.5 | −22.5 | −23.6 | −21.8 | −17.5 | −20.5 | −17.7 | −12.3 | −8.0 | −6.8 | −8.0 | | |
| 10 | −16.9 | −20.2 | −18.9 | −14.2 | −18.3 | −15.4 | − 9.7 | − 5.1 | −3.6 | −5.8 | | | |
| 12 | −34.6 | −28.0 | −19.9 | −23.7 | −19.4 | −12.3 | − 6.8 | − 4.9 | −5.7 | −7.3 | | | |
| 14 | −20.8 | −12.4 | −19.01 | −14.8 | − 7.3 | *− 1.7* | *− .2* | *− 1.6* | −3.6 | | | | |
| 16 | − 5.3 | −16.4 | −11.8 | − 3.4 | 2.4 | 3.6 | *1.6* | *− 1.0* | *−1.8* | | | | |
| 18 | −17.2 | −10.8 | *− .5* | 5.9 | 6.6 | 3.9 | *.7* | *− .4* | | | | | |
| 20 | −18.3 | − 2.1 | 6.3 | 7.1 | 3.8 | *.2* | *− 1.0* | | | | | | |
| 22 | 16.2 | 21.3 | 18.6 | 12.3 | 6.7 | 4.4 | | | | | | | |
| 24 | 30.9 | 24.1 | 14.9 | 7.6 | 4.8 | | | | | | | | |
| 26 | 21.0 | 9.8 | *1.9* | *− .3* | | | | | | | | | |
| 28 | *− 1.0* | − 7.9 | − 8.2 | | | | | | | | | | |
| 30 | −17.1 | −14.5 | | | | | | | | | | | |
| 32 | −15.4 | | | | | | | | | | | | |
| 34 | −10.2 | | | | | | | | | | | | |

*Italic numbers represent deviations of less than plus or minus 2 percent.*

**Table 11.—Estimates of Expected Performance for Various Selected Sequences**

| NUMBER OF SEQUENCES AVERAGED | SEQUENCE SUBSETS USED TO COMPUTE EXPECTED PERFORMANCE* | EXPECTED PERFORMANCE POWER PLUS SHORTAGE COSTS (MILLIONS OF DOLLARS) | PERCENT DEVIATION FROM 18-SEQUENCE AVERAGE |
|---|---|---|---|
| 18 | All 18 sequences | 131.7 | 0.0 |
| 9 | 1 sequence per category | 121.7 | − 7.6 |
| 9 | 1 sequence per category | 129.4 | − 1.8 |
| 9 | 1 sequence per category | 132.4 | 0.5 |
| 9 | 1 sequence per category | 131.1 | − 0.5 |
| 8 | 2 sequences each from categories 1, 3, 7, and 9 | 137.0 | 4.0 |
| 6 | 2 sequences each from categories 1, 5, and 9 | 127.4 | − 3.3 |
| 6 | 2 sequences each from categories 7, 5, and 3 | 153.6 | 16.6 |
| 6 | 2 sequences each from categories 1, 4, and 7 | 120.6 | − 8.5 |
| 6 | 2 sequences each from categories 2, 5, and 8 | 137.6 | 4.4 |
| 6 | 2 sequences each from categories 3, 6, and 9 | 137.1 | 4.1 |

* The categories are those shown on the two-dimensional selection chart in Figure 38.

- What expected level of demands can be met if no import water is available?
- What is the relationship between increased demand requirements and incurred shortages?
- What are the optimal levels of shortages for various assumed unit penalty costs? and
- Based upon the above, what is the optimal level of utilization of the limited available supply?

As was the case in the example problem, only the Cypress Creek and Sulphur River basins are considered. Likewise, the demand locations are the same and the system configuration for the 1990 plan shown in Figure 30 and Table 9 is used to perform the following demonstration analysis. The 18 sequences selected in Chapter VI are used to demonstrate the impact that the stochastic variability of the supply and demand quantities have on the problem being analyzed and to help determine the optimal expected level of demands that can be met with a limited water supply.

The results of this analysis are shown in Figure 47. These results were obtained by simulating the performance of the 1990 plan shown in Figure 30 against all 18 selected sequences of supply quantities and varying levels of assumed demands (irrigated acreages). Figure 47a and Table 12 show that 11 levels of required demands were evaluated, ranging from 80 million to 159 million acre-feet of required water. These amounts of water are for a 36-year simulation period—the same period used to find the minimum-expected-cost plan in the first part of this chapter.

The limited supply, based upon the historical sequence, is 124.6 million acre-feet for the 36-year simulation period (3.46 million acre-feet per year); whereas the expected supply, based upon the average of all 18 sequences, is 125.8 million acre-feet (3.49 million acre-feet per year). The historical demand is 121.3 million acre-feet (3.37 million acre-feet per year); whereas the expected demands are 122.3 million acre-feet (3.40 million acre-feet per year). The distribution of the total supply and demand by sequence for the 1990 condition is shown in Table 13. As can be seen the variation in the total supply and demand about the average of all 18 sequences is less than plus or minus 3

**Table 12.—Selected Information for 11 Demand Levels Analyzed**
**(Based on average of all 18 sequences)**

| DEMAND LEVEL | TOTAL 36-YEAR DEMANDS REQUIRED (MILLIONS OF ACRE-FEET) | PERCENT OF 1990 DEMAND LEVEL | DEMANDS REQUIRED (MILLIONS OF ACRE-FEET) PER YEAR | SHORTAGES INCURRED (MILLIONS OF ACRE-FEET) PER YEAR | MAXIMUM INCURRED SHORTAGE (MILLIONS OF ACRE-FEET) PER YEAR | MINIMUM INCURRED SHORTAGE (MILLIONS OF ACRE-FEET) PER YEAR | DEMANDS MET (MILLIONS OF ACRE-FEET) PER YEAR |
|---|---|---|---|---|---|---|---|
| 1 | 80 | 65 | 2.20 | 0.2 | 1.0 | 0.0 | 79.4 |
| 2 | 86 | 70 | 2.37 | | | | |
| 3 | 92 | 75 | 2.54 | 1.2 | 4.4 | 0.0 | 90.0 |
| 4 | 98 | 80 | 2.71 | | | | |
| 5 | 104 | 85 | 2.88 | | | | |
| 6 | 110 | 90 | 3.05 | 5.9 | 13.7 | 0.7 | 104.0 |
| 7 | 116 | 95 | 3.22 | | | | |
| *8* | *122* | *100* | *3.49* | *12.4* | *21.3* | *4.5* | *110.0* |
| 9 | 135 | 110 | 3.73 | 21.1 | 29.8 | 14.4 | 113.5 |
| 10 | 147 | 120 | 4.07 | | | | |
| 11 | 159 | 130 | 4.41 | | | | |

Demand Level 8 is the central case from which percentage deviations were made and corresponds to the 1990 demand level.
The total supply is 124.6 million acre-feet or 3.46 million acre-feet per year.

Table 13.—Supply and Demand Totals for 36-Year Periods by Sequence

| SEQUENCE NUMBER | 36-YEAR AMOUNTS (MILLIONS OF ACRE-FEET) TOTAL SUPPLY | TOTAL DEMANDS | TOTAL DEMANDS MET |
|---|---|---|---|
| 1 | 124.6 | 121.3 | 107.0 |
| 2 | 125.9 | 120.0 | 106.2 |
| 3 | 126.6 | 123.9 | 110.4 |
| 4 | 125.9 | 121.2 | 113.1 |
| 5 | 127.4 | 122.2 | 117.8 |
| 6 | 123.2 | 123.2 | 110.7 |
| 7 | 128.2 | 125.3 | 107.6 |
| 8 | 128.0 | 121.3 | 113.7 |
| 9 | 125.7 | 120.5 | 110.2 |
| 10 | 125.1 | 123.1 | 108.2 |
| 11 | 127.0 | 121.9 | 115.8 |
| 12 | 123.1 | 122.0 | 113.6 |
| 13 | 127.9 | 124.3 | 114.9 |
| 14 | 126.1 | 121.2 | 104.3 |
| 15 | 125.4 | 124.1 | 106.1 |
| 16 | 122.3 | 122.7 | 109.9 |
| 17 | 128.7 | 121.8 | 100.1 |
| 18 | 124.2 | 123.0 | 110.6 |
| Average | 125.8 | 122.3 | 110.0 |
| Average Annual | 3.49 | 3.40 | 3.06 |

percent. This small variation in the average is expected; however, as can be seen in Figure 47a, the amount of shortages incurred varies considerably more (e.g., plus or minus 50 percent from the expected).

The large variation in shortage amounts among the 18 selected sequences indicates that the amount of demands that can be met on the average could vary significantly. For example, consider the case (the 1990 demand level) where the demands required are 122.3 million acre-feet for the 36-year period. The actual demands met with the Figure 30 system configuration varied from approximately 100 million to 116 million acre-feet—a variation of approximately 8 percent above and below the expected level of 107.5 million acre-feet. Converting this variation to irrigated acreage equivalents, 8 million acre-feet provides, on the average, sufficient supplemental water to sustain approximately 120,000 acres.[33] Therefore, there would be approximately a 5 percent chance (1 out of 18) that an irrigation project constructed now would, on the average over the next 36 years, have either 120,000 too many or too few acres. This amount of error in the over-sizing of a project could have considerable effects on the benefits derived from that project and may indicate that a more conservatively sized project should be built. The expected size of the irrigation project is 1.8 million acres (an average of 1.86 acre-feet of supplemental irrigation water per acre per year at the wholesale delivery point).

[33] The average rainfall in the demand area is 18 inches per year.

This variation also indicates that the firm yield can fluctuate from as low as 2.0 million acre-feet per year to as high as 2.5 million acre-feet per year. This represents a fluctuation of approximately plus or minus 12 percent of the expected firm yield of 2.22 million acre-feet per year. In comparison, the historical sequence has a firm yield of 2.15 million acre-feet per year, thus, providing in this case an accurate estimate of the expected firm yield.

Analyzing further, the information contained in Figure 47a shows that as the demands required become larger the expected shortages increase at an increasing rate. Concurrently, the amount of system losses (evaporation and external spills) decrease at a decreasing rate. The decreased losses and, therefore, better utilization of the limited water supply is expected because, with increased demands, the level of the reservoirs, on the average, tends to be lower. With reservoirs lower, less evaporation occurs and there is greater opportunity to retain large flows that would otherwise spill from the system.

Figure 47b shows that as the demands and shortages become greater the demands met approach the total expected supply. However, because of evaporation losses, the demands met never can equal the supply, even if all system spills are eliminated. Similarly, at the upper end of the curve where the demands met approach 115 million acre-feet, shortages incurred to accomplish this are 42 million acre-feet—a highly unsatisfactory condition.

While Figures 47a and 47b contain the hydrologic response of the problem being analyzed, Figures 47c and 47d contain the corresponding cost information. In both Figures 47c and 47d the family of curves shown reflects how the system's total and unit present value cost response changes with various unit penalty costs and levels of demands met. Figure 47d is developed from information contained on Figures 47b and 47c, and contains the answer to the problem being analyzed. As is shown in Figure 47d and itemized in Table 14, the amount of demands that can be met optimally (at minimum cost) is a function of the penalty cost used. The minimum-cost trajectory shows that the lower the penalty cost, the greater is the amount of demands that can be optimally met. This is reasonable because with lower penalty costs more shortages can be tolerated economically and with more shortages tolerated a greater demand can be specified. This also indicates that the lower the penalty cost the greater will be the effective usage of the limited water supply. For example, Table 14 shows that with a penalty cost of $100.00 per acre-foot, 77.7 percent of the available supply is transferred to the demand areas; whereas at a unit penalty cost of $20.00, 85.7 percent of the available supply is transferred to the demand area. Both of these solutions represent optimal shortage conditions; however, for this particular example problem, if the penalty cost is actually $20.00 per acre-foot it really does not pay to transfer any water to the demand areas because the cost to deliver is approximately $32.00 per acre-foot. Both Table 14 and Figure 48 graphically show how the penalty cost interacts with the unit delivery cost, and the ratio of penalty costs to delivery costs. For discussion purposes herin this ratio is assumed to be equal to a pseudo benefit-cost ratio as shown in Figure 48 and as qualified at the bottom of Table 14. With this in mind, if as assumed in the example, the penalty cost is $100.00 per acre-foot, the pseudo benefit-cost ratio is 2.9. If, on the other hand, the penalty cost is $20.00 per acre-foot, the pseudo benefit-cost ratio is 0.6.

For this year's project (Project II), no attempt was made to quantify the methodology for interacting the actual penalty-cost information (the minimum-point trajectory in Figure 47d) with the actual benefits. This is an area needing further study, and is a portion of next year's research (Project III).

Therefore, the only conclusion that can be drawn from Figures 47 and 48 is that the optimal level of demands met is between 77 and 84 percent of the available supply and this amount of water is between 22 and 33 percent greater than the expected firm yield of the system. It is again stressed, however, that these conlcusions are problem-specific. Although they may be used to guide the direction of future studies, they are at this time based on unverified hydrologic and cost data sets. The data sets used were developed purely for model development and verification purposes, and the results should be reviewed in that context.

## Concluding Remarks

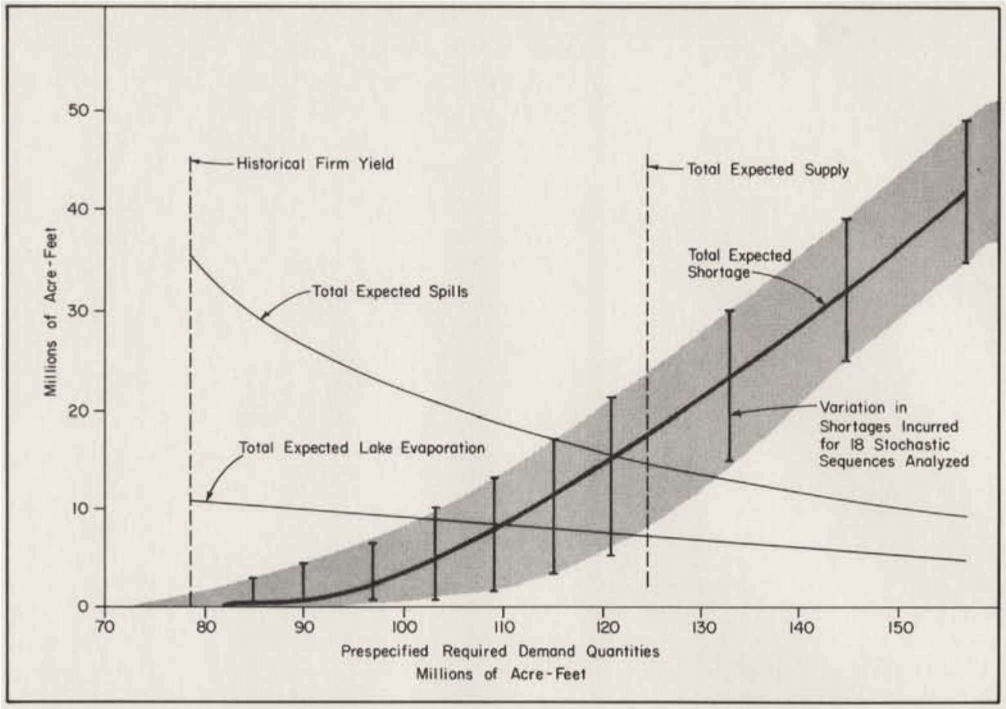
The methodology described in this chapter does not make use of existing stochastic programming procedures such as the chance-constrained programming of Charnes and Cooper (1959) and other similar methods (Chow and Meredith, 1969b). These methods, if used, would not permit the representation of the system in the modeling detail possible through the use of SIM-III. Furthermore, if it were not necessary to model the system as precisely as possible through the use of SIM-III, stochastic programming procedures could be used; however, in most cases they do not provide the output detail necessary to measure the possible variability in system responses. Often, information concerning the variability in system response is as important to the analyst in planning as is the minimum-expected-cost plan. Examples of the variability analyses which can be conducted are given in the following chapter.

**Table 14.—Optimal Demands Met, Unit Cost of Delivery, and Pseudo Benefit-Cost Ratio Information**

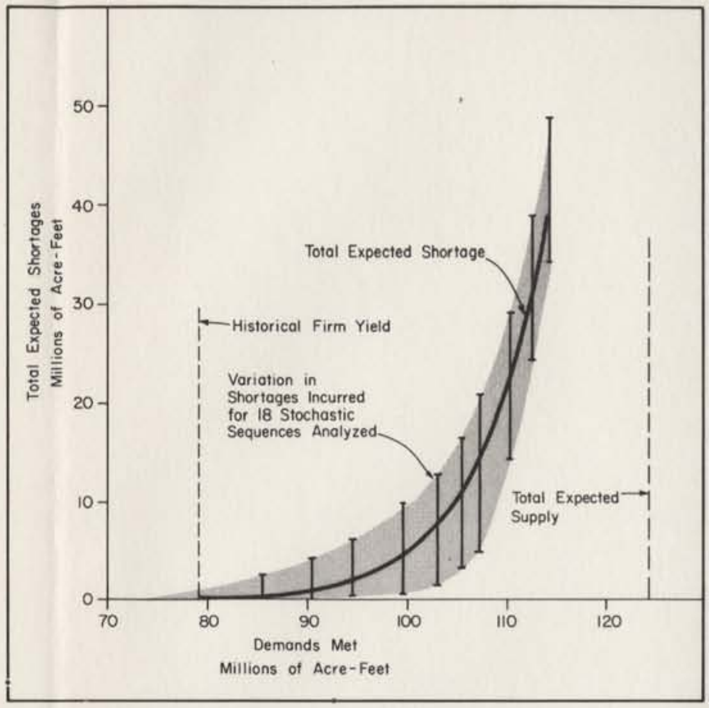
| SHORTAGE PENALTY COST (DOLLARS PER ACRE-FOOT) | OPTIMAL LEVEL OF DEMANDS MET | | | | | |
|---|---|---|---|---|---|---|
| | TOTAL OVER 36 YEARS (MILLIONS OF ACRE-FEET) | AVERAGE ANNUAL (MILLIONS OF ACRE-FEET) | PERCENT ABOVE EXPECTED FIRM YIELD | PERCENT OF AVERAGE AVAILABLE SUPPLY | UNIT COST OF DELIVERY (DOLLARS PER ACRE-FOOT) | RATIO OF PENALTY COST TO DELIVERY COST* |
| 100 | 97.5 | 2.71 | 22.1 | 77.7 | 34.90 | 2.9 |
| 80 | 99.5 | 2.76 | 24.3 | 79.1 | 34.45 | 2.3 |
| 60 | 102.0 | 2.83 | 27.5 | 81.1 | 33.90 | 1.8 |
| 40 | 104.5 | 2.90 | 30.6 | 83.1 | 33.15 | 1.2 |
| 20 | 107.5 | 2.99 | 34.7 | 85.7 | 32.00 | 0.6 |

* This ratio is based upon the assumption that the unit penalty cost (being equal to the damage incurred for not delivering water) approximates the benefits from delivery of the water.

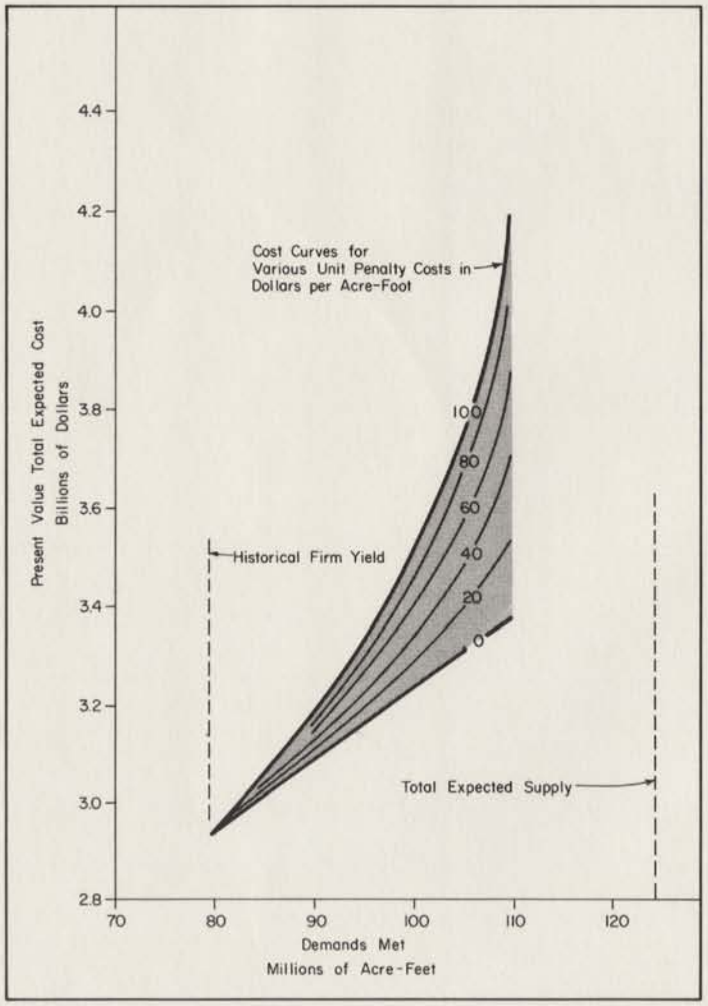
It is possible that stochastic programming procedures may be utilized effectively in establishing initial low-cost plans to be used as starting points for more detailed simulation and optimization programs such as SIM-III. Initial study of the feasibility of this approach is underway and some promising results are expected.



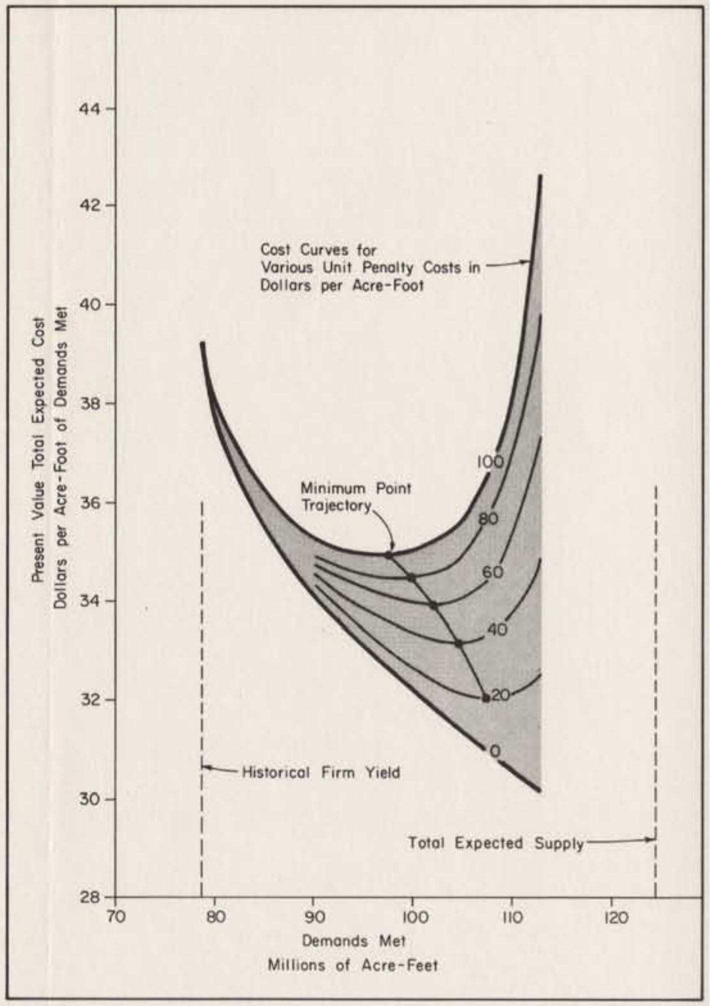

a. Shortages Versus Demands




b. Shortages Versus Demands Met




c. Total Cost Versus Demands Met




d. Unit Cost Versus Demands Met

Figure 47

Sizing Demands to a Limited Supply

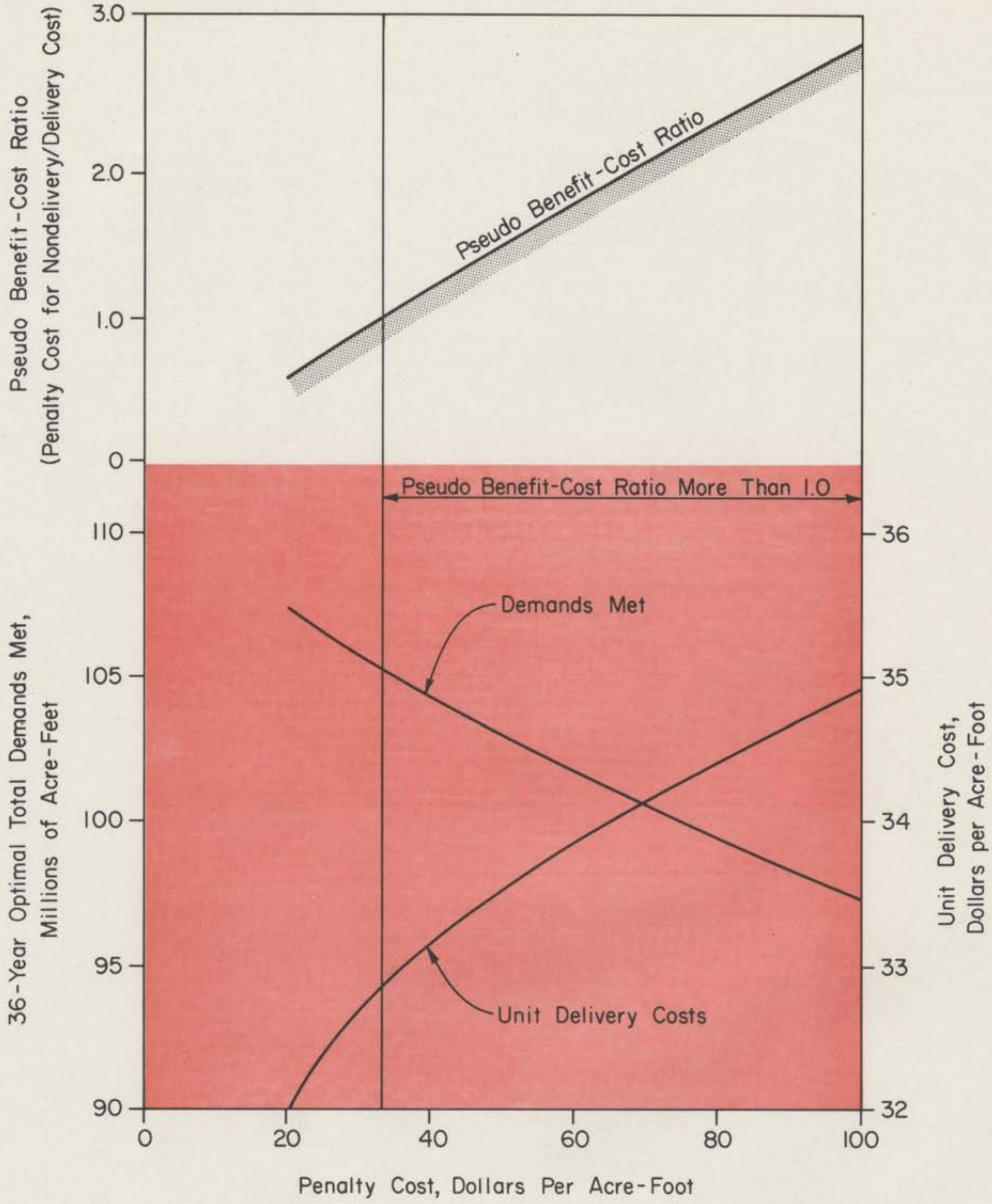



Figure 48
Penalty Costs Versus Demands Met, Unit Delivery Costs, and Pseudo Benefit-Cost Ratios

# VIII. ANALYSIS OF VARIABILITY

This chapter describes the results of analyzing the impact that stochastic variability in the hydrologic input data have on the cost response of the prototype being simulated. This chapter also provides the framework for determining how this variability might cause the planner to select a more costly staging plan in lieu of the minimum-expected-cost plan derived in Chapter VII because too much variability exists in the physical and cost performance of the minimum-expected-cost plan.

In the discussion to follow, an example set of the economic response data is presented initially and its distribution is analyzed. Then, the components of the total present value cost are studied to assess their contribution to the variation of the total cost. Following this assessment, an analysis of the total cost variance is presented and the consequences of basing planning decisions on minimizing expectations are analyzed. Next, the results of a correlation study between the economic variation and hydrologic attributes of drought are discussed. Finally, the question of the number of sequences needed to compute reliable expectations is considered.

In the previous chapter, as shown in Figures 45 and 46, 14 different implementation plans were subjected to 18 preselected runoff and demand sequences in the search for the plan that minimizes the expected or average cost. The analysis of cost variation in these 14 implementation plans, which is presented in this chapter, utilizes the cost data generated in that search. Table 15 contains an example data set for one implementation plan and lists only those data which vary; these are import water, power, shortage, and total present value costs. The example plan corresponds to the point $\Delta C=-2{,}000$ and $\Delta T=+10$ shown on Figure 45.

**Table 15.—Cost Summary for the 18 Selected Sequences**

| SEQUENCE NUMBER | PRESENT VALUE COST (BILLIONS OF DOLLARS)* IMPORT WATER COST | POWER COST | SHORTAGE COST | TOTAL COST |
|---|---|---|---|---|
| 1 | 0.05 | 1.37 | 0.29 | 4.20 |
| 2 | .04 | 1.31 | .58 | 4.42 |
| 5 | .05 | 1.39 | .46 | 4.39 |
| 6 | .05 | 1.41 | .33 | 4.27 |
| 10 | .04 | 1.38 | .48 | 4.39 |
| 11 | .06 | 1.38 | .52 | 4.44 |
| 12 | .05 | 1.42 | .35 | 4.30 |
| 20 | .05 | 1.40 | .35 | 4.28 |
| 21 | .05 | 1.42 | .43 | 4.38 |
| 23 | .04 | 1.37 | .28 | 4.19 |
| 40 | .04 | 1.37 | .35 | 4.24 |
| 55 | .06 | 1.37 | .40 | 4.31 |
| 63 | .05 | 1.40 | .45 | 4.38 |
| 65 | .05 | 1.35 | .66 | 4.54 |
| 81 | .07 | 1.48 | .34 | 4.37 |
| 73 | .06 | 1.39 | .47 | 4.40 |
| 87 | .07 | 1.45 | .23 | 4.25 |
| 96 | .05 | 1.36 | .52 | 4.42 |
| Expected Cost | | | | 4.36 |
| Standard Deviation | | | | 0.09 |
| Coefficient of Variation (Percent) | | | | 2.1 |

* Costs included in total and not shown individually (reservoir costs, and conduit capital and operation and maintenance costs) are the same for all 18 sequences. Each row corresponds to one optimization analysis.

Values shown above are for the example staging plan corresponding to the point $\Delta C=-2{,}000$ and $\Delta T=+10$ shown on Figure 45.

## Distribution of Costs

How is the present value of the total cost distributed and how is it disaggregated into its components? As an example, consider the total cost data listed in Table 15. Figure 49 shows these data plotted on a normal probability scale; it also shows a theoretical normal distribution based on the calculated mean and standard deviation of the data. The data points shown on Figure 49 are positioned according to the n+1 plotting position assumption. Thus, for this case, the 18 data points are ordered highest to lowest and a probability of 1/19 is assigned to each datum. The accumulation of these probabilities are the values plotted on the ordinate of this figure.

Eighteen observations are normally considered to be a weak basis upon which to form an inference. However, because of the sequence selection procedure employed, the 18 observations can be considered to be representative of the range of responses which would occur if all sequences which were generated were used in the analysis. As shown in Figure 49, the distribution which is derived from this small sample roughly follows the theoretical normal distribution. Therefore, for operational purposes, the economic responses are assumed to be normally distributed. In support of this data interpretation, the reader will recall that the deterministic optimizations operate through a considerable number of additive linear transformations. Under these circumstances, the Central Limit Theorem, which states that the sums of large numbers of random events tend to be normal, supports the normality assumption. The normality assumption is used to derive probability statements presented in the next section.

Consider now the distribution of costs among its components. Table 16 contains the results of an analysis of the variation in the different cost items. These results show that variation is high in import and shortage costs and low in power costs. The lack of much variation in power costs is directly related to the high penalty costs for shortages, and the constraints imposed on the canal flows. The high penalty costs forced demands to be met as long as it was technically possible, and the canal flow constraints kept a sufficient supply of water moving to West Texas. These two factors plus the relatively constant pumping lift of the system kept the variation in power costs to a minimum.

Coefficients of variation (expected value divided by the standard deviation) of import and shortage costs are 15.8 and 26.9 percent, respectively. Import costs also have a very low expected cost, and consequently, the variation in this small cost will not have much effect on variations in total cost. On the other hand, shortage costs are about 8 percent of the total cost and their variation (as measured by the standard deviation) is greater than the variation in total costs. Therefore, shortage costs are very important in describing variations in total cost.

## Variance Analysis

For each capacity expansion program considered, the coefficient of variation of total costs, expressed as a percentage, was computed. These values are shown on Figure 50 along with contours that illustrate the coefficient of variation response surface. The contours appear regular, which indicates a functional relationship between capacity expansion and total cost variation. A general trend is noted that as construction is deferred (T moves right), the variation in total cost increases, and that as the capacity is increased (C moves up), the variation is decreased. It is also noted that the contours of the coefficient of variation do not correspond with the general shape of the contours of expected value as shown in Figures 45 and 46. In fact, there is a slight trend indicating an increase in the coefficient of variation as the expected cost decreases.

**Table 16.—Variation of Cost Components**

| | BILLIONS OF DOLLARS | | COEFFICIENT OF VARIATION (PERCENT) |
|---|---|---|---|
| | EXPECTED VALUE | STANDARD DEVIATION | |
| Present Value Import Costs | 0.05 | 0.0082 | 15.8 |
| Present Value Power Costs | 1.43 | .0393 | 2.8 |
| Present Value Shortage Costs | .32 | .0873 | 26.9 |
| Present Value Total Costs | 4.34 | .0816 | 1.9 |

n = 18.
The total variance is not the sum of the components because of covariance among the cost components.
Case: $\Delta C = -1{,}000$, $\Delta T = +10$ (Of the combinations studied, this had the lowest expected total cost.)

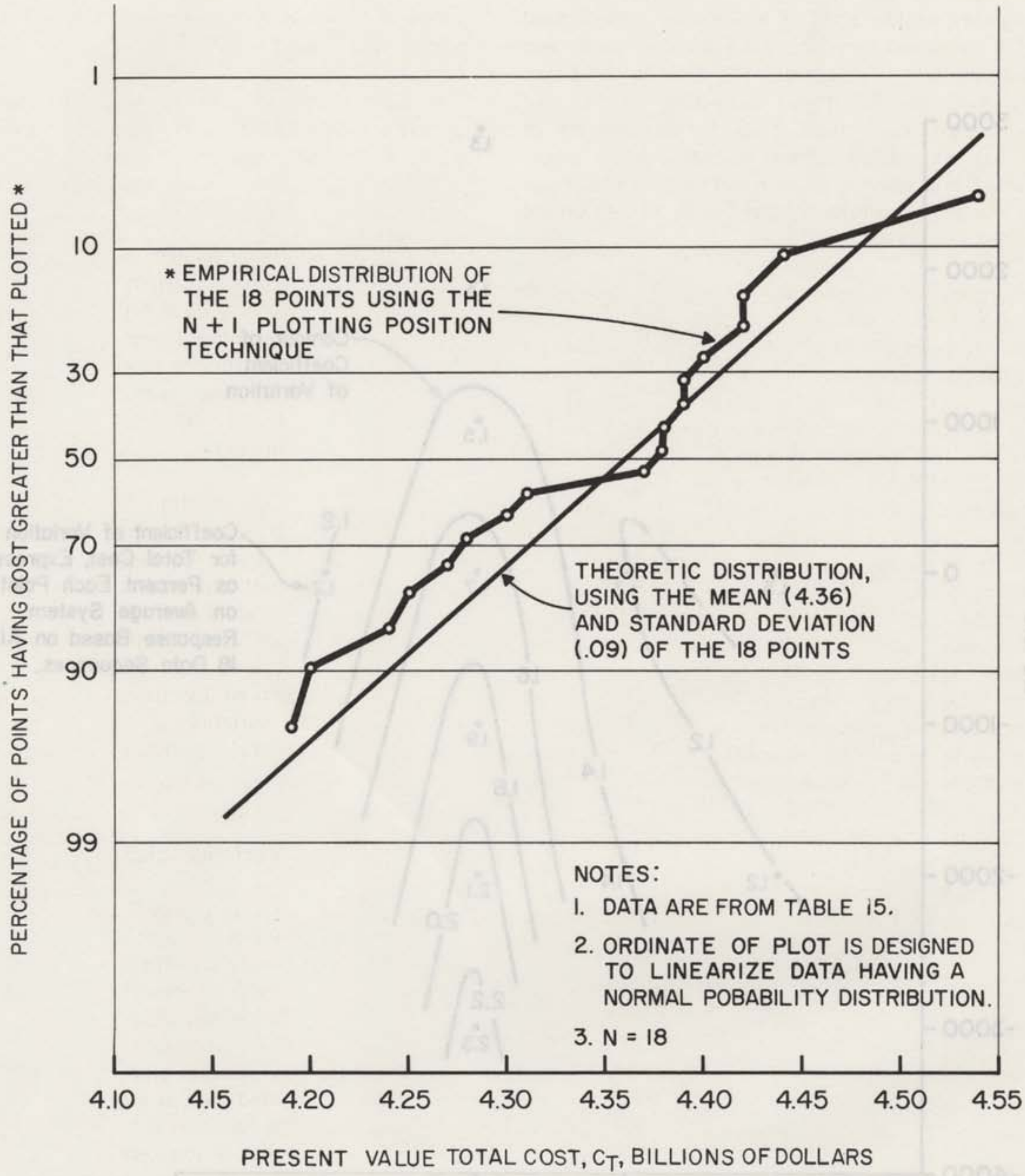



Figure 49

Cumulative Probability Distribution of Present Value Total Cost

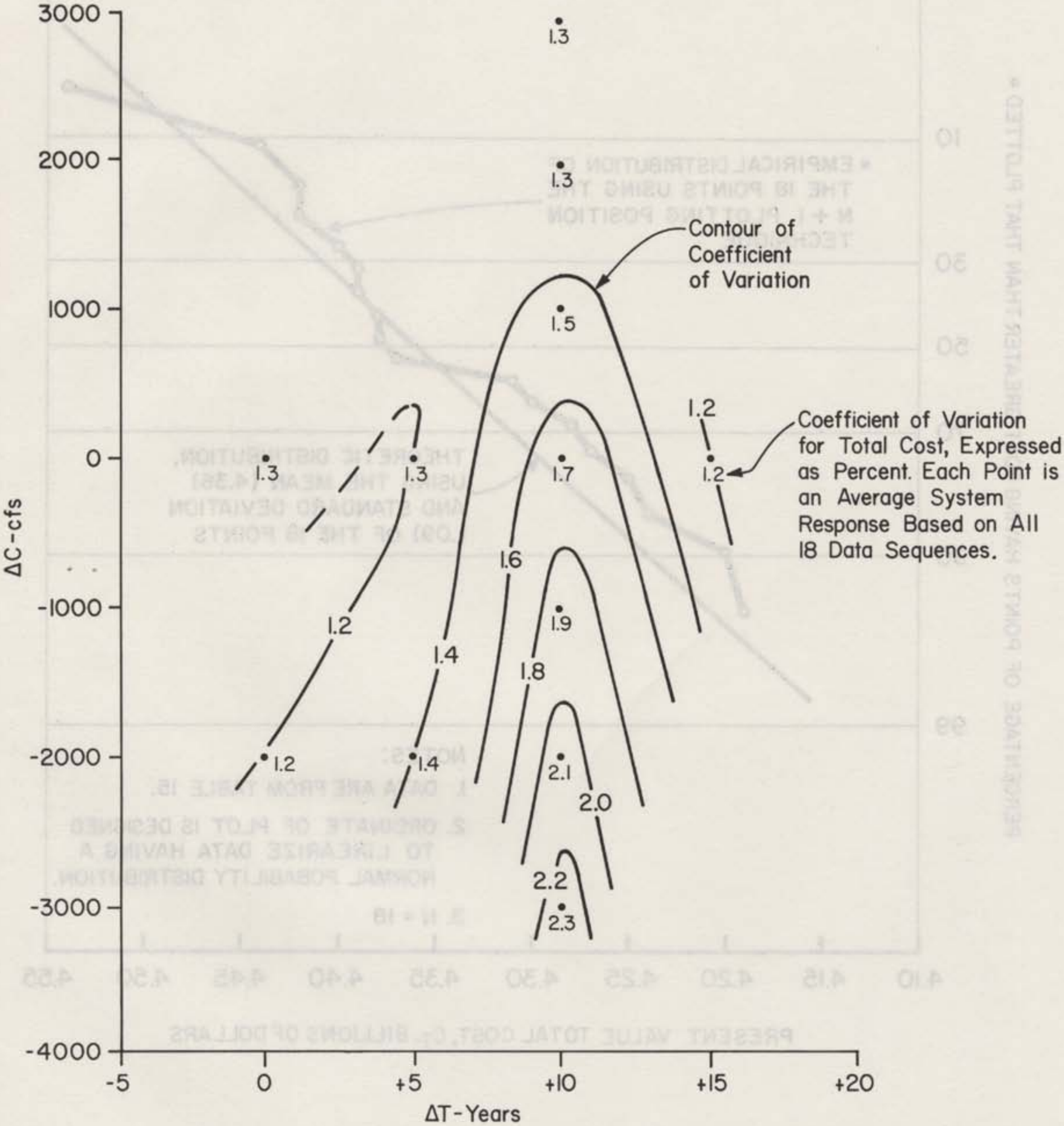



Figure 50

**Present Value Cost--Coefficient of Variation Response Surface**

The values of the coefficient of variation shown on Figure 50 are small, indicating that hydrologic variability has only a modest impact on the variability of the economic response. The values range from 1.2 to 2.3 percent. A good number to use in representing the coefficient of variation of the economic response is 2 percent. It follows then, assuming normality, that the economic response is expected, with 95 percent probability, to fall within 8 percent of the expected value. Eight percent corresponds to a range of four standard deviations.

There are two possible assumptions in the deterministic optimization procedure that could reduce the impact of hydrologic variability on economic response. These are (1) the reservoir operation rule and (2) the assumption concerning the availability of import water.

To assess the degree to which system cost variability is influenced by the reservoir operation criteria, the minimum-expected-cost staging plan was analyzed using two sets of operation criteria. The first set assumed a 1-year knowledge of the future hydrologic events prior to simulation, whereas the second set assumed no prior knowledge. This comparative analysis resulted in an expected present value total cost of $4.54 billion and a coefficient of variation of 2.7 percent for the second set of operation criteria; the comparable cost with the first set of operation criteria was $4.34 billion and a coefficient of variation of 1.9 percent. Thus, perfect knowledge operation criteria lowered both variability (1.9 percent as opposed to 2.7 percent) and expected costs ($4.34 billion as opposed to $4.54 billion).

To determine whether variability was influenced by the assumption concerning availability of import water, the initial plan ($\Delta C=0$, $\Delta T=0$) was studied at 1990 canal capacities and demand values with the constraint that no water could be imported. This condition produced a coefficient of variation in yearly costs (total costs divided by 36 with no interest rate adjustment) of 3.32 percent. Although this number is not strictly comparable with the 1.9 percent presented in Figure 50, it strongly suggests that the availability of import water reduces the economic effects of hydrologic variability.

Finally, consider the expected costs (Figure 46) and coefficients of variation (Figure 50) jointly. For each implementation plan considered, there exists a conservative economic response which has a 1 percent chance of being exceeded. This conservative response is computed as the expected cost plus 2.35 times its standard deviation, assuming that costs are distributed normally. Figure 51 illustrates cost contours for such conservative economic responses. It is reassuring that the least costly implementation plan indicated by this plot ($C=-1{,}000$ and $T=+10$) is the same one that resulted in minimum total expected costs (see Figure 46). There is, however, no reason to believe this will be the general case. Different levels of conservatism (for example, 5 percent rather than 1 percent) may yield different results. Additional planning problems should yield information regarding decisions based on linear combinations of the mean and standard deviation.

## Correlation Study

To analyze the factors contributing to cost variation, a correlation study was made. Figure 52 shows the correlation matrix for a set of 12 variables. These variables are the position and magnitudes of droughts of duration 42, 54, 66, and 78 months plus the total present value cost and its three component variables. This matrix contains coefficients of correlation between each of the 12 variables.

Of particular interest are the correlation coefficients of the cost components with the hydrologic attributes which are shown in the enclosed lower left hand rectangle of the matrix. Fisher's Z transformation for 18 observations indicates that the sample correlation coefficient should exceed 0.5 before one can be 95 percent certain that the underlying population value is non-zero. Most values in the enclosed rectangle are less than 0.5, which indicates weak correlations between the four cost variables and the eight hydrologic attributes. All correlation coefficients between cost components and the position of the droughts are well below the 0.5 significance level. These low coefficients indicate that, for the test case, the position of droughts has little effect on cost.

On the other hand, there are some significant correlations between cost components and drought magnitude. Import costs correlate fairly well with the magnitude of the 78-month drought but poorly with the shorter term events. The assumption limiting the availability of import water could cause this situation. Shorter term droughts, which are more severe in magnitude, could require more import water than is available. However, the limit on imports forces shortages to occur and breaks down the correlation with the shorter term events. Correlations between shortage costs and drought magnitude support this argument. The magnitude of the 42-month drought, which is the shortest event considered, correlated significantly with shortage costs. Thus, the evidence contained in the correlation matrix indicates that droughts of duration less than 42 months should have been considered.

Power costs correlate weakly with drought magnitude. These correlation coefficients are all negative, which indicates that power costs decrease as droughts become more severe and shortages are incurred even though water is being imported. This reinforces an earlier conclusion that shortage costs are more important than import costs in describing the variation in total

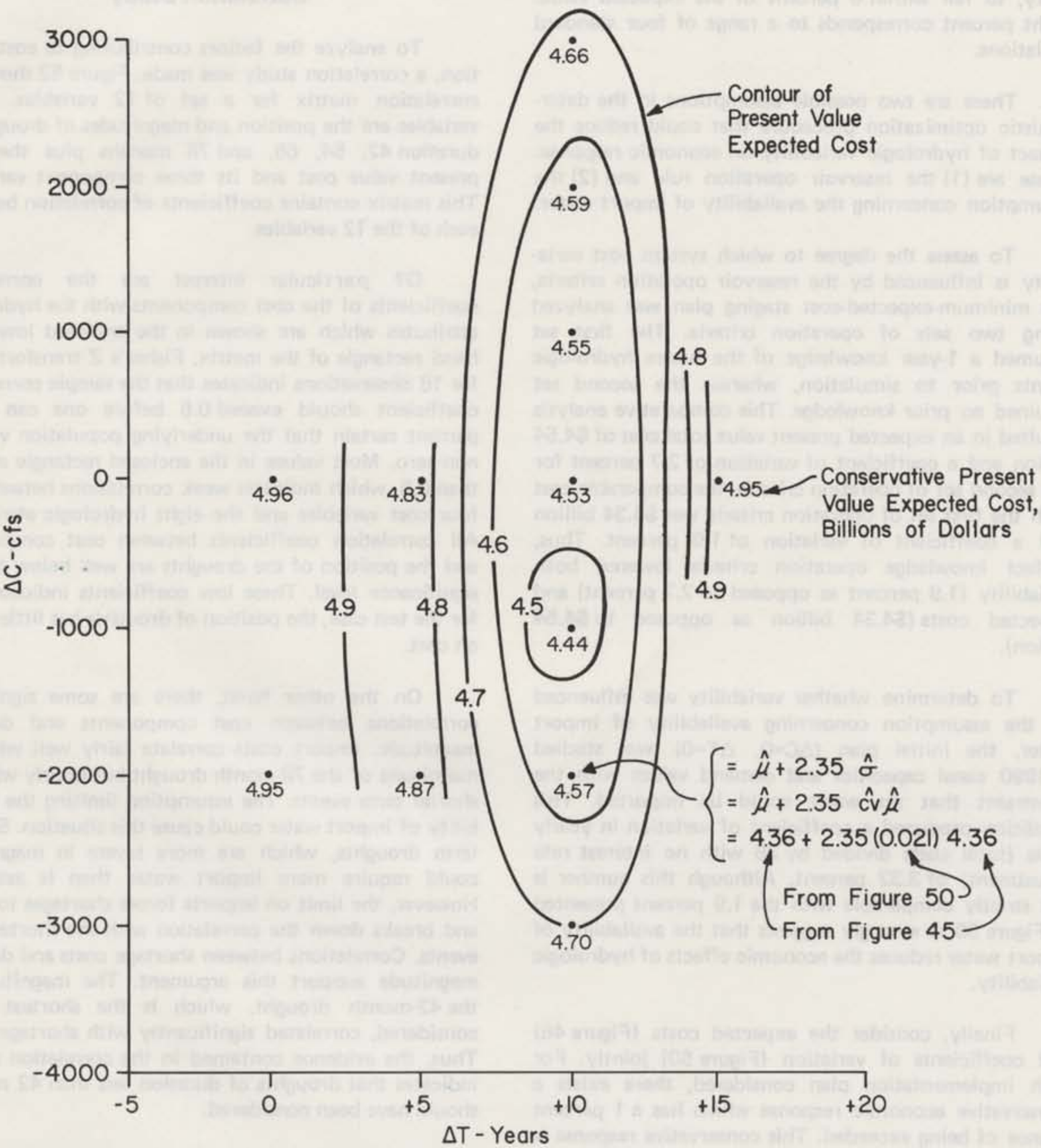



Figure 51

Conservative Present Value Cost Response Surface

| | | $P_{42}$ | $M_{42}$ | $P_{54}$ | $M_{54}$ | $P_{66}$ | $M_{66}$ | $P_{78}$ | $M_{78}$ | $C_1$ | $C_2$ | $C_3$ | $C_T$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| HYDROLOGIC ATTRIBUTES | $P_{42}$ | 1.0 | | | | | | | | | | | |
| | $M_{42}$ | .16 | 1.0 | | | | | | | | | | |
| | $P_{54}$ | .97 | .25 | 1.0 | | | | | | | | | |
| | $M_{54}$ | .02 | .86 | .09 | 1.0 | | | | | | | | |
| | $P_{66}$ | .73 | .26 | .73 | .31 | 1.0 | | | | | | | |
| | $M_{66}$ | -.03 | .79 | .03 | .83 | .24 | 1.0 | | | | | | |
| | $P_{78}$ | .88 | .18 | .88 | .03 | .59 | -.19 | 1.0 | | | | | |
| | $M_{78}$ | -.06 | .73 | .01 | .76 | .10 | .89 | -.15 | 1.0 | | | | |
| COSTS | $C_I$ | .23 | -.20 | .26 | -.14 | .19 | -.38 | .32 | -.57 | 1.0 | | | |
| | $C_2$ | .02 | -.50 | -.03 | -.35 | .05 | -.33 | .01 | -.46 | .64 | 1.0 | | |
| | $C_3$ | .04 | .46 | .06 | .13 | -.12 | .07 | .19 | .03 | -.13 | -.47 | 1.0 | |
| | $C_T$ | .08 | .30 | .08 | .01 | -.10 | -.07 | .25 | -.19 | .18 | -.09 | .92 | 1.0 |

*KEY (see text for definition of position and magnitude)*

$t$ = LENGTH OF CRITICAL PERIOD (months)
$P_t$ = POSITION OF DROUGHT
$M_t$ = MAGNITUDE OF DROUGHT
$C_I$ = PRESENT VALUE IMPORT COSTS
$C_2$ = PRESENT VALUE POWER COSTS
$C_3$ = PRESENT VALUE SHORTAGE COST
$C_T$ = PRESENT VALUE TOTAL COST

Figure 52

Correlation Matrix for 12 Cost and Hydrologic Variables costs. Total costs did not correlate significantly with any of the drought attributes.

Additional evidence in support of the conclusions drawn in this chapter is contained in the correlation coefficients among the cost components. Both import and shortage costs correlate well with power costs. However, when they are correlated with total costs only shortage costs are significant. Since shortages costs are a direct function of shortages, the variation in total costs appears to be mainly attributable to shortages.

## Required Number of Sequences

A final question is considered in this chapter: how many records are necessary to obtain reliable estimates of present value expected cost? The question is approached from a theoretical viewpoint through consideration of the standard error of the mean. The sample mean, $\hat{\mu}$, is an unbiased estimate of the true mean, $\mu$, of a population (providing it is based on a random sample), and the sample mean has a standard deviation equal to the standard error, SE($\hat{\mu}$). The standard deviation of the population mean is computed by

$$SE\ (\mu) = \frac{\sigma}{\sqrt{n}}$$

where $\sigma$ is the population standard derivation of the original data and n is the population size. The value $\sigma$ is not known, therefore, the operational equation is

$$SE\ (\hat{\mu}) = \frac{\hat{\sigma}}{\sqrt{\hat{n}}}$$

where $\hat{n}$ is the sample size and $\hat{\sigma}$ is the sample standard deviation. The percent error is defined as follows

$$\text{Percent Error} = \left(\frac{|\mu - \hat{\mu}|}{\mu}\right) \times 100 \ .$$

It is desired to keep the percent error small to increase ones confidence that $\hat{\mu}$ is approximately equal to $\mu$. Figure 53 shows probability levels for various combinations of n and percent error. The figure assumes $\hat{\mu}$ is normally distributed with mean $\hat{\mu}$ and standard deviation SE ($\hat{\mu}$). Figure 53 shows from a theoretical viewpoint that one can obtain, with high probability, a small percent error with small sized samples. Consider the top curve, the 99 percent confidence plot. The probability statement associated with this curve is that there is a 99 percent chance that the percent error will be less than the values traced by the curve. Similar statements apply to the other curves. For the example problem four sequences are sufficient to get only 2.5 percent error or less with 99 percent confidence. In other words, the 18 sequences selected for this study are adequate.

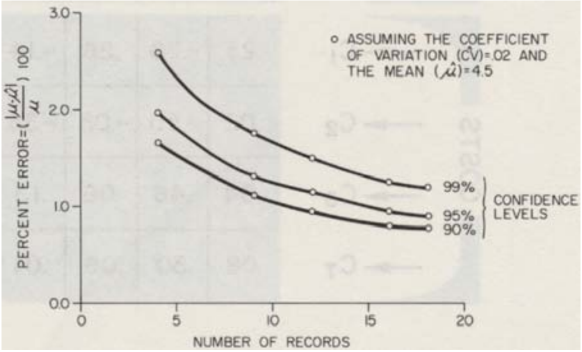


Figure 53.—Percent Error Versus Number of Records

# IX. SENSITIVITY ANALYSES

This chapter describes the procedure comprising Step Six in the overall plan optimization procedure discussed in Chapter II. This is the final step in the planning procedure and involves testing the sensitivity of the cost and physical response of the simulated prototype to variations in all of the input parameters used by the models. For purposes of this discussion, the minimum-exptected-cost plan developed in Step Five is used and referred to as the central case. Input parameters evaluated in this sensitivity analysis are:

- prespecified water requirements,
- monthly distribution of the water requirements,
- gross evaporation rates in the demand area,
- quantity of import water available,
- duration over which import water is available,
- rainfall in the demand area,
- root depth of the various crops in the demand area,
- reservoir inflow quantities in the supply area,
- reservoir storage capacities in the supply area,
- reservoir surface area in the supply area,
- net lake surface evaporation rates in the supply area,
- power costs for pumping,
- different methods of discounting the total cost of implementation plans, and
- the assumed starting storage contents of reservoirs.

This analysis also measures the impact that stochastic variability in the supply and demand quantities has on the Step Five minimum-expected-cost staging plan. Based upon this information and a measure of tolerable risk, the sensitivity analysis provides the means for perhaps selecting an implementation plan other than the minimum-expected-cost plan indentified in Step Five if the risk or variability for that plan is found to be unacceptably high.

## Response to Changes in Requirements for Water

The projected requirements for water constitute some of the most important data in water planning studies because they furnish the driving force creating the need for water development. Water requirements to be supplied by proposed projects are developed based on expected increases in population or industrial and agricultural activities. As in all projections, inherent uncertainties are associated with the magnitude of these requirements. If water demands develop at a greater or lesser rate than that projected as a basis for planning, the operational requirements for the system will change.

Consider the information shown in Figure 54. The change in total system cost (capital cost plus operation and maintenance cost) is depicted for water requirements varying above and below those projected. The projected water requirement corresponds to the central plot of the curve (corresponding to a cost of $9.18 billion). Note that the rate of change of total system cost increases at a greater rate as demands rise. Water requirements shown are cumulative values for the 36-year planning period at the 2020 level of demand.

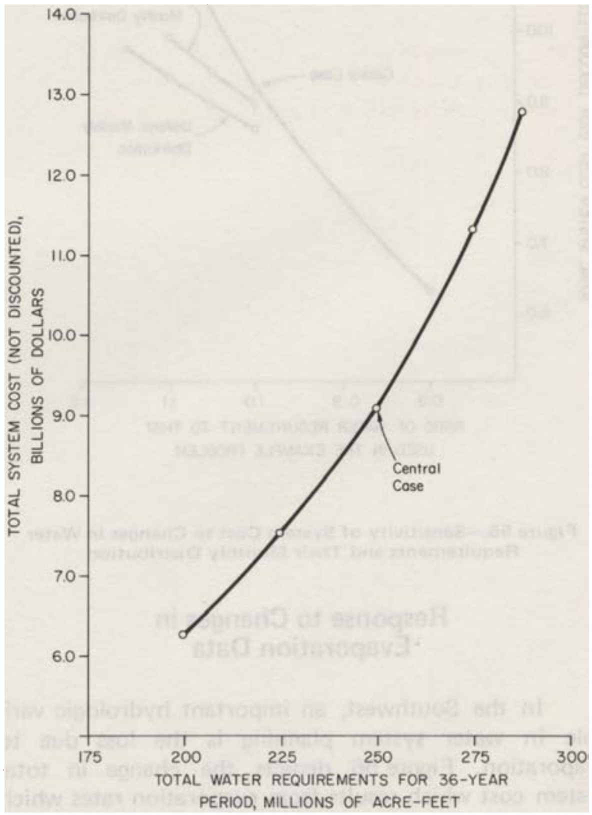



Figure 54.—Sensitivity of System Cost to Changes in Water Requirements

The effect of the monthly distribution of the demand for water is illustrated by Figure 55. In the figure, the response of total system cost is shown as a result of variation in the quantity of water required when the monthly distribution of that total requirement is varied. Three monthly demand distributions were considered, and the total quantity of water demand was varied for each of the three distributions. The distributions were: (1) a uniform distribution, applying an equal percentage of the annual demand in each month, (2) an average distribution, applying a distribution of monthly percentages based upon observed uses, and (3) a stochastic variation in demand in which the demands were computed using a consumptive-use model with stochastic hydrologic data. The stochastic variation results in the greatest system costs because of the erratic nature of the demands.

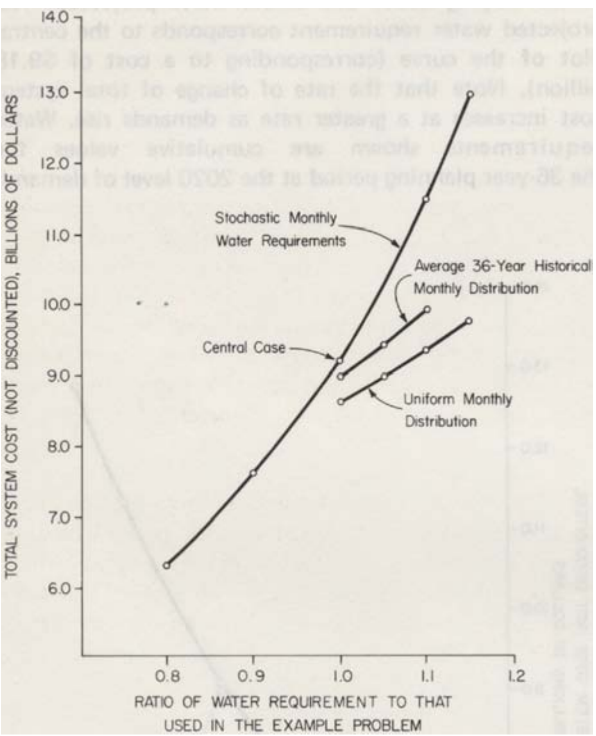



Figure 55.—Sensitivity of System Cost to Changes in Water Requirements and Their Monthly Distribution

## Response to Changes in Evaporation Data

In the Southwest, an important hydrologic variable in water system planning is the loss due to evaporation. Figure 56 depicts the change in total system cost which results from evaporation rates which are greater or lesser than those originally used as a basis for planning analyses. Evaporation rates were estimated by computing lake surface rates based on measurements of actual evaporation less rainfall which is considered to have fallen on the lake surface. The information presented in Figure 56 can be compared with that given in other plots such as Figure 54 to determine which data have the greatest bearing on system response when considering revising data for more detailed planning studies. In this example, water requirements have significantly greater influence on total system cost than evaporation. This may not be true in every case, however.

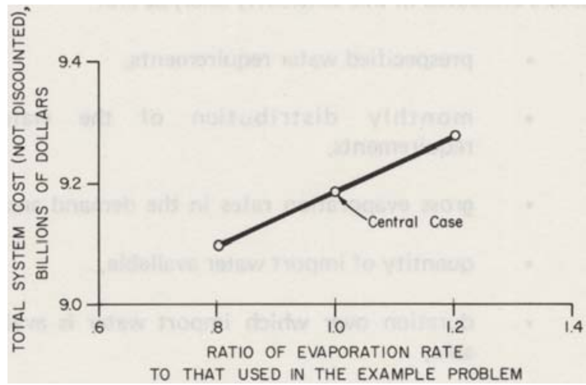



Figure 56.—Sensitivity of System Cost to Changes in Evaporation Rate

## Response to Changes in Import Availability

The example water development system at high levels of water requirement is dependent on some external source for import water. Four curves in Figure 57 illustrate the variation in total system cost which would result from import water available for various part-year durations, ranging from 2 to 6 months, with the total annual quantities varying from the minimum to maximum expected quantity available for each part-year period. A uniform distribution of import water availability was assumed for each period.

This information can be used to determine when changes in import availability increase costs to the point that revision of the entire project is in order, and to specify the worth of alternative courses of action. As indicated in the previous section, these data can be used to assess the need for improving the data concerning the availability of import water and to guide the allocation of resources to the revision of the data. Further, data such as these can be used to establish the optimal quantity and duration of import required to augment the supply available within the system.

## Response to Changes in Streamflow Data

In most planning studies it is necessary for computational expendiency to assume that the future

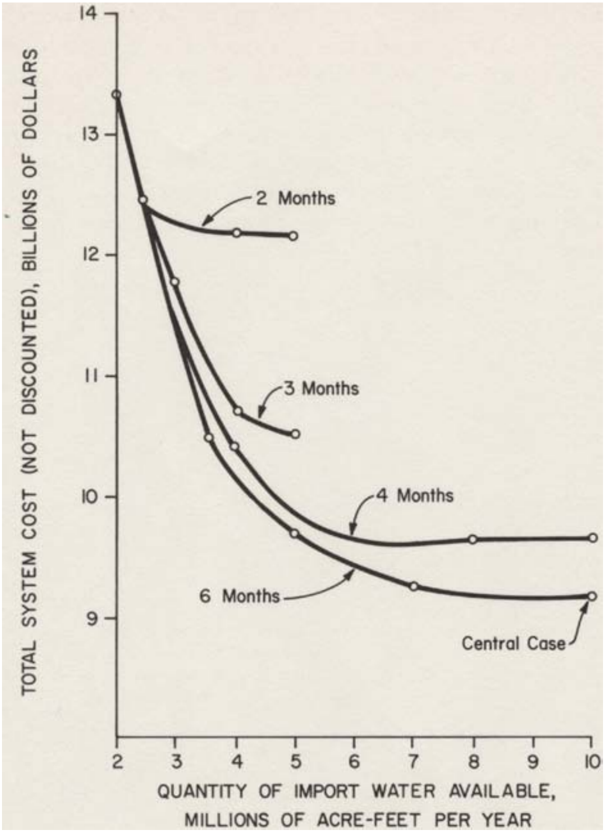



Figure 57.—Sensitivity of System Cost to Changes in Import Water Availability

streamflow conditions can be adequately represented by historical records. However, streamflow contains an inherent risk element which introduces variability into the planning process. Therefore, in the investigation discussed herein, it was decided to generate equally likely flow sequences and to study the effect of alternate sequences on system response. Initially, 100 sequences of 36-year duration were generated and a subset of 18 sequences was selected to represent the full range of possible wet and dry sequences which are possible and to exhibit the full range of critical period occurrence throughout the planning period. The monthly streamflow synthesis program of the U.S. Army Corps of Engineers, Hydrologic Engineering Center (1967), was used to generate the sequences. The method for selecting the subset of 18 representative sequences is described in Chapter VI.

A plan of development as defined previously in the section describing the physical problem was developed for the system. Then 18 separate simulations of the system were performed, using each of the 36-year sets of stochastic hydrologic input data (streamflows, demands, and evaporation rates) as input to the SIM-III simulation model described previously. A summary of the data and the system responses is given in Table 17. The table presents the mean, range, and standard deviation for the total supply and demand, for the undiscounted and present value system costs, shortage costs, and power costs, and for the total shortages and spills. Water requirements, availabilities, shortages, and spills are given in millions of acre-feet for the 36-year planning period. The requirements and costs are not comparable with previously presented information in this chapter because the data in Table 17 are for 1990 conditions while previously given data are for 2020 conditions. Considering the system responses in Table 17, a planner would not be overly concerned about the significance of the variability in the available supply and the demand. However, the variability generated in the system shortages and costs are extremely significant.

## Irrigation-Demand Sensitivities

Unit acre demands for irrigation water are computed by a computer program titled DEMAND-II.34/ The product of this computational procedure is a sequence (or sequences) of monthly irrigation demands reflecting the stochastic properties of the quantities upon which the water balance is based, e.g., rainfall and evaporation.

The sensitivity of this model is illustrated in part by Figure 58. In this figure, percent variation for the total cumulative demand over the 36-year planning horizon, 1985 to 2020, is contrasted to percent variation for the total cumulative precipitation and evaporation in the demand area. The dominant influence of evaporation is well illustrated. A change in evaporation of 20 percent is identified with a change in demand of about 40 percent, while a comparable change in precipitation induces a change in demand of roughly 11 percent.

The effect of the assumed depth of root penetration is also illustrated in Figure 58. The relative insensitivity of model response to this variable is suggested by the fact that for these conditions a 20 percent variation in root depth caused only about a 5 percent change in estimated demand.

## Firm-Yield Sensitivities

As was discussed in Chapter IV, firm-yield analysis plays an important role in an optimal water resource development plan. During that portion of the analysis, most of the reservoirs were sized and reservoir operation criteria were preliminarily specified. Therefore, it should be of interest to know how accurate the yields being computed are, and to what degree the variables such as runoff, reservoir size, reservoir surface area, and evaporation rates affect the accuracy of the firm yield. To help determine this, a set of SIMYLD-I runs were made,

34/ DEMAND-II is described in detail in Volume IID of this report.

## Table 17.—Results of Analysis of 18 Stochastic Sequences for System Response, 1990 Conditions*

| | TOTAL WATER REQUIREMENT | TOTAL WATER AVAILABLE | TOTAL SHORTAGES | COST NOT DISCOUNTED | | | PRESENT VALUE COST | | | TOTAL SYSTEM SPILLS |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | TOTAL COST | SHORTAGE COST | POWER COST | TOTAL COST | SHORTAGE COST | POWER COST | |
| High | 125.3 | 128.6 | 19.54 | 5.76 | 1.95 | 2.92 | 3.36 | .94 | 1.33 | 16.63 |
| Low | 119.9 | 122.3 | 2.58 | 4.50 | .26 | 2.48 | 2.70 | .12 | 1.12 | .63 |
| Range | 5.4 | 6.3 | 16.95 | 1.26 | 1.70 | .43 | .66 | .82 | .20 | 16.00 |
| Mean | 122.3 | 125.8 | 10.06 | 5.06 | 1.01 | 2.73 | 2.92 | .42 | 1.24 | 7.31 |
| Standard Deviation | 1.4 | 1.8 | 4.21 | .33 | .42 | .11 | .18 | .22 | .06 | 2.38 |

* Water quantities are in millions of acre-feet.
Costs are in billions of dollars.

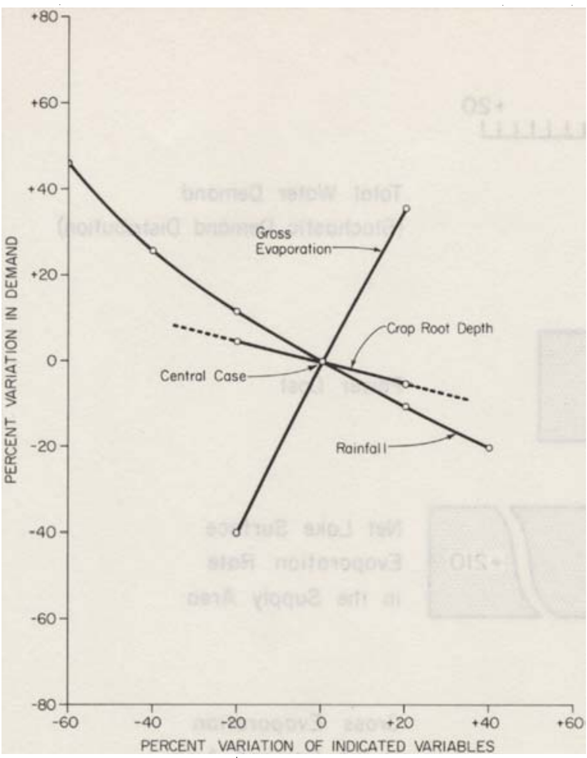



Figure 58.—Sensitivity of High Plains Irrigation Demands to Changes in Selected Variables

varying the independent variables mentioned above by up to plus and minus 20 percent. The results of that process are shown in Figure 59. The figure shows that as either storage or runoff are increased by 20 percent, a corresponding 10 to 11 percent increase in firm yield occurs. Conversely, as these variables are decreased by 20 percent the yield decreases by about 11 to 12 percent. The relationship is apparently not quite linear. Figure 59 also shows that as evaporation rates are increased by 20 percent, the firm yield decreases by about 14 percent; whereas a decreased evaporation rate of 20 percent causes only a 7.5 percent increase in firm yield. In addition, for a 20 percent increase in reservoir surface area, only a 4.0 percent decrease in firm yield occurs. This relationship is apparently fairly linear because a corresponding decrease in surface area by 20 percent causes a 4 percent increase in the computed firm yield.

## Response to Changes in Economic Information

As a part of the sensitivity studies, quantitative results were obtained concerning changes in economic information such as power cost for pumping and methods of cost discounting.

Figure 60 illustrates the effect of variation of the cost of power on the total system cost. Power cost was

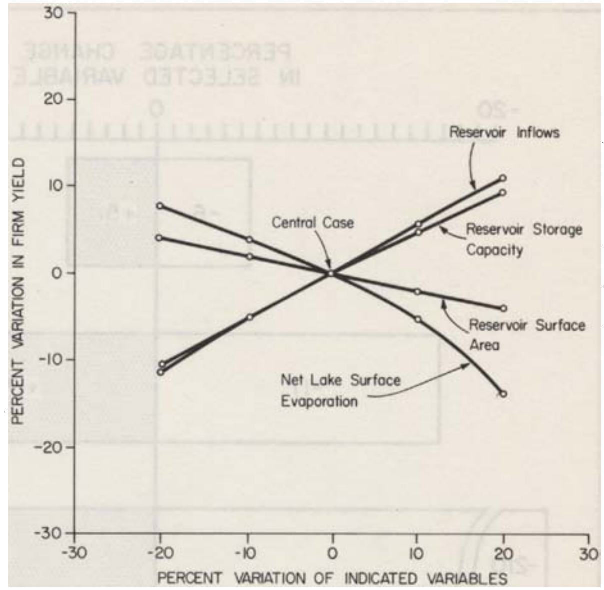



Figure 59.—Sensitivity of Northeast Texas Firm Yield to Changes in Selected Variables

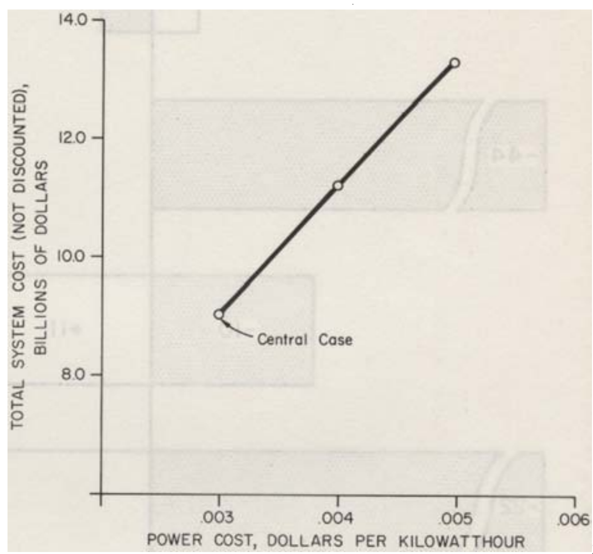
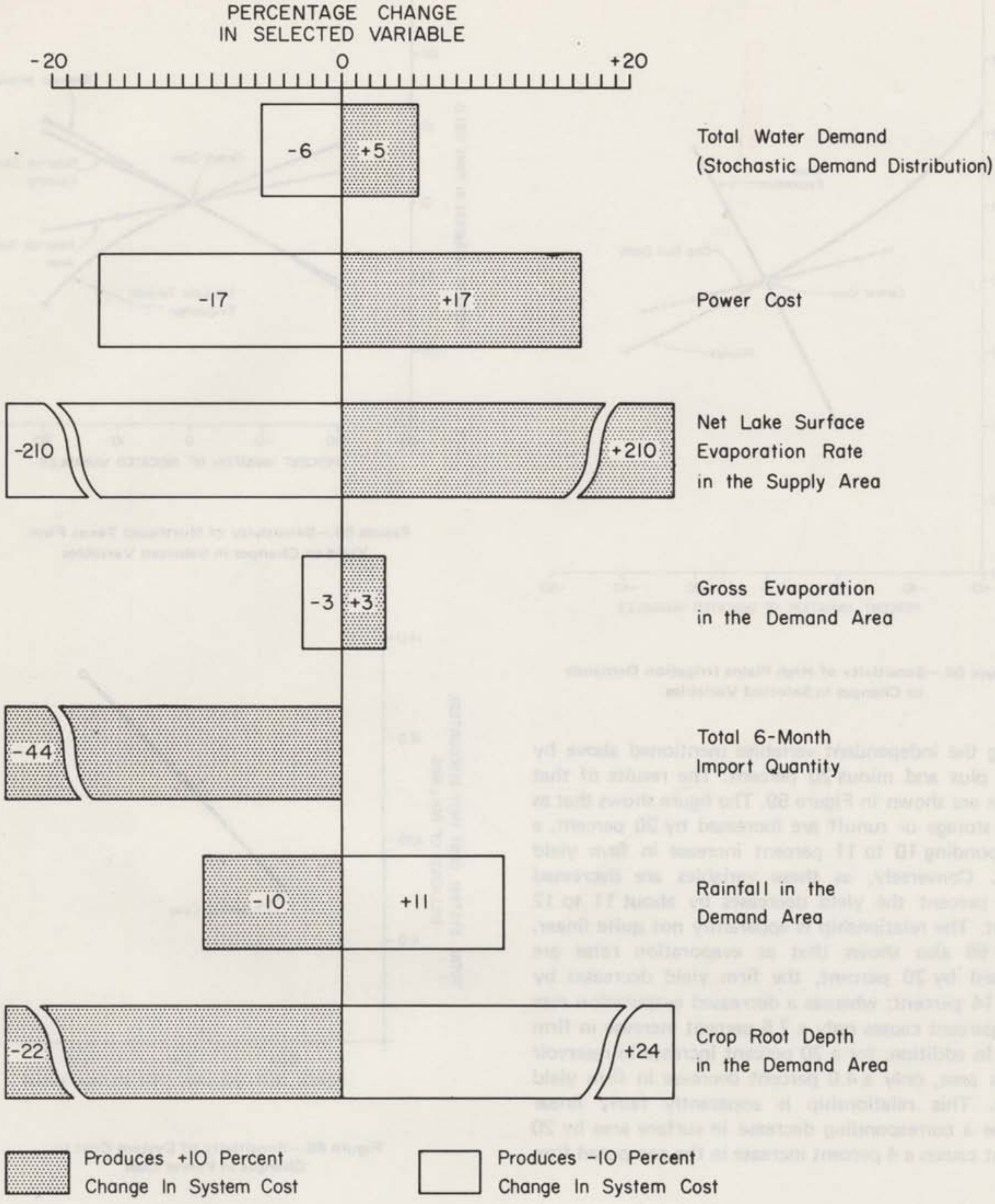



Figure 60.—Sensitivity of System Cost to Changes in Power Cost

varied from 3 to 5 mils per killowatt-hour and the total system cost resulting from a 36-year simulation was determined. As should be expected, the power cost has a direct effect on the total system cost. The magnitude of the change is important in assessing the potential effect of changes in the cost of power.

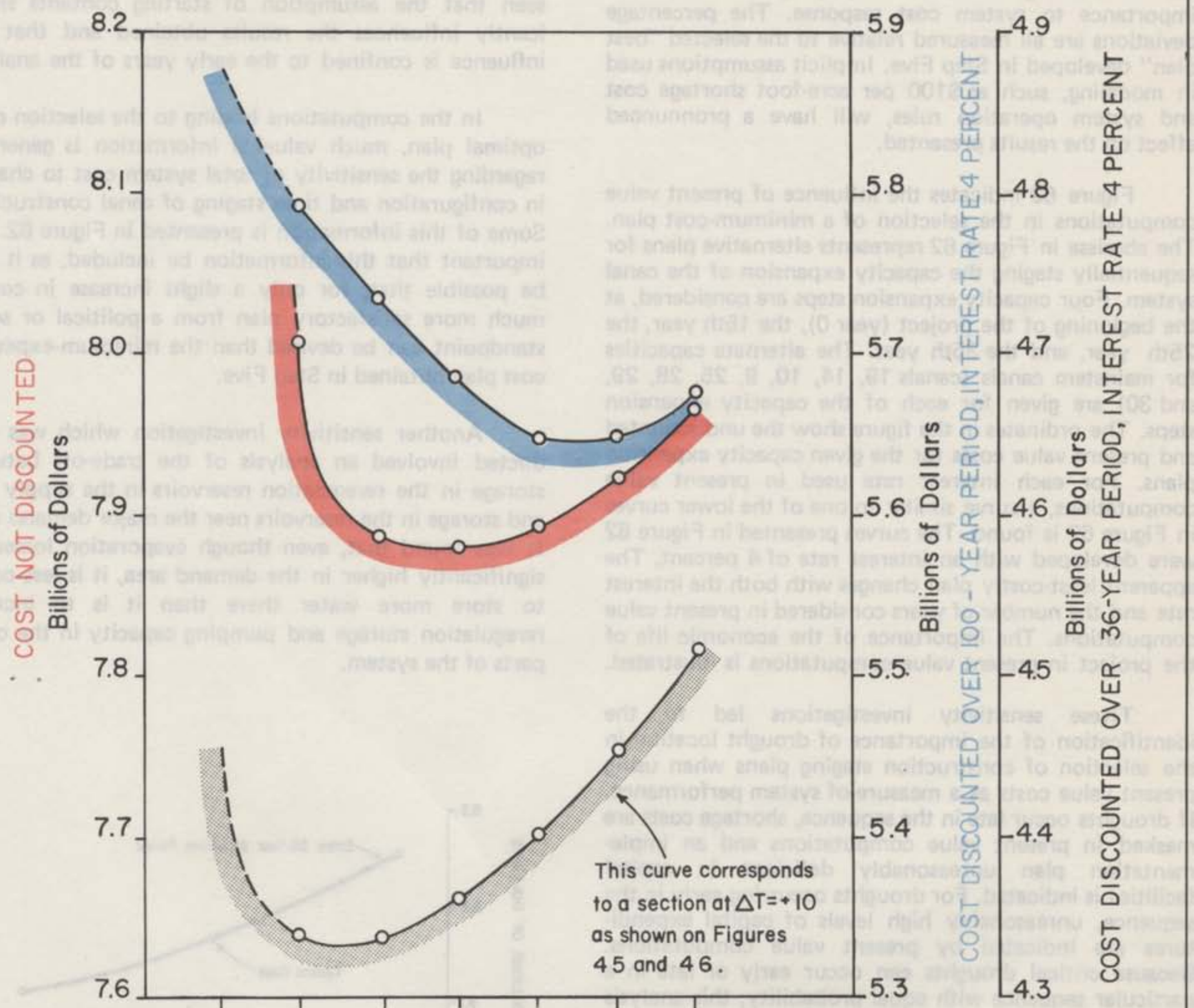
Comparative analysis of the data contained in Figure 60 and the data presented in previous figures in this chapter can provide information similar to that given in Figure 61. This figure indicates the percentage




Note: The shorter the length of the bar the greater the influence of the variable on total system cost.

**Figure 61**

**Percentage Changes In Selected Variables Which Produce A 10-Percent Change In Total System Cost**




| MAINSTEM CANAL SIZE (THOUSANDS OF CFS) | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | 4 | 5 | 6 | 7 | 8 | 9 | 10 | Initial Size, Year 0 |
| | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 2nd Size, Year 15 |
| | 14 | 15 | 16 | 17 | 18 | 19 | 20 | 3rd Size, Year 25 |
| | 18 | 19 | 19 | 20 | 20 | 20 | 21 | Final Size, Year 35 |

Figure 62

## Total System Cost as Influenced by Canal Staging and Discounting

change in selected variables required to produce a 10 percent change in total system cost. This type of information can be useful in allocating effort aimed at selectively improving the planning information base. It should be emphasized that Figure 61 indicates only the relative importance of various planning variables as measured by the sytem cost response. The length of the bar in the figure is inversely proportional to its importance to system cost response. The percentage deviations are all measured relative to the selected "best plan" developed in Step Five. Implicit assumptions used in modeling, such as $100 per acre-foot shortage cost and system operation rules, will have a pronounced effect on the results presented.

Figure 62 indicates the influence of present value computations in the selection of a minimum-cost plan. The abscissa in Figure 62 represents alternative plans for sequentially staging the capacity expansion of the canal system. Four capacity expansion steps are considered, at the beginning of the project (year 0), the 15th year, the 25th year, and the 35th year. The alternate capacities for mainstem canals (canals 19, 14, 10, 9, 25, 28, 29, and 30) are given for each of the capacity expansion steps. The ordinates in the figure show the undiscounted and present value costs for the given capacity expansion plans. For each interest rate used in present value computations, a curve similar to one of the lower curves in Figure 62 is found. The curves presented in Figure 62 were developed with an interest rate of 4 percent. The apparent least-costly plan changes with both the interest rate and the number of years considered in present value computations. The importance of the economic life of the project in present value computations is illustrated.

These sensitivity investigations led to the identification of the importance of drought location in the selection of construction staging plans when using present value costs as a measure of system performance. If droughts occur late in the sequence, shortage costs are masked in present value computations and an implementation plan unreasonably deficient in capital facilities is indicated. For droughts occurring early in the sequence, unreasonably high levels of capital expenditures are indicated by present value computations. Because critical droughts can occur early or late in a particular sequence with equal probability, this analysis led to the development of a procedure which involves computing average annual system costs before applying present value computations so that unwarranted plans are not indicated because of a particular drought location in a hydrologic sequence.

## Response to System Changes

The quantity of storage water which is assumed present in the reservoirs in the system at the beginning of the analysis period is known to influence the results obtained. If reservoirs are assumed empty at the beginning of the analysis, the system is heavily penalized by critical sequences occurring at the beginning of the record. When reservoirs are assumed full or partially full at the beginning of the analysis, early droughts have an artifically small effect on system performance. If a "warm-up" prior period of several years is used to simulate possible starting contents, the contents are dependent almost entirely on the sequence used in the prior period. Figure 63 indicates the effect of varying the ratio of assumed starting contents to total reservoir storage capacity. The upper curve indicates the total cost for the entire 36-year simulation period. The lower curve illustrates the total system cost for years 14 through 36—the final 23 years of analysis. It is easily seen that the assumption of starting contents significantly influences the results obtained and that this influence is confined to the early years of the analysis.

In the computations leading to the selection of an optimal plan, much valuable information is generated regarding the sensitivity of total system cost to changes in configuration and time staging of canal construction. Some of this information is presented in Figure 62. It is important that this information be included, as it may be possible that, for only a slight increase in cost, a much more satisfactory plan from a political or social standpoint can be devised than the minimum-expected-cost plan obtained in Step Five.

Another sensitivity investigation which was conducted involved an analysis of the trade-off between storage in the reregulation reservoirs in the supply area and storage in the reservoirs near the major demand area. It was found that, even though evaporation losses are significantly higher in the demand area, it is less costly to store more water there than it is to increase reregulation storage and pumping capacity in the other parts of the system.

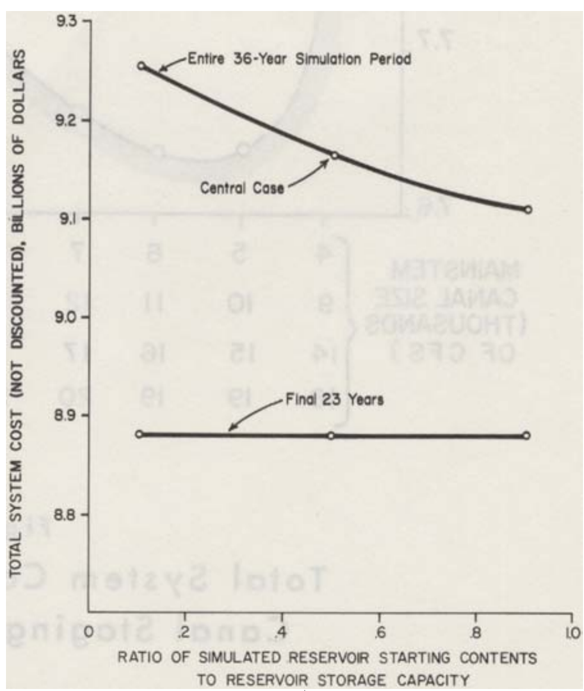



Figure 63.—Sensitivity of System Cost to Changes in Simulated Reservoir Starting Contents

# REFERENCES


Beard, L. R., 1965, Use of interrelated records to simulate streamflow: Am. Soc. Civil Engineers, Water Resources Eng. Conf. Mobile, Ala., reprint 64.

_____1969, Hydrostatistics, principles of application, *in* The progress of hydrology: First Internat. Seminar for Hydrology Professors Proc., Univ. Ill.

Charnes, Abraham, and Cooper, W. W., 1959, Chance-constrained programming: Management Science, v. 6, p. 73-80.

Chow, V. T., ed., 1964, Statistical and probability analysis in hydrologic data, part IV *of* Sequential generation of hydrologic information, *in* Handbook of applied hydrology: McGraw-Hill Co.

Chow, V. T., and Meredith, D. D., 1969a, Water resources systems analysis, part I *of* Annotated bibliography of programming techniques: Univ. Ill., Dept. Civil Eng.

_____1969b, Water resources systems analysis, part IV *of* Annotated bibliography on stochastic processes: Univ. Ill., Dept. Civil Eng.

Fiering, M. B., 1967, Streamflow synthesis: Harvard Univ. Press.

Hall, W. A., and Dracup, J. A., 1970, Water resources systems engineering: McGraw-Hill Co.

Hofschmidt, M. M., and Fiering, M. B., 1966, Simulation techniques for design of water-resource systems: Harvard Univ. Press.

Hydrologic Engineering Center, 1967, Monthly streamflow synthesis: U.S. Army Corps of Engineers.

Kane, J. W., 1967, Monthly reservoir evaporation rates for Texas, 1940 through 1965: Texas Water Devel. Board Rept. 64.

Linsley, R. K., and Frangini, J. B., 1964, Water resources engineering: McGraw-Hill Co.

Maass, Arthur, and others, 1962, Design of water resource systems: Harvard Univ. Press.

Meier, W. L., and others, 1970, Sensitivity analysis, a necessity in water planning: Sixth Am. Water Resources Assoc. Conf., Las Vegas, Nev.

Texas Water Development Board, 1968, The Texas water plan: Texas Water Devel. Board planning rept.

_____1969, Systems simulation for management of a total water resource: Texas Water Devel. Board Rept. 118.

Water Resources Council, 1970a, Principles and procedures for planning water and related land resources: Water Resources Council Task Force Rept.

_____1970b, Findings and recommendations: Water Resources Council Task Force Rept.

Weiss, A. O., and Beard, L. R., 1970, A multibasin planning strategy: Sixth Am. Water Resources Conf., Las Vegas, Nev.

Weiss, A. O., Meier, W. L., and Beard, L. R., 1971, Effective use of stochastic information with detailed analytic models: Internat. Symposium on Stochastic Hydraulics, Pittsburg, Pa.

Wilde, D. J., and Beightler, C. S., 1967, Foundations of optimization: Prentice-Hall, Inc.

Young, G. K., Orlob, G. T., and Roesner, L. A., 1970, Decision criteria for using stochastic hydrology: Am. Soc. Civil Engineers Proc., Jour. Hydraulics Div., v. 96, no. HY4.

Young, G. K., and Pisano, W. C., 1968, Operational hydrology using residuals: Am. Soc. Civil Engineers Proc., Jour. Hydraulics Div., v. 94, no. HY4.