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Colorado River Basin Modeling Studies: Proceedings of a Seminar Held at Utah State University Logan, Utah July 16-18, 1975

Calvin G. Clyde
Donna H. Falkenborg
J. Paul Riley

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COLORADO RIVER BASIN MODELING STUDIES

Proceedings of a Seminar
Held at Utah State University
Logan, Utah
July 16-18, 1975

Editors
Calvin G. Clyde
Donna H. Falkenborg
J. Paul Riley

Utah Water Research Laboratory
College of Engineering
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Logan, Utah 84322

March 1976
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Edited by
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Donna H. Falkenborg
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Sponsored by
Utah Water Research Laboratory
and
Civil and Environmental Engineering Department
College of Engineering, and
Conference and Institute Division
Utah State University

Utah Water Research Laboratory
College of Engineering
Utah State University
Logan, Utah 84322

March 1976
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This Seminar was held under the sponsorship of the Utah Water Research Laboratory and the Department of Civil and Environmental Engineering assisted by the Conference and Institutes Division of Utah State University. The proceedings was prepared by the publication team of the Utah Water Research Laboratory under the direction of Donna H. Falkenborg.

The wise counsel of the program committee consisting of Jay M. Bagley, Don Barnett, Calvin G. Clyde, Ival Goslin, and Norman Stauffer is gratefully recognized.

The substantial investment of time and effort made by the authors, panel members, session chairmen, and participants is greatly appreciated. The willingness of dedicated people to share knowledge and experience made possible the Seminar and the Proceedings.

Calvin G. Clyde
J. Paul Riley
Conference Co-Chairmen
PREFACE

Computer modeling is an important and valuable tool for water resources planning and management. In recent years many different modeling approaches and techniques have been developed. Some have been applied in the Colorado River Basin. A brief and incomplete search showed over fifty reports on modeling in the Colorado River Basin alone.

The main motivation for this Seminar was the indication that (1) much duplication of effort is occurring among Colorado River modeling studies due to lack of information and (2) much of the knowledge now available on modeling is not being effectively applied to real problems.

The overall goals and broad objectives of this conference were to provide a forum whereby management policies, existing computer modeling techniques, methodologies, and studies applied to the planning, design, construction, operation, management, and development of the water and land resources in the Colorado River Basin might be comprehensively reviewed, discussed, and analyzed, and projections of future needs and trends developed.

Specifically, the Seminar attempted to:

Provide a forum for policy and decision-makers and public officials to review, evaluate, discuss, and project the needs and applicability of modeling techniques in river basin planning and management.

Acquaint the participants with the present status and trends in computer models as they are applicable to water resource systems.

Bring a knowledge of the state-of-the-art in computer modeling studies to institutions, agencies, and individuals.

Help participants avoid or minimize duplication of efforts in future work related to the theme of the seminar.

Emphasize the importance of comprehensive systems analyses, recognizing that subanalysis of a comprehensive system through uncoordinated submodels generally is not sufficient.
Enhance, facilitate, and promote interdisciplinary com­munication in the area of water resource planning and management.

Put in perspective the contribution and importance of each discipline involved in modeling work.

Calvin G. Clyde
Acting Director
Utah Water Research Laboratory
Logan, Utah
23 January 1976
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TOTAL WATER MANAGEMENT CONCEPT
IN THE COLORADO RIVER BASIN

by

James J. O'Brien*

This conference provides a great opportunity to interchange ideas from a diversity of disciplines on matters of importance to the Colorado River Basin and the nation. As one of the tools used in total water management, mathematical models provide a more precise and common basis for communication between the numerous interests and disciplines that must necessarily participate in total water management studies and their application. By planning this conference to serve many interests, from those interested in an overview of modeling to those directly involved in the development of the models, the conference should go a long way toward broadening our understanding of modeling applications in the Colorado River Basin... a basin on the threshold of rapid change and of ever growing importance to our nation's future.

The use of Colorado River water is of concern to many people and institutions. Particularly concerned are the states which have a common bond in the basin. In fact, these states presently have several organizations such as the Committee of 14 and the Salinity Control Forum where mutual problems are tackled and solved by discussion, negotiation, and compromise in an atmosphere of interstate cooperation that is without precedent.

Furthermore, I can think of no more appropriate and timely subject for modeling than the Colorado River Basin... appropriate, because of the great complexity of factors affecting supply and demand of water in the basin... timely, because of the potential increase in depletions throughout the basin and related matter of controlling salinity increases in the lower main stem.

The water supply and demand situation in the basin and the many constraints upon that supply and demand require the best possible utilization of the water and the related land resources of the basin and export regions. Efforts to achieve the best possible utilization can only be as effective as the accuracy and completeness of information available to decision-makers who must wrestle with the tradeoffs necessary in developing the program. Indeed, the models must encompass and perform both the descriptive and prescriptive functions—accurately relating how the system works and displaying how to modify or operate the system to achieve goals. With appropriate displaying of model input and output, the potential effects of contemplated actions may be arrayed so that sound judgment may be exercised in determining the courses of action to be followed in developing and utilizing the basin's water resources.

So that I may better deal with the application of the total water management concept in the Colorado River Basin, I will first briefly describe the pertinent physical aspects of the basin.

This may be somewhat redundant for those of you who are fully familiar with the basin and I hope you bear with me. A brief description will help to get us off to an even start.

The Colorado River drainage basin is bounded on the north and east by mountains forming the Continental Divide and on the west by other Rocky Mountain ranges. The lower portion of the basin is dominated by plateaus, northwest trending mountain ranges, intervening basins, and deserts. From its sources in western Colorado, southwest Wyoming, and northeast Utah, the Colorado River travels 1,400 miles in a southwesterly direction to the Gulf of California. It drains a vast area of 242,000 square miles in the United States and 2,000 square miles in Mexico. The entire State of Arizona, major parts of Colorado, Utah, and Wyoming, and small but significant parts of Nevada, New Mexico, and California lie within the hydrologic basin.

The Colorado River Basin has been divided into the Upper Basin and a Lower Basin for the purpose of apportioning surface flows. The Upper Basin is defined as that portion of the basin drainage above Lee
Ferry, a point 1 mile below the mouth of the Paria River near the Arizona-Utah border. Annual virgin flow for the Colorado River for apportionment purposes is determined at Lee Ferry.

The surface flows of the Colorado River Basin are relied upon very heavily by the basin states and Mexico. Yet the flows are small when compared to other drainage systems. The long-term average annual virgin flow of the Colorado River at Lee Ferry is 14.9 million acre-feet while the analogous flow of the Columbia is 180 million acre-feet and that of the Mississippi is 440 million acre-feet.

Of major importance, too, are the large variations in annual flows which have ranged from 5.6 million acre-feet to 24 million acre-feet. For the period of 1931 to 1964, the annual virgin flow averaged only 12.9 million acre-feet per year.

Total demands placed upon surface flows of the basin currently exceed 12 million acre-feet per year and depletions exceed 9 million acre-feet per year.

Groundwater availability and use vary considerably in the Colorado River Basin.

Aquifers capable of significant yields are quite limited in the basin except in central and southern Arizona. The reliance upon groundwater varies accordingly.

Groundwater consumed in the basin is about 67,000 acre-feet in the Upper Basin and 3.5 million acre-feet in the Lower Basin. Some 3 million acre-feet, or 85 percent of the total groundwater use, occurs in the Gila Subbasin in southern Arizona.

The control of surface flows in the Colorado River Basin has been very extensive.

The initial major storage feature on the Colorado main stem and still the keystone of the Lower Basin control system is Hoover Dam which was completed in 1936.

Construction of other storage and diversion control features in the Lower Basin during the first half of this century resulted in virtually
complete control of the Colorado below Lee Ferry. These structures include Laguna, Imperial, Parker, and Davis dams.

Major development in the Upper Basin began in the 1950's, the most important feature being Glen Canyon Dam.

Completed in 1964, Glen Canyon Dam provided the Upper Basin the storage needed to meet downstream obligations. Other important features included Flaming Gorge, Navajo, Blue Mesa, and Morrow Point dams.

A substantial part of the surface water supply is diverted from the basin. The diversions amount to about 5 million acre-feet and ultimately could increase to about 7.5 million acre-feet. About 75 percent of these diversions are used for irrigated agriculture. Imports into the basin amount to a mere 6,000 acre-feet and are primarily for agriculture.

Despite the projected heavy demand for waters of the basin, the Colorado River system should yield a supply sufficient to meet demands for the midterm, probably until the year 2000.

In the Lower Basin, full operation of the Central Arizona Project—some 10 to 15 years from now—will bring to an end any sizable surplus in the Lower Basin.

When diversion into the CAP begins, Arizona and California will be using water to the limits of their entitlements. Nevada is expected to utilize its entitlement early in the twenty-first century. Thus, the future pattern of requirements in the Lower Basin states is firmly established.

By contrast, the future development of the Upper Basin and its water requirements are now actively taking shape. The final form will depend upon the rate at which federal and other water projects in the Upper Basin are put into operation, the rate of oil shale and coal development, the rate of expansion of municipal and industrial uses, and the amount of agricultural water rights sold for other uses.

But quantity of flow in the basin is not the only problem. More critical at present is the quality of the water in the Lower Basin.

As the waters of the basin are consumed or are used and then returned to the river system, the flows downstream of the point of diversion or return become more saline.
Natural sources of salinity such as mineral springs, marine shales, and other salt-laden geologic formations add to the problem.

The combined effect of water depletions and salt-loading has caused a rise in the salinity of the waters in the lower main stem. The concentrations of dissolved solids in this reach are approaching threshold limits for some uses and without a salinity control program the salinity is projected to increase.

While several studies to identify the contribution of salinity concentrations from various sources in the basin have been made, additional research is needed to quantify all the sources. The available information indicates the following decreasing order of contributions: (1) natural sources, (2) irrigation sources, (3) reservoir evaporation, (4) out-of-basin export, and (5) municipal and industrial sources.

Recent studies by the Bureau of Reclamation estimate total annual economic losses of about $230,000 for each part per million of future increase in salinity of the Colorado River at Imperial Dam.

The total damages attributable to salinity in the Colorado River system for 1973 were about $53 million. By the year 2000 these damages would reach about $124 million per year if appropriate control measures are not applied.

Federal, state, and local agencies and organizations are diligently working to resolve the salinity problems of the Colorado River. The enactment of the Colorado River Basin Salinity Control Act, Public Law 93-320, and related actions taken by the basin states with respect to the establishment of salinity standards under Public Law 92-500, highlight the great cooperation being experienced in solving the salinity problem.

Up to this point, I have sought to present to you an overview of the water situation in the Colorado River Basin. It should be clear that it will take water statesmanship, selfless cooperation, and intensive pursuit of programs to improve the short water supply/salinity issues of the river.

The solution of this issue leads us to the concept of total water management. This concept is viewed with such importance that the Assistant Secretary of the Interior for Land and Water Resources, Jack Horton, has proposed it as one of four cornerstones of Interior's water policy.

Total water management is a unifying concept and strategy for water and related land resources planning, development, financing, and operations.
Drainage basins and export areas are dealt with as hydrologic, economic, and institutional systems, taking account of all impacts: physical, economic, social, and environmental. The purpose is to achieve more equitable resource allocation and more efficient resource use and to permit a more appropriate distribution of costs and payment obligations among beneficiaries, users, and entities.

A principal aim of total water management is to achieve more efficient use and regulation, and improvement of the quality of the available water supply through the coordinated effort of local, state, and federal entities.

The total water management concept stresses nonstructural improvements in the use of the water and land resources where possible, although structural additions might prove necessary. The goal would, therefore, be achieved primarily by stressing items such as the following:

- Appropriate involvement of the various local, state, and federal entities having an interest in the resources;
- Coordinated use of surfacewater and groundwater supplies;
- Improved irrigation practices;
- Modifications to existing structures;
- Installation of salinity control measures; and
- Identification of potential major structural additions to be studied independently of total water management.

Such actions would be accomplished within the current and future requirements for coordinated operations of the control structures.

Management of the flows of the Colorado River Basin has become intricate and complex because of the compacts, laws, contracts, court decrees, and an international treaty governing the water allocations and use. Extensive control works have been built to meet the water regulation required by these legal commitments. A total water management program embracing systematic conservation efforts, nonstructural improvements, augmentation opportunities, and closely integrated operation
will provide a comprehensive means of developing and implementing solutions to the salinity and water quantity problems of the basin.

The scope of studies for total water management in a major river basin can well be imagined. You can readily see that such studies could take years to complete. With this in mind, Reclamation is taking the approach of attempting to provide information on the various aspects of the total water management study as soon as possible so that the various options identified can be displayed for evaluative and decision-making purposes.

It is this type of study for the Colorado River Basin which I will be relating to for the remainder of my discussion here today. The objectives of the study are three-fold and in each of these objectives I can see many opportunities for the effective use of mathematical modeling.

The first objective is to identify and analyze the changing needs of the river basin. The bulk of the development of the Colorado River Basin has taken place in the last 60 years. Just reflect for a second on the enormous changes that have occurred in that crucial time period. I am sure you will agree that needs in the basin may also be changing from those originally satisfied.

The second objective of the study is to examine the use of the basin water resources to see if onfarm practices, reservoir operations, and structures can be modified to achieve better management. The sophistication of planning for, providing facilities for, and operating an integrated water resource system has been a progressive development with far less thought of integration and maximum practical utilization during the early years of basin development. Simple, single-purpose works exist in the basin along with complex multiple-purpose works, each governed by laws and managed by agencies that have likewise evolved unevenly. Studies will be made of those criteria and structural options available which could have a beneficial impact on the utilization of the water resources. By the same token, it is probable that use of the water in applications such as irrigation and industry can be more efficient and return flow pollution reduced.
The third objective of a total water management study would be to explore alternative means of meeting changing river operation conditions and water needs, within the constraints of existing water rights and other legal and institutional parameters, and with various alternatives having some or most such constraints removed.

The body of law concerning the Colorado River Basin has evolved over many decades. It has also become increasingly complex. Modeling studies can assist in bringing the effects of these legal and institutional constraints into perspective. It is recognized, of course, that legal and institutional parameters involve many vested rights and any changes would require agreement of all affected parties.

Public participation in this study of total water management potential will rest upon the philosophy that every reasonably significant interest in the basin's water and land resources would be represented.

Ultimate involvement in the study will include federal and state agencies, water districts, and other quasi-governmental agencies, and, most emphatically, a broad spectrum of the public.

The general plan of study, procedural study details, organization, and other study prospectives are now under development. The effort will be a cooperative one integrating the diverse interests in the water and land resources of the basin. A plan of study will be developed and placed before the public for review and comment. This material will be analyzed and used as appropriate in the study.

Clearly, computer applications involving data acquisition and management, analyses and operations as they relate to the surface water, groundwater, and water use systems will be made. Derivation of the plan will involve examining the interrelated structures and physical features of the basin with the aim of optimizing the use and development of the available water supply.

The study will examine and evaluate the existing system to determine whether operations and facilities should be modified to achieve better management for today's values and objectives. Initial emphasis will be placed upon gains that could be made through application of
nonstructural measures such as irrigation scheduling, water yield management, and operations of the main river system and related operational subsystems.

Potential augmentation will be considered in the study including weather modification and the desalting of sea water and geothermal brines. These methods of augmentation are possible and, in the case of weather modification, quite promising. However, ensuring efficient use of the present supply remains a vital facet of any effort to meet increasing demands.

The prospects for importation frequently arise, but they must be viewed in the light or the constraints therein contained in Public Law 90-537 (Colorado River Basin Act). Any serious future consideration of this approach by the states involved would depend upon showing that the importation requirement is surplus to the needs of the exporting basin and that users within the Colorado River Basin are indeed experiencing shortages even though the water is being used in a beneficial, practical, and efficient manner.

Key results of the study would include a display of options for decision-makers. The display would list each option, its benefits, its detriments, constraints on its implementation, and appropriate recommendations.

The study would be directed initially at the basin-wide level. The basin-wide part of the study would deal with more general aspects such as the overall supply situation, depletions, meeting compact and treaty commitments, meeting salinity standards required for the main stem, and predicting overall basin supply needs. The subbasin part of the study would then look with reasonable detail at matters peculiar to each subbasin such as the operation of existing projects, irrigation practices, water quality problems, weather modification potential, and energy development.

The scope of opportunities for river basin modeling in a total water management program can be demonstrated by the areas in which Reclamation is presently developing mathematical models. Essentially, three
mathematical river basin models have been developed, each for a specific purpose. The first model is identified as the "Colorado River Salt Routing Model" which uses simplified tributary inflow assumptions and readily permits evaluation of salinity impacts resulting from water resource developments and salinity control works. This model was recently used by the Colorado River Basin Salinity Control Forum in arriving at proposed numeric criteria for salinity standards to be submitted to the Environmental Protection Agency. The model also provided direction for the development of the more encompassing and complex model known as the "Colorado River Simulation Model" (CRSM). The CRSM model provides data analysis along with a capability for a simulation and incorporation of alternative operating criteria. It is a sophisticated representation of the Colorado River System and is set up to analyze impacts of changes in operating criteria, effects of future developments, augmentation and other influences on the flow, including stochastic hydrology and salinity control measures. The third model known as the "Colorado River Storage Project" (CRSP) model was initially developed to incorporate detailed power and water operation criteria and later expanded to account for development and depletion variations and water quality effects. The CRSP model was recently utilized to assist in a comprehensive study to size the Yuma Desalting Plant as required by the Colorado River Basin Salinity Control Act of 1974.

Also, in the area of water quality, we are using a model that predicts the quality of return flows including the major mineral and nitrate loadings and the changes in soil chemistry resulting from irrigation. Under development are models to predict the temperature regimen of reservoirs and another to simulate alternative ways of developing and managing groundwater.

Another area in which modeling has been applied is weather modification. Weather modification technology is sufficiently developed to help increase water supplies. The present knowledge about clouds and precipitation, as well as social, legal, and environmental implications of cloudseeding, is incomplete. But there is a tremendous potential in
weather modification. Scientists conducting Reclamation's Colorado River Basin Pilot Project estimate that annual runoff could be increased by 1.3 million acre-feet through weather modification over selected areas in the Upper Basin.

In working toward the goal of total water management through cooperative effort, Reclamation has embarked on another program which in this case is intended to improve the efficiency with which irrigation water is used. This program, termed Irrigation Management Services, or IMS, was begun in 1970. Although still in the developmental stage, the IMS program is currently servicing 20 operating irrigation districts throughout the west, six of which are in the Colorado River Basin. The six districts within the basin have some 33,000 acres under the program. A mathematical model serves as an important tool in carrying out this work. All information thus far shows that when properly followed IMS brings higher profits to the irrigator and is a valuable tool in conserving water.

Another use of modeling is being made in the evaluation of the economic impact of changes in salinity levels of the Colorado River. This study, entitled "The Colorado River Regional Salinity Project," is co-sponsored by the Bureau of Reclamation and the Office of Water Research and Technology. Use of linear programming techniques have been made to ascertain estimated decreases in net profit available to farmers as a result of salinity impacts.

But the developments thus far in modeling to assist in analyzing hydrologic systems and, in particular, for assisting in total water management studies have only scratched the surface. As is shown on the agenda for this seminar, a number of different models are under development which could add additional capability to evaluating total water management options. With the subbasin and basin-wide approach to studying total water management, several types of models will be needed. For the basin-wide studies the systems models now available may be sufficient. For the subbasins studies, modifications of the basin-wide or conjunctive use models will need to be developed. Such models would then become part of the basin model.
Mathematical modeling will be vital to proper study of many other areas. For example, farm systems and management practices should and can be analyzed to identify and evaluate methods for bringing about water savings, crop yield increases, and other benefits.

Project systems can be evaluated for the potential of using closed conduits, automation, and similar changes.

River system improvements that could be explored include the potential for modification of existing dams and powerplants and of non-structural changes such as vegetation management and identify potential major additions that may be independently studied.

Other areas of potential model developments or improvements that would be desirable include:

- Reservoir evaporation;
- Reservoir salt precipitation;
- Long-term effects on water quality caused by bank storage, consumptive use, and loss criteria;
- Effects of major floods within subbasins or larger areas; and
- Salt routing from and through the diffuse sources.

Much remains to be done in developing and improving models for the Colorado River Basin. They are an essential tool in achieving better management of the water resources in the basin. The water supply of the Colorado River Basin, while modest, is being called upon to meet great and ever increasing demands; so much so that within 25 years every indication is that the supply will fall short of the demands unless conservation and augmentation are systematically implemented.

Efforts to maintain an adequate water supply and to control the salinity in the Colorado River system must be continuous and unrelenting. It is evident that the Colorado River is receiving a great deal of study. We can all gain by improved communication and better coordination of our studies in general and the modeling work in particular. In the last 20 years the growth of computer science and technology has exerted an unparalleled influence on water resource studies. This will be clearly demonstrated as the program of this conference moves forward. It
seems to me that total water management can be the unifying and directing vehicle for the computer applications involved in data collection, simulation, optimization, and operation. There are new needs and opportunities being generated for the use of water and related resources within the basin. The total water management study approach can be the focal point for interagency and public examination of existing systems to determine whether operations and facilities should be modified to achieve better management in this extremely complex river basin.
MODELING PURPOSES AND STRATEGY

by

J. Paul Riley*

Introduction

The problems of managing water resource systems are basically those of decision-making based upon considerations of the physical, biological, economic, sociological, and other processes involved. These processes are strongly interrelated and constitute a dynamic and continuous system. Any combination of these interrelated system variables yields a management solution. In recent years, the advent of electronic computers has stimulated the use of modeling analysis for planning and management of large and complex systems. In essence, the model is intended to reproduce the behavior of the important system variables of the prototype under study.

Once a prototype system is identified, the various processes in the system may be represented by either physical or mathematical models. Figure 1 indicates the two general categories of mathematical modeling as being simulation and mathematical programming. Mathematical programming is an optimizing procedure whereby a solution is sought in terms of a specific objective function. Frequently, this procedure requires considerable simplification of the real system. Simulation is an attempt to represent as realistically as possible (or necessary) the processes of the real world. Simulation by physical models has found application to many practical problems, such as the design of highway bridges and hydraulic structures. However, for complex systems such as those encountered in water resource management, mathematical simulation often proves to be the only feasible tool.

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Figure 1. Two basic model categories.
for predicting the system behavior. For this reason, this paper places emphasis on a discussion of mathematical simulation, whereby models are synthesized and solved by means of electronic computers.

Mathematical simulation is achieved by using algebraic relationships to represent the various processes and functions of the prototype systems, and by linking these equations into a systems model. Hopefully, simulation models have three basic properties: realism, precision, and generality.

Thus, computer simulation is basically a technique of analysis whereby a model is developed for investigating the behavior or performance of a dynamic prototype system subject to particular constraints and input functions. The model behaves like the prototype system with regard to certain selected variables, and can be used to predict probable responses when some of the system parameters or input functions are altered.

As illustrated by Figure 1, it is possible to employ either stochastic or deterministic techniques, or various combinations of both, in the representation of a system. The approach which is adopted is dependent upon a number of conditions including availability of information about the system and the kinds of problems which the model is required to solve. The predictive power of the model within the system response space usually will vary with the degree to which the model is stochastic or deterministic. The predictive capability of a model in terms of the physical interpretation employed in its development (stochastic to deterministic) is illustrated schematically by Figure 2.

**Some Advantages of Simulation Modeling**

Basically, computer simulation models are advantageous because of:

1. **The answers they give**
   a. Some answers just otherwise unattainable.
   b. Evaluation of a wide array of alternatives.
Figure 2. A schematic representation of the predictive capability of a model in terms of the physical interpretation employed in its development.
c. Non-destructive testing.
d. Distribution of errors and judgment variations among several coefficients.
e. Allows the synthesis of many system processes and relationships into an integrated package.

2. The questions they ask

   a. Indications are provided in quantitative terms of progress toward system definition and conceptual understanding.
   b. The relative importance of various system processes and input functions is suggested.
   c. Priorities are suggested in terms of planning objectives and data acquisition.
   d. A clear identification is required of problems and objectives associated with the system being studied.

3. The insights they provide

   a. A basis for coordinating information and efforts of personnel across a broad spectrum of scientific disciplines.
   b. Models are a very effective teaching device.

In summary, a model provides for maximum utilization of a given information base or data pool in terms of predictive capability of system performance. Each system performs with a total response space, and the greater the information base, the greater is the possibility of developing a model which accurately predicts system performance within this space.

The Process of Simulation Model Development

As already suggested, a model is an abstraction from reality, and in this sense is a simplification of the real world which forms the basis of the model. The degree of simplification is a function of both intent or planning and knowledge about the real world. Forrester (1961) pointed out that verbal information and conceptualization may be translated into mathematical form for eventual use in a computer. Therefore, the model development process should proceed essentially from the verbal symbols which exist in both theoretical and empirical studies to the mathematical symbols which will compose the model.

The development of a working mathematical model requires two major steps. The first step is the creation of a conceptual model.
which represents to some degree the various elements and systems existing in the real world. This conceptualization is based on known information and hypotheses concerning the various elements of the system and their interrelationships. In general, the conceptualizations and hypotheses of the real world of a particular study was formulated in terms of the available data. Efforts are made to use the most pertinent and accurate data available in creating the conceptual model. As additional information is obtained, the conceptual model is improved and revised to more closely approximate reality.

The second major step in the development of a working mathematical computer model is between the conceptual model and the computer or working model itself. During this step an attempt is made to express in both mathematical and verbal forms the various processes and relationships identified by the conceptual model. Thus, the strategy involves a conversion of concepts concerning the real world into terms which can be programmed on a computer. This step usually requires further simplification, and the resulting working model may be a rather gross representation of real life.

Dr. Yen T. Chow has compared the loss of information, first between the real world and the conceptual model, and second, between the conceptual model and computer implementation to filtering processes as depicted by Figure 3 (Riley, 1970). The real world is "viewed" through various kinds of data which are gathered about the system. Additional data usually produce an improved conceptual model in terms of time and space resolutions. The improved conceptual model then provides a basis for improvements in the working model. Output from the working model can, of course, be compared with corresponding output functions from the real world, and if discrepancies exist between the two, adjustments are indicated in both the conceptual model and the working model. The important steps involved in the process of model development are depicted by the diagram of Figure 4. The paragraphs which follow are devoted to a brief discussion of the steps indicated by this diagram.
Figure 3. Steps in the development of a model of a real world system.
Figure 4. Steps in the development and application of a simulation model.
Identification of objectives

Clearly, the starting place in a systems approach to water resources development would suggest a clear delineation of the different purposes and objectives in water development. What do we want to accomplish? Why engage in control and management of the resource? In the final analysis it becomes apparent that there is a hierarchy of related objectives which pyramid down from some overall human objective. For example, engineering objectives regarding storage, regulation, and distribution of water is a logical consequence and component of some higher order objectives based on human factors. These objectives are all related horizontally and vertically such that a change in objectives, criteria, and priorities at one level may require changes in others. In this sense we have a "system" of objectives which serve as guides and criteria in planning and development of the resource system itself.

There have been many instances of water development where this unified spectrum of objectives has not been appreciated. Objectives have sometimes been limited to considerations of a particular component of development projects and have not been properly integrated with the all-important human objectives. Objectives which center around building of a dam, for example, without a thorough appreciation of the ultimate social and economic objectives to be achieved by its operation have ultimately proved to be of little stimulus to the general economy. We may design and build magnificent dams and canals which are necessary to control, convey, and manage water so as to bring land under irrigation. However, if the lands to be served are inherently unproductive, or if the potential irrigator has not been trained or experienced in irrigation practices essential for sustained irrigation agriculture, or if credit and marketing problems have not been considered, we may have wasted resources in the construction of the dam without ever accomplishing the real objectives of feeding people.
System identification

The basis of system identification is the conceptual model of the real world developed through various kinds of data which are gathered about the system. In a sense, points at which the system is monitored may be regarded as being "windows" through which the dynamic operation of the real world system is observed at a particular point in space and perhaps in time.

The spacing of these observations in the space and time dimension largely determines the refinement of the conceptual model in terms of actual or real world conditions. For example, a gross conceptual model which is intended to represent the basic structure of hydrologic-biologic world is shown by Figure 5. A close examination of any one of the three major components depicted by this figure would reveal some of its internal processes, and thus lead to an improved conceptual understanding of the system. For example, a relatively detailed conceptual model of a typical hydrologic system is illustrated by the block flow diagram of Figure 6. In this diagram the blocks indicate storage locations within the system and the lines represent various processes by means of which water is transferred from one storage location to another. As the real world system is better understood, the conceptual model is adjusted to coincide more closely with the system of the real world. In this case, the filtering loss is lessened between the real world and the conceptual model, as indicated by Figure 3.

Evaluation and analysis of available data

This is one of the most important and time-consuming steps in the simulation of water resource systems. As already indicated, the data provide an understanding of the real world, and thereby establish a basis for evaluating model performance. The accuracy of predictions from a particular model are governed to a large degree by the reliability of the information on which the model is based and the accuracy of the data which are input to the model to provide the predicted output functions.
Model Formulation

Model formulation is the step between the conceptual model and the working model indicated by Figure 3. The form of the model which is used is dependent entirely upon the requirements of the problem (the objectives) and the data which are available for the study. The flow diagram of Figure 4 indicates four basic model categories, namely, distributed parameter, lumped parameter, stochastic, and deterministic.

In general terms, the mathematical representation of natural hydrologic systems may be achieved by means of either a lumped parameter model or a distributed parameter model (Chow, 1967a, b). In addition, processes within the hydrologic system may be represented by relationships which are deterministic or stochastic or a combination
Figure 6. A flow diagram of the hydrologic system within a typical drainage basin.
of the two (Figure 2). For example, a system might be represented as a lumped parameter model with stochastic processes, or as a distributed parameter model with deterministic processes. For lumped parameter models, space coordinates, or position, is neglected, and all parts of the system being simulated are regarded as being at a single point in space. On the other hand, if the space dimension is represented by various distributed points or areas within the internal space of the system, a distributed parameter model is constituted.

With reference to distributed and lumped parameter models, practical data limitations and problem constraints require that increments of time and space be considered during model design. For example, a monthly time increment might be entirely satisfactory for problems concerned with reservoir storage requirements for irrigation. However, for problems which deal with spillway design capacities, a daily, or even hourly, time increment might be needed. In addition, data, such as temperature and precipitation readings, are usually available as point measurements in terms of time and space, and integration in both dimensions is usually accomplished by the method of finite increments.

The complexity of a model designed to represent a hydrologic system largely depends upon the magnitude of the time and spatial increment utilized in the model. In particular, when large increments are applied the scale magnitude is such that the effect of phenomena which change over relatively small increments of space and time is insignificant. For instance, on a monthly time increment, interception rates and changing snowpack temperatures are neglected. In addition, the time increment chosen might coincide with the period of cyclic changes in certain hydrologic phenomena. In this event net changes in these phenomena during the time interval are usually negligible. For example, on an annual basis, storage changes within a hydrologic system are often insignificant, whereas on a monthly basis, the magnitude of these changes are frequently appreciable and need to be considered.
As time and spatial increments decrease, improved definition of the hydrologic processes is required. No longer can short-term transient effects or appreciable variations in space be neglected, and the mathematical model, therefore, becomes increasingly more complex with an accompanying increase in the requirements of computer capacity and capability.

Model verification

Computer synthesis. A computer model of a hydrologic system is produced by programming on a computer the mathematical relationships and logic functions of the hydrologic cycle. The model does not directly simulate the real physical system, but is analogous to the prototype, because both systems are described by the same mathematical relationships. A mathematical function which describes a basic process, such as evapotranspiration, is applicable to many different hydrologic systems. The simulation program developed for the computer incorporates general equations of the various basic processes which occur within the system. The computer model, therefore, is free of the geometric restrictions which are encountered in simulation by means of network analyzers and physical models. The model is applied to a particular prototype system by establishing, through a verification procedure (sometimes called validation or parameter identification), appropriate values for the "constants" of the equations required by the system.

Model calibration. A general hydrologic model is applied to a particular basin through a verification procedure whereby the values of certain model parameters are established for a particular prototype system. Verifications of a simulation model is performed in two steps, namely, calibration, or parameter identification, and testing of the model. Data from the prototype system are required in both phases of the verification process. Model calibration involves adjustment of the model parameters until a close fit is achieved between
observed and computed output functions. It therefore follows that the accuracy of the model cannot exceed that provided by the historical data from the prototype system.

Testing the model. As indicated in the previous section, model verification involves the two steps of calibration and testing. Model calibration is achieved by a fitting process which establishes the model parameters for a particular set of data from a given hydrologic unit. Model testing involves using a second and independent set of data from the same hydrologic unit, and again operating the model in order to determine the level of agreement between the observed and predicted (or computed) output functions. Thus, model testing is simply an independent test of results achieved under the calibration phase.

Model results and interpretations

The model is, of course, operated during the verification procedure, and at this time comparisons are made to test the ability of the model to represent the system of the real world. It is very possible that these tests indicate that some adjustments are necessary, either in the data on which the model is based, or in the structure of the model itself. The various options associated with this looping, or "feedback," procedure are indicated by the flow path labeled "compromises" on the diagram of Figure 4. When suitable model verification has been achieved, the model is ready for further operations involving management and sensitivity studies.

Sensitivity studies. A sensitivity analysis is performed by changing one system variable while holding the remaining variables constant and noting the changes in the model output functions. If small changes in a particular system parameter induce large changes in the output or response function, the system is said to be sensitive to that parameter. Thus, through sensitivity analyses, it is possible to establish the relative importance with respect to system response of various system processes and input functions. This kind of information is useful from
the standpoint of system management, system modeling, and the assignment of priorities in the collection of field data.

Management studies. A simulation model does not of itself produce an optimum solution in terms of management objectives. The technique does, however, facilitate a rapid evaluation of many possible management alternatives. An analytical optimizing procedure used in conjunction with a simulation model could produce system optimization in terms of a specific objective function. However, the simulation model of itself is capable of providing the water resource planner and manager with the kind of information needed to facilitate the selection of a "best" alternative from a very large number of possible choices.

Though perhaps not directly a part of the simulation or modeling process, the loop should be closed, so to speak, by the feedback of results from the implementation of the alternative selected to the initial problem situation. This suggested feedback loop is illustrated in Figure 4.

Some Examples of Computer Simulation Studies

In order to demonstrate both the general utility of simulation models and the broad scope of this approach, some examples of computer simulation studies are cited here. Most of the examples are drawn from the extensive modeling program at Utah State University and will briefly trace the development of this program. Development of the hydrologic simulation research program at Utah State University began in 1963 (Bagley et al., 1963), and has proceeded in stages to increasingly detailed models. The important underlying feature throughout the entire program has been that all of the separately described processes and phenomena are interlinked into a total system. Thus, for each model it is possible to evaluate the relative importance of the various parameters, explore critical areas where data and perhaps theory are lacking, and establish guidelines for more fruitful and meaningful study in subsequent phases of the work. Some specific studies performed under this program are mentioned in the following paragraphs.
The first hydrologic model, using monthly time measurements, gave good results for interbasin effects. The second model was designed for an investigation of in-basin problems, but still utilized a large time increment (Riley et al., 1966). Under the third phase of the program, models have been developed which simulate the hydrologic processes over small geographic units and short period of time (Riley et al., 1967; Narayana et al., 1969; and Amisial et al., 1968). Time increments for studies in this category have ranged from five minutes to a single day. A general conceptual model of a hydrologic system based upon short increments of space and time is shown in Figure 7. The hydrograph of rainfall excess is obtained by chronologically deducting the losses due to interception, infiltration, and depression storage. Routing of the rainfall excess is based on either the general continuity equation and stage-discharge relationship (Narayana et al., 1969), or by solving the unsteady state flow equations in accordance with Amisial et al. (1968). Other examples include a model which simulates the snow accumulation and melt processes over short intervals of space and time (Eggleston et al., 1970). Typical output from the programs of Narayana (1969), Amisial (1968), and Eggleston (1970) are shown by Figures 8, 9, and 10, respectively. All models were verified or calibrated on the basis of data from other events so that the agreement between the measured and simulated output functions shown by these figures represents a test of each model.

An illustration of the utility of a simulation model for a model sensitivity analysis is shown by Figure 11 (Amisial et al., 1968). This figure, which consists of computer plots, illustrates the relative sensitivity of the model to various hydrologic parameters which influence the runoff characteristics of a southwest watershed in the U.S.

Examples of the addition of other dimensions to the hydrologic components include the work of Hyatt et al., (1968) and Packer et al., (1968) in which gross salinity and economic models were superimposed upon the hydrologic model. To illustrate, typical output from the hydro-salinity model is shown in Figure 12 in which comparisons are made.
between computed and measured mean monthly inflow rates for water and salts from two hydrologic units within the Upper Colorado River drainage.

In the study by Packer et al., (1968) fundamental hydrologic and economic processes were synthesized into a single working model. A general flow chart of the total hydrologic-economic system is shown in Figure 13. With this simulation model, effects of parameter changes on any part of the system are readily observed. By utilizing the model, it is possible to estimate the marginal primary benefits of water by computing incremental changes in net return to the farm unit as the result of changes in the water supply. Cropping patterns also can be varied within the model and the resulting changes in net returns computed. Other management possibilities which might be investigated by the means of the model include water export or import alternatives with respect to the area under consideration.

Recently a general approach has been developed in which a digital computer is used to simulate the surface water-groundwater system. The model provides for detailed definition of both the surface and subsurface hydrology in terms of a grid network. Areal variations in
Figure 8. Comparison between simulated and observed runoff hydrographs, Waller Creek at Austin, Texas.
Figure 9. Outflow from subwatershed 11 for the event of July 29, 1966.

Figure 10. A comparison between computed and observed snow depths for a site at the Central Sierra Snow Laboratory, 1949-50.
Figure 11. Overland flow hydrograph for a subzone of Walnut Gulch watershed, Arizona, as affected by changes in certain parameters.
Figure 12. Computed and observed mean monthly outflow rates for water and salts from two hydrologic units within the Upper Colorado River Basin.
Figure 13. Hydrologic-economic flow system showing the production function as a link between the two systems.
hydrologic parameters, boundary conditions, vegetative distribution, and aquifer parameters are input variables at the grid nodes. The time varying responses of water table levels are obtained at each node. Typical output for a model of this nature is shown in Figure 14 (Morris and Riley, 1970). Many other practical examples can be cited which demonstrate the soundness and validity of the computer simulation approach to the operation and management of water resource systems.

Note: The number on each plot refers to the native vegetation density or that portion of the total surface area assumed to be occupied by native vegetation.

Figure 14. Observed and simulated water table levels for December 1969, Atlantico 3 Project, Colombia, South America.
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THE TECHCOM METHODOLOGY

by

Dean F. Peterson*

Introduction

The effort that led to the TechCom methodology took place because of frustrations encountered by water resource development advocates in attempting to increase federal expenditures for water development, especially under Office of Management and Budget requirements based on national economic efficiency. While economic efficiency is not a general criterion for resource allocation by OMB, progress toward some spectrum of social achievement presumably is. Jack Carlson, then an Assistant Director of OME, at a conference in Fort Collins in 1969 flatly stated that water development projects must be evaluated on the basis of their contribution to national social goals. The methodology was worked out by a seven-member Technical Committee recruited from academia, their graduate students, and various professional associates and consultants (The Technical Committee, 1971; The Technical Committee, 1974). This paper presents only a very brief summary. The serious student should refer to these reports, particularly the 1974 publications.

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1 The Senate reflected its concern in the language of the FY 1970 Appropriations Act for the Office of Water Resources Research (now Office of Water Research and Technology) which included the admonition: "... that concerted attention be given to research on opportunities for Federal-State water resource development and management to advance the nation's high priority social goals."

2 These two documents describe the logistical background, methodology and the field test in detail. They also include discussions of the method and its use in relation to philosophical and political contexts. A list of participants and reports is included as well as a
Description of TechCom

Conceptual model

The model proposed by the Technical Committee consists of an hierarchical array of elements called (social) goals, subgoals, social indicators, and action (or decision) variables. One visualizes that a change in any element of the model, in general, can effect a change in some or all of the other model elements. An expression which states a relationship between two elements is called a connective.

Structurally, nine word-described primary goals reflecting the aspirations of contemporary American society form the top layer of the hierarchy which is arranged in a treelike structure as illustrated in Figure 1.

The set of primary goals chosen by the Technical Committee consists of:

1. Collective Security
2. Environmental Security
3. Individual Security
4. Economic Opportunity
5. Cultural and Community Opportunity
6. Aesthetic Opportunity
7. Recreational Opportunity
8. Individual Freedom and Variety
9. Educational Opportunity

Admittedly, the choice of the primary goal-set is arbitrary. The rationale leading to this choice is discussed in the Phase I report and comprehensive list of references. The Technical Committee included:

C. D. Gordon, F. F. Slaney & Company, Vancouver; Marion Marts, University of Washington; Robert Roelofs, University of Nevada; Henry F. Caulfield, Colorado State University; Ralph d'Arge, University of Wyoming; Ted Roefs, Office of Water Research & Technology, Washington, D.C.; D. F. Peterson, Utah State University.

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Figure 1. Structure of Techcom System.
in other reports. One important consideration was that the set be comprehensive.

Each primary goal is defined by a finite number of word-stated subgoals. As needed, additional levels of subgoals (sub\textsuperscript{n}-goals--where n is the hierarchical level) are utilized to add needed definition to immediately superior level subgoals. For example, one primary goal (number 4 of the TechCom set) is economic opportunity. This goal is defined as (disaggregated into) the subgoals: Present living standard, future living standard, and equality of economic opportunity. The three subgoals were each further disaggregated in the fashion indicated below:

4 Economic opportunity
   41 present living standard
      411 income
      412 consumption of goods and services
         4121 prices of goods and services
         4122 quality of goods and services
         4123 selection of goods and services
      413 leisure time
      414 stability of the economy
   42 future living standard
      421 employment potential
      422 savings and investment potential
      423 retirement potential
   43 equality of economic opportunity

Goals and subgoals are not \textit{per se} measurable or measured but are concepts perceived as desirable by people and verbally expressed in abstract form. There is no suggested priority among goals or subgoals at any level.

At the lowest subgoal level, one perceives measurable (or measured) properties which collectively describe conditions relevant to the achievement of a subgoal. These variables are called \textit{social indicators}.
For example, the Technical Committee reasoned that subgoal 414 stability of the economy is described by some combination of the following social indicators:

- (1) growth rate of per capita income (percent)
- (2) rate of inflation (nationwide)
- (3) unemployment (percent)
- (4) business failures as a percent of the total number of businesses

The first digit of each index number refers to the number of the primary goal; each successive digit indicates the subgoal number; the level of the hierarchical echelon is indicated by the position of the digit in the index number. For social indicators the last digit is parenthesized. In some cases a particular social indicator may apply to more than one subgoal. Figure 1 shows a partial disaggregation of the Economic Opportunity goal.

Public actions can be expected to result in changes in social indicators and to affect, thus, the achievement or non-achievement of social goals. Such actions or policy changes are called action variables. For example, construction of a dam and reservoir will induce changes in social indicators which will probably relate to one or more subgoals under all or most of the primary goals. A similar train of effects will ensue if numerical standards for salinity are enforced by policy on the Colorado River, for example. By predicting social indicator changes for various actions considering policy alternatives one can judge the relative effects on subgoals and goals. TechCom offers a methodology for quantifying these effects.

Connectives can exist between action variables, social indicators, and subgoals within categories, or between one element of a category and one of another category, i.e., between an action variable and either a social indicator, or a subgoal or goal; or between social indicators and subgoals and goals. This relationship is illustrated in Figure 2. Connectives may be in the form of numerical coefficients, tables, graphs, algebraic expressions, or matrices. They may be formulated
from scientific, economic, or social theory, or from empirical data or a combination of these. In many cases, a degree of value judgment may be required in estimating connectives; this is bound to be the case for connectives between measured or measurable social indicators and goals or subgoals.

Field test

A substantial, however partial, test of the methodology was conducted using the Lower Rio Grande Basin in New Mexico as a test case. Rather than postulating specific preconceived water demand projections and structural responses to provide them, five alternative future development scenarios were projected. These were based on the positions reflected by principal interest groups in New Mexico and included:

1. A default plan, i.e., continue present pattern of development and water use.
2. A recreation development plan emphasizing provision of picnicking, camping, and boating facilities around extant

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3 This approach did not seem to apply to the New Mexico Rio Grande where available water supplies are essentially fully developed, i.e., no structural solution that would increase usable water supplies (other than by importation) seems apparent. One should not conclude, however, that the TechCom methodology is restricted to non-structural alternatives.
water recreation sites and cultural facilities around monuments and pueblos.

3. An industrial development plan featuring light, non-polluting industry.

4. An undevelopment plan representing a strict anti-development stance.

5. A cotton phase-out plan reflecting a possible phase-out of the cotton parity program.

Utilizing the 20-sector economic input-output model developed for New Mexico and information from banks, lending agencies, etc., growth projections were made at 5-year intervals for a 20-year period beginning in 1968 for each of the scenarios.

Because of limited time and resources, only three goals: 4. Economic Opportunity, 6. Aesthetic Opportunity, and 7. Recreational Opportunity, and part of one subgoal, 13. Health Security were examined. From the information mentioned in the previous paragraph and other sources, numerical values of 128 relevant social indicators were predicted for each scenario for each 5-year period (3,200 values in total). This work was accomplished (at the University of California, Riverside) utilizing a computerized Social Indicator Projections System. Principal efforts of the system were inversion of the input-output model for each of the 25 projections and development of algorithms to derive the projected values of the social indicators. 4

System for Quantified Planning Inquiry (SQPI)

The planner/decision-maker needs to

1. Be able to compare, hopefully with some degree of quantification, the consequences to a goal or subgoal of each of several alternative actions.

2. Be able to retrieve both the process and the information that led to the calculated index of achievement.

4 The New Mexico test was purely of the methodology and its feasibility. It is not in any sense a plan or even a comparison of plans for that area.
In manual planning efforts, a hierarchical succession of screening, condensing, and reporting loses most of the information and produces a set of successively abstract decision options. Because of the permanent information loss, this process is irreversible and cannot be adequately reviewed. Each level of decision is based on both a large measure of subjective preference and limited, often transitional, sets of information.

TechCom formalizes the planning abstraction process and preserves or even enhances its flexibility. The SQPI (a computerized system designed and implemented at the University of Arizona) permits a lead-planner/decision-maker to compare predicted social-achievement indices resulting from alternatives, and preference weightings. It reviews, in retrospective sequence, for various actions, the weights (connectives) assigned at each step in the process ending with specific social indicators and their projected values. Instead of two or three alternative evaluations set in concrete by the successive abstractions of the manual planning process, the planner/decision-maker can look at virtually an unlimited set of evaluation options. This process is particularly suited to public participation, because preferences (connectives) used by the planner can be displayed and weighted and the consequences of conflicting interest-group preferences examined either informally or through scientifically designed preference public surveys.

Discussion

Goal space

Our country's store of investment capital is allocated: 1} by the market place, 2) by political decision, 3} by some combination thereof, i.e., a politically regulated or managed market. The share that is allocated by politics continues to increase as does the impact of politics on the common market place.

Market bargaining values the goals of individuals. In political bargaining, goals of constituencies are at stake. But there must also exist in the political market place, pluralities of consensus about the
general welfare or else the political structure will fail. These areas of consensus constitute the arena of "national social goals" and it is under this rubric that most of our federally-controlled capital is allocated. This is the justification on the executive side, for the existence of that vast part of bureaucracy, particularly the Office of Management and Budget, that is involved in planning, evaluation, and bureaucratic bargaining.

The common goal space between our abstract goals and measurable indicators is a vast unknown, Figure 3. Like physical space spans matter, it is incommensurable both in terms of distance and direction and, again like physical space, it must be rapidly expanding as measured by the broadening gap between stated policy and political action. There have been two recent movements toward examining this space. One is the "social indicator" movement in which essentially ad hoc lists of measurements that might be socially relevant have been proposed. The second effort consists of two goals studies under the auspices of the Chief Executive (President's Commission on National Goals, 1960; U.S. National Goals Staff, 1970). The latter are suggested administrative policy guidelines rather than comprehensive goal statements. Beyond this, our national goals are perceived to be embodied in such all embracing, but highly abstract, terms as "general welfare," "quality of life," etc. The taxonomic efforts that led to TechCom were essentially efforts to describe the structure of this space. The particular taxon chosen was in a sense quite arbitrary. At the top, the taxonomic set was intended to be comprehensive.

5 Federal agency's missions are instrumental responses to perceived goals. Agencies become the narrow advocates of these goals.

6 The writer gains the impression that these lists have been derived essentially from subjective interest, intrinsically, rather than as efficient and meaningful measurements of a comprehensive and critically-ordered set of social objectives or goals.

7 Gr. taxi to order or classify. Taxon, sing., a particular ordering classification; taxis, pl. Taxonomy, ordering or classifying.
Regardless of the primitive nature of TechCom, the Technical Committee thinks better navigation in this unmarked space of political economy is long overdue. More rational planning must be the appropriate response. The Committee thinks that a way to approach this

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Figure 3. Goal space.
problem is to make the goal-space more tractible for decision evaluation.

Connectives

The goal-space is an idea space, word-described, with decreasing resolution at lower hierarchical levels. In the sense used by the Committee, goals are the dimensional characteristics of the goal space, not the metrics of specific targets. While the taxon chosen for describing the space was influenced by subjective preference and background just as is vocabulary, once established, it is intrinsically neutral.

By introducing connectives among the elements of the goal taxon and action- or policy-changed social indicators, the result is an evaluative mechanism in which goal and subgoal dimensions measure the objective function. If all the connectives were known, the objective function could be reduced to a single dimension, which might be called "general welfare" or "quality of life." The Committee feels that planning dialogues should center at lower levels in the goal structure where the goal metrics are less abstractly stated and there is non-animity about goal ideas. In TechCom this level can be chosen to suit the desires of the particular constituencies in debate and the timeliness of the issues under discussion.

Social indicators may be technical, like "Dissolved Oxygen," which cannot be related directly to an intrinsic subgoal, or more perceptual, like number of deaths from water hazards, which can be associated with a water safety goal dimension. Connectives are needed to related technical indicators to the perceptual ones. Sometimes scientifically-justified connectives can be formulated. More often these may have to be based on the technical judgments of experts.

Social-indicator/subgoal connectives and inter-subgoal and goal connectives apparently will have to be derived from constituency preferences. Clearly, connectives for various constituencies will be different.
In the New Mexico study, the Committee used survey techniques, including Delphi, to derive these directly. Possibly inferential techniques using budget allocations, legislative history, etc., might be developed. Figure 4 shows a Delphi-derived connective between the social indicator "Unemployment" and the subgoal "Economic Stability."

Table 1 shows public survey derived weighting coefficients connecting four subgoals to the subgoal "Present Living Standard."

Table 1. Preferences of various interest groups.

<table>
<thead>
<tr>
<th>Audience</th>
<th>General Public</th>
<th>Conservationists</th>
<th>Industrialists</th>
<th>Non-Anglo Ethnic</th>
</tr>
</thead>
<tbody>
<tr>
<td>411 Income level</td>
<td>0.29</td>
<td>0.28</td>
<td>0.26</td>
<td>0.34</td>
</tr>
<tr>
<td>412 Consumption of goods and services</td>
<td>0.17</td>
<td>0.17</td>
<td>0.17</td>
<td>0.21</td>
</tr>
<tr>
<td>413 Leisure time</td>
<td>0.18</td>
<td>0.20</td>
<td>0.22</td>
<td>0.17</td>
</tr>
<tr>
<td>414 Stability of the economy</td>
<td>0.36</td>
<td>0.34</td>
<td>0.35</td>
<td>0.28</td>
</tr>
<tr>
<td></td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
</tbody>
</table>

Application to Colorado Basin

Clearly, resource management and development policy in the Colorado Basin will have impacts which will be both substantial, and socially comprehensive at regional and national levels. These will not be contained within benefit-cost analysis--then are mostly external. Continuing the distributional politics of the past can only tragically ruin the common pasture. External imposition of ad hoc standards by fiat or coercion has some tactical merit in that it seriously raises the issues, but is about as rational a policy approach in the long run as rolling the dice. Implicitly this approach assumes there are no trade-offs, i.e., the marginal costs are either zero or are the same for all alternatives. Not only is this approach inherently inefficient, but may be internally self-canceling as well. Extension of the philosophy of water quality management designed to protect people in cities from
Y-AXIS: "ECONOMIC STABILITY" VS. X-AXIS: UNEMPLOYMENT

Figure 4. Example of first and second round Delphi determination of Q.
water-borne disease to the problem of management of salinity in the Colorado River is a thoughtless extrapolation. A TechCom or TechCom-like approach could have the advantage of insuring a substantially larger measure of social comprehensiveness in the decision process for the Colorado. It could possibly provide rational strategy guidance for planning in an arena where benefit-cost analysis is hopelessly naive, distributional politics hopelessly disastrous and ad hoc regulation hopeless inefficient.

References


MULTIPLE OBJECTIVES IN WATER AND RELATED
LAND RESOURCE PLANNING

by

Yacov Y. Haimes*

Introduction

The planning of water and related land resources in a river basin should be responsive to a diversified set of objectives and goals which are often in conflict and competition with one another. The final recommended plan (e.g. by a planning board) should account for the trade-offs among these objectives with respect to the following elements:

- time horizon: short, intermediate, and long term
- client: various sectors of the public
- scope: national, regional and local needs
- constraints: legal, institutional, environmental, social, political, and economic

There are many ways and means of identifying and classifying the objectives and goals for such a planning effort. The U.S. Water Resource Council (21) advocates four major objectives. These are the enhancement of

- the national economic development
- regional economic development
- environmental quality
- social well-being

On the other hand, Peterson et al. (19) identify these nine major goals divided into two major groups:

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Maintenance of security

- environmental security
- collective security
- individual security

Enhancement of opportunity

- economic opportunity
- recreational opportunity
- aesthetic opportunity
- cultural and community opportunity
- educational opportunity
- individual freedom and variety.

Since this paper was inspired through working with the Maumee River Basin Planning Board, the objectives identified by the Maumee Citizens Advisory Committee (CAC) for a Level-B planning effort will be discussed and analyzed here. These objectives are:

- Enhance water quality
- Protect fish and wildlife
- Enhance recreation opportunities
- Reduce flood damage
- Protect agricultural land
- Supply water needs
- Reduce erosion and sedimentation

The single objective consideration that dominated most past mathematical models has undoubtedly contributed to the present skepticism in systems modeling and optimization as applied to water resources problems. These sources of skepticism can be summarized as follows (7):

- Lack of multiobjective considerations
- Lack of proper balance between modeling and optimization (overemphasis on optimization techniques at the expense of better and more realistic models)
- Lack of interaction with the decision-makers
- Lack of adequate considerations of "soft" elements such as legal, political, institutional, and social aspects
• Lack of a total analysis of the whole system
• Lack of a follow-up study also known as post-audit
• Lack of proper planning for data.

In the case study discussed here, a continuous effort has been and is still being made to respond to the above criticisms.

__Multiobjective Analysis and the SWT Method__

Fundamental to multiobjective analysis is the Pareto optimum concept also known as a non-inferior solution. Qualitatively, a non-inferior solution of a multiobjective problem is that where any improvement of one objective function can be achieved only at the expense of degrading another objective function.

To define a non-inferior solution mathematically, consider the following multiobjective function problem also known as a vector optimization problem (e.g., the seven objectives in the Maumee River Basin discussed in the previous section):

\[
\begin{align*}
\text{Minimize } & \left[ f_1(x), f_2(x), \ldots, f_n(x) \right] \\
\text{subject to } & x \in X \\
\text{where } & x \text{ is } N\text{-dimensional vector of decision variables} \\
& X \text{ is the set of all feasible solutions} \\
& X = \left\{ x \mid g_i(x) \leq 0, i = 1, 2, \ldots, m. \right\}
\end{align*}
\]

Definition: A decision \( x^* \) is said to be a non-inferior solution to the problem posed by the system (1), if and only if there does not exist another \( \bar{x} \) so that \( f_j(\bar{x}) \leq f_j(x^*), \ j = 1, 2, \ldots, n, \) with strict inequality holding for at least one \( j \).

The Surrogate Worth Trade-off (SWT) method is used to analyze and optimize the multiobjectives planning problem. A detailed discussion of the SWT method is available elsewhere (10, 11, 16) and therefore only a brief summary of it is presented here:

• The SWT method is capable of generating all non-inferior solutions to a vector optimization problem.
• The method generates the trade-offs between any two objective functions on the basis of duality theory in nonlinear programming. The trade-off function between the \(^i\) th and \(^j\) th objective functions, \(\lambda_{ij}\), is explicitly evaluated and is equivalent to \(-\frac{\delta f_i}{\delta f_j}\).

• The decision-maker interacts with the systems analyst and the mathematical model at a general and very moderate level. This is done via the generation of the Surrogate Worth functions, which relate the decision-maker's preferences to the non-inferior solutions through the trade-off functions. These preferences are constructed in the objective function space (more familiar and meaningful to the decision-makers) and only then transferred to the decision space. This is particularly important, since the dimensionality of the objective function space. These preferences yield to an indifference band where the decision-maker is indifferent to any further trade-off among the objectives.

• The SWT method provides for the quantitative analysis of non-commensurable objective functions.

• The method is very well suited for the analysis and optimization of multiobjective functions with multiple decision-makers (16).

• The method has an appreciable computational advantage over all other existing methods when the number of objective functions is three or more.

• For a review and evaluation of multiobjective programming techniques, the reader is referred to the work of Cohon and Marks (2).

The Maumee River Basin Planning: A Case Study

Background

The Maumee Basin had a population of approximately 1,520,000 in 1970, nearly 20 percent of which was located in three Standard...
Metropolitan Statistical Areas (SMSA). The Maumee study area contains approximately 8,981 square miles (5,748,000 acres). The basin is divided into six planning subareas (PSAs) by county boundaries, of which 58,400 acres are water surface (see Map, Figure 1). Most of this land is nearly level to very gently sloping, with local areas of moderately sloping relief among streams and on glacial moraines. The heart of the basin, between Toledo (in Ohio) and Ft. Wayne (in Indiana) was formerly an old glacial depression known as the Great Black Swamp. Soils, for the most part, have developed into fine textured glacial fills and lake-laid clays. They are slowly to very slowly permeable and have poor to very poor natural drainage. These conditions, together with intensive row cropping, produce substantial runoff during heavy rainfall, causing sheet erosion. About 85 percent of the 6,919 square miles (4.4 million acres) within the hydrologic boundaries are used for agricultural purposes. Of this amount, about 3.96 million acres are used as cropland, 0.13 million acres as pasture, 0.37 million acres as woodland, and 0.15 million acres as miscellaneous agricultural land. Urban, transportation, and other non-agricultural uses occupy 0.4 million acres and the remaining area is classified as miscellaneous use or as water surface.

The major problems in the Maumee River Basin arise from the intensive use of the natural resource base, the degradation of natural habitat, and current patterns of land use. For more information on the basin, the reader is referred to the Great Lakes Basin Commission reports (3).

Hierarchical modeling

Four major sources of complexity arise in attempting the modeling task for the Level-B planning problem discussed previously. These sources, which are inherent in all regional water resource problems and due to the coupling in the system (14), are listed below:

- Temporal Coupling: The planning time horizon in this study spans the period from 1975 to 1990. The dynamic changes in
Figure 1. Maumee River Basin.
the demographic, economic, hydrologic, and other elements should be accounted for. In this study, three 5-year periods were considered.

- Political-geographical Coupling: The basin was divided into six major planning subareas (PSA) based on Standard Metropolitan Statistical Areas (SMSA), which in turn are based on county lines. These county and three state lines cross hydrologic boundaries.

- Hydrological Coupling: The Maumee River Basin is composed of eight hydrologically distinct sub-basins which cross SMSA boundaries.

- Functional Coupling: The seven major identified objectives (flood, recreation, sedimentation, etc.) are coupled within each other so that enhancing one objective affects all others.

Clearly, each of the above classes of coupling provides a basis for a different system decomposition with a corresponding hierarchy of models. Figure 2 depicts such a hierarchy of two layers, where the first is the decomposition layer and the second is the coordination layer. The first layer is composed of two levels. The second level constitutes the six PSAs based on the geographical-political decomposition. The first level constitutes the seven objective functions in the planning study based on the functional decomposition. The second layer is the overall hierarchical coordination layer where the SWT method is applied for that purpose. The temporal and hydrological coupling are analyzed implicitly.

Other hierarchical structures are possible and their choice depends on the specific needs and goals of the systems analyst as well as on the type and availability of data. For a detailed discussion on decomposition and multilevel approach as applied to water resources systems planning, the reader is referred to Haimes and Haimes et al. (2, 4, 5, 6, 8, 9, 13, 14, 15, 17, and 18). Overlapping coordination between two or more hierarchical model structures is discussed in reference (14). Such two structures may be a hydrological decomposition
Figure 2. Hierarchical modeling for level-B river basin planning.
and a functional decomposition at the second and first levels respectively in one structure where the hierarchical model structure given in Figure 2 represents the other.

In the case study discussed here four major phases in modeling have been experienced:

1. At the first phase a single objective oriented model was developed for each of the seven objective functions as applied to one specific planning subarea (i.e. PSA 5).

2. At the second phase a gradual integration of these single objective oriented models took place where the final product was a planning subarea multiobjective integrated model for PSA 5. This model was calibrated and analyzed with data for PSA 5.

3. At the third phase the subarea multiobjective integrated model was modified, calibrated and tested for its applicability to all other planning subareas.

4. At the fourth phase an overall multiobjective integrated model will be constructed, calibrated and tested for the entire river basin. This phase is in the process of being implemented.

In developing the various functional relationships (cause and effect) a linear function was assumed for simplicity whenever there was incomplete information or data. These first-order linear approximations will be improved with the acquisition of additional information, data, and experience.

To simplify the presentation here, only a qualitative presentation of the final version of the submodels and the planning subarea multiobjective integrated model is discussed. These are:

- Point Source Pollution submodel
- Land Use submodel
- Stream Quality submodel
- Water Supply submodel and
- Planning Subarea Multiobjective Integrated model
The analysis of outdoor recreation, flood plain management, and wildlife is imbedded in the Land Use submodel. The time domain in the analysis is considered in three discrete periods: the first, 1975-1980; the second, 1980-1985; and the third, 1985-1990. Four pollutant constituents are considered in the analysis. These are sediment, phosphorus from point sources, phosphorus from distributed sources, and BOD load.

Model description

Point Source Pollution Submodel. In this model the planning for construction and/or capacity expansion of wastewater treatment plants is considered. The dynamic planning assumes a continued growth of waste production due to both population and industrial growth in the region. The model is also capable of handling the changes in stream quality standards as may be imposed in compliance with P. L. 92-500. This part of the capacity expansion algorithm is based on a dynamic programming model developed by Haimes et al. (15, 18, 1). In short, the objective of this dynamic planning model is to determine the most economical expansion schedule for the wastewater treatment plants in the region so that the increasing wastewater treatment demand is satisfied.

Each planning subarea (PSA) in the Maumee River Basin is divided into a number of reaches. The Streeter-Phelps equation is solved to determine the demand in dissolved oxygen at each reach due to discharges of effluents upstream. An application of the SWT method to water quality management is presented by Haimes and Hall (12).

Land Use Submodel. Major consideration is given to the activities of the agricultural sector in the Maumee River Basin due to the fact that over 80 percent of the basin is agricultural. Erosion and sedimentation are major concerns to this Level-B planning. In this model it is assumed that soil sedimentation and accompanying phosphorus eroded from agricultural lands are transported to the basin streams and hence to the Maumee Bay. The basic analysis in this submodel is based on the MORE (Multiple Objectives Resource Evaluation) system developed by the
Economic Research Service of the Department of Agriculture (12). The MORE system is essentially a linear programming (LP) model where the objective function is the cost of various land management practices and the constraints are different levels of sedimentation due to given levels of agricultural activities. This MORE-LP model was originally solved parametrically where changes in the sedimentation and crop yield levels are studied.

The present Land Use submodel adds to both the cost function and the constraints of the MORE system where the three time periods are also incorporated into the model. The present constraints are as follows:

- Environmental output constraints including sedimentation and phosphorus.
- Production response output constraints including various crop productions.
- Outdoor recreation and constraints where various land based recreational activities are considered.
- Flood plain management constraints.
- Wildlife constraints where various levels of hunting and other activities are considered.
- Land availability use constraints where the total land available in any specific PSA is limited.

This model thus derives the trade-offs among the following major objectives:

- Increase crop production
- Reduce sedimentation and phosphorus runoff
- Enhance land-based outdoor recreation opportunities
- Enhance wildlife habitat
- Reduce flood damage

This Land Use submodel draws the needed functional relationships and coefficients from several other linear programming submodels not discussed here.

Stream Quality Submodel. This submodel essentially integrates the two above submodels where the overall cost of point and nonpoint
sources of pollution control are augmented as well as the contribution of phosphorus from both of these sources.

Water Supply Submodel. The purpose of this model is to determine the optimum quantities of water conjunctively used from ground and surface water sources in the basin to meet projected water needs. These needs are based on OBERS Series E Projections to meet the growing demands of water for agricultural, domestic and industrial use. This can be done by constructing new surface water and groundwater projects in a sequence over the planning period, which results in a minimum sum of capital and variable operation and maintenance costs by using the same dynamic programming model discussed in the Point Source Pollution submodel section. The proposed construction and expansion projects are drawn from a complete set of feasible groundwater and surface water projects so that the total utilization of all these projects lies within the limitation of hydrologic resources of the basin.

Overall Multiobjective Integrated Model. This model integrates all above four submodels, where all seven original objectives identified by the CAC have been accounted for in these submodels.

Note that the first step in the SWT method is the conversion of the multiple objective formulation into the $\varepsilon$-constraint formulation where one objective is kept as a primary one and all the rest are viewed as constraints. The $\varepsilon$-constraint formulation provides for the generation of all non-inferior solutions as well as the corresponding trade-off functions.

The PSA multiobjective integrated model is already presented in $\varepsilon$-constraint form. The primary objective function is composed of all the above submodels' cost functions. The $\varepsilon$-constraints formulation includes constraints related to sedimentation, BOD and phosphorus, flood control, outdoor recreation, wildlife, and water supply.

Solving this integrated model lies within the capabilities of the SWT method. This model has been programmed on the UNIVAC 1108 where various (Pareto optimal) alternative plans and their associated
trade-offs can be generated. This model can be used for both simula-
tion (answering 'what if' questions) as well as for optimization purposes.

Epilogue

The purpose of this paper was not to provide a quantitative pre-
sentation of the mathematical models developed for the planning study, 
but rather a qualitative discussion of these models and the modeling 
process that takes place. Detailed analyses of all the submodels and 
results from the application of the Surrogate Worth Trade-off method 
will be available in subsequent reports on this on-going project. Readers 
who are interested in obtaining further information on this project are 
encouraged to contact the author.

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References


Introduction

This paper briefly outlines our on-going work in structuring and evaluating optimizing mathematical models for scheduling investment for salinity control in the Colorado River. We wish to emphasize that this is only an indication of work in progress and does not present final results.

This study considers the problem of scheduling investment in salinity control projects on the Colorado River. Each project offers the possibility of preventing an amount of salt from entering the river. Diversions (consistent with "present modified" flows) are made along the river for various users. These users (primarily lower basin) incur damages as river salinity rises with time due to intensifying use upstream. The problem is to find that schedule of investments which minimizes the discounted sum of project investment and operating costs, and downstream salinity damages over time, subject to optional equity-oriented restrictions on (1) financing arrangements and (2) quality at selected points along the river.

Note that while quality restrictions are mentioned, they are optional. Hence this is more than a cost-effective analysis. Rather, the approach taken is to let the salinity profile of the river be determined endogenously within the model, with the optimal timing of investment based on the dynamic trade-off of investment costs and associated downstream damages.

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Some Details

At the beginning of the study period, a set of base (modified) conditions prevails. This means the average (present modified) annual flows and salinities of the upper mainstream Colorado, San Juan, Green, and San Rafael Rivers, as well as man-caused diversions (for agriculture, power plants and M & I), are known. The upstream diversions are assumed to increase over time (up to their legal limit) according to some predetermined schedule. This generally causes a concentrating of salts in the lower river. The salinity level at each important point of the river, indexed by j, is projected over all time periods, t, t=1, 2, ..., T, given that noncontrol projects are built. Diversions to users (see Table 1) below Lake Mead are also projected over time. The damages to the user at quality point j in period t, as a function of salinity, may therefore be expressed in terms of tons of salt, since the flows at j are assumed as given and constant in each period. Based on a preliminary investigation, the assumption that salinity control projects do not materially affect flow in the river seems justified.

Table 1. List of users (j's) downstream of Lake Mead.

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Imperial Valley I.D.</td>
</tr>
<tr>
<td>2</td>
<td>Coachella Valley I.D.</td>
</tr>
<tr>
<td>3</td>
<td>Palo Verde I.D.</td>
</tr>
<tr>
<td>4</td>
<td>MWD</td>
</tr>
<tr>
<td>5</td>
<td>Colorado River Indian Reservation</td>
</tr>
<tr>
<td>6</td>
<td>Other Yuma County Farms</td>
</tr>
<tr>
<td>7</td>
<td>Salt River Project (CAP)</td>
</tr>
<tr>
<td>8</td>
<td>Gila Project</td>
</tr>
<tr>
<td>9</td>
<td>Central Arizona Service Dist. M &amp; I</td>
</tr>
<tr>
<td>10</td>
<td>Lower Main Stem Service Area M &amp; I</td>
</tr>
</tbody>
</table>

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Work to Date on Salinity Management Models

An excellent summary of on-going work on Colorado River Salinity management models is contained in the U.S. Bureau of Reclamation Status Report Colorado River Water Quality Improvement Program (10). The Bureau has developed an accounting salt routing model ("River Network Salt Routing Model"--also known as Ribbens' Model) and a synthetic hydrology model ("System Simulation Model"). Both relate flow and salt inputs to quality downstream; the former on a deterministic basis, and the latter on a stochastic basis. We have also developed a simplified version of Ribbens' Model since it is necessary to have a rapid, easy-to-use routine for evaluating the impact of salinity control projects at various points on the river. Although this model sacrifices some of the detail of Ribbens' Model, its results are very close to those of Ribbens'.

Work by other researchers is in progress on the estimation of salinity damage functions for Los Angeles and Imperial Valley (1). These are for "direct" net disbenefits. Other work is in progress using input-output models to estimate secondary salinity impacts (6). These damage functions are:

... to be attached to the Colorado River simulation model in order to ascertain the economic impact of various management alternatives, salinity controls schemes, water resource development projects, and selected scenarios of future basin conditions. (1, p. 35).

With regard to control project costs, work is progressing on costs associated with salinity-related irrigation management and other
on-farm management techniques (7, 9, 11). The Bureau is developing data and cost estimates for the point and diffuse source control projects given in Table 2.

Table 2. List of major controls. a

<table>
<thead>
<tr>
<th>Gross Salt Load</th>
<th>Controllable Salt Load b</th>
</tr>
</thead>
<tbody>
<tr>
<td>(Thousand tons per year)</td>
<td>(Thousand tons per year)</td>
</tr>
<tr>
<td>1. Paradox Valley</td>
<td>200</td>
</tr>
<tr>
<td>2. Grand Valley</td>
<td>600</td>
</tr>
<tr>
<td>3. Gunnison (Lower)</td>
<td>1100</td>
</tr>
<tr>
<td>4. Las Vegas Wash</td>
<td>208</td>
</tr>
<tr>
<td>5. La Verkin Springs</td>
<td>109</td>
</tr>
<tr>
<td>6. Palo Verde Irrigation District</td>
<td>148</td>
</tr>
<tr>
<td>7. Colorado River Indian Reservation</td>
<td>30</td>
</tr>
<tr>
<td>8. Uinta Basin</td>
<td>450</td>
</tr>
<tr>
<td>9. Glenwood-Dotsero Springs</td>
<td>500</td>
</tr>
<tr>
<td>10. Big Sandy River</td>
<td>180</td>
</tr>
<tr>
<td>11. McElmo Creek</td>
<td>130</td>
</tr>
<tr>
<td>12. Price River</td>
<td>240</td>
</tr>
<tr>
<td>13. San Rafael River</td>
<td>210</td>
</tr>
<tr>
<td>14. Dirty Devil</td>
<td>200</td>
</tr>
<tr>
<td><strong>Totals</strong></td>
<td><strong>4305</strong></td>
</tr>
</tbody>
</table>

a Taken from reference 10.

b If the i th "project" is built, this much salt could be kept from the river.

c Does not include Gunnison.

The Need for an Optimizing Investment Planning Model

Attaching damage and cost functions to a simulation model and conducting an "intuitive" search of possible investment policies is an acceptable first cut at salinity management. However, it is likely to overlook many attractive alternatives, especially where there are many
timing and sequencing possibilities for projects. Instead, we present here a framework for identifying "globally" optimum investment policies, subject to constraints as necessary. In other words we find the investment policy which is "best" within the context of the problem definition. The underscored words are very important, for it is here that political and institutional reality are reflected in the set of feasible alternative control options available for searching. In other words, we do not seek "the" optimal policy, but a set of policies that are best, subject to various constraints (a set of parametric results).

We realize that the desirable detail and informational content of a stochastic model cannot be incorporated in an optimizing framework of the type presented below. In this regard, our regional optimizing model should be considered as a "screening model" to find sets of policies which can then be explored in more detail using the Bureau of Reclamation's simulation model.

The Investment Timing Models

The major contribution of this project is to formulate and evaluate models for scheduling control projects. We are considering two different approaches to this task. One uses a mixed-integer mathematical programming model, and the other exploits the recursive notion of dynamic programming. Each has its characteristic advantages and disadvantages, and a goal of the research is to compare and test the suitability of the approaches for planning salinity control. We discuss both below.

The Mixed-Integer Programming Model

We present two basic versions of the mixed integer programming model here. One is for the case where control projects are represented by one-time-only zero or one decisions. In other words, the project
can be built at a single predetermined (salt removing) capacity, and once built, cannot be enlarged. This is the discrete version. The second model assumes projects of various sizes could be built (capacity is a continuous variable) at once or in installments. This is the continuous version. We present these in this order, following some fundamental assumptions.

Assumptions:

1) Salinity at the \( j \)th point on the river during period \( t \) if no projects are built can be estimated using an accounting model.

2) Removal of a ton of salt by the \( i \)th control project (at \( i \)) would result in a reduction of \( \beta_{ij} \) (\( 0 \leq \beta_{ij} \leq 1 \)) tons at point \( j \).

3) The time lag until the reduction occurs at \( j \) is negligible, relative to the length of time periods in the model.

4) There is no interaction among control projects vis-a-vis salinity reduction at \( j \).

5) The reduction in flow caused by salinity control projects is negligible compared to mainstream flow.

6) Damages from salinity at \( j \) in period \( t \) are independent of salinity at \( j \) in previous periods.

7) Damages may be related to "salinity" as expressed in terms of Total Dissolved Solids.

Define:

\[ u_j \] = set of available control projects upstream of quality point \( j \) (a "quality point" is, for example, the inlet to Colorado River Aqueduct).

\[ T_i \] = tons of salt that could be withheld from river if "project \( i \)" is built.

\[ T_{N_jt} \] = Total tons of salt in river at quality point \( j \) if no control projects built.

\[ S_{jt} \] = Total tons of salt in river at quality point \( j \) in period \( t \) (since river flows are known and constant during a particular period, \( S_{jt} \) is equivalent to the salinity level).

\[ S_{jt} \] = maximum allowable tons of salt at point \( j \) in period \( t \), a politically determined constraint.
\( K_{it} \) = cost of constructing project \( i \) in period \( t \), including an allowance for the present value of operating and maintenance costs.

\( X_{it} \) = 0 or 1; indicates period in which project built.

\( Q_{jt} \) = flow in river at point \( j \) during period \( t \).

\[ D_{jt}(\frac{S_{jt}}{Q_{jt}}) = \$ \text{ damages to } j^{th} \text{ user, a function only of } S_{jt}, \text{ because } Q_{jt} \text{ is a known a priori.} \]

\( \alpha = \) inflation-compensated discount factor.

\( m = \) number of projects.

\( n = \) number of quality points.

**The discrete model**

\[
\min \sum_{t=1}^{T} \alpha^t \left( \sum_{i=1}^{m} K_{i} X_{it} + \sum_{j=1}^{n} D_{jt} \left( \frac{S_{jt}}{Q_{jt}} \right) \right) \quad \ldots \quad (1)
\]

s.t. \[
\sum_{t=1}^{T} X_{it} \leq 1 \quad \forall i \quad \ldots \quad (3)
\]

\[
X_{it} = 0, 1 \quad \forall i, t \quad \ldots \quad (4)
\]

\[
S_{jt} \geq 0 \quad \forall j, t \quad \ldots \quad (5)
\]

Since damages are assumed to be convex in salinity, it will be necessary to make piece-wise linear approximations to the functions \( D_{jt}(\frac{S_{jt}}{Q_{jt}}) \).

**The continuous model**

Several of the salt control projects could be built at one level and enlarged over time. For example, Grand Valley's canals could be lined in several stages. For these projects we assume a plot of total cost vs. tons removed would have the form illustrated in Figure 1.
This could be approximated using a fixed charge and strictly convex piecewise linear function, as shown in Figure 2.

These cost relationships may represent either of two possible cases. In the first, projects of continuous capacity may be built, but the project size is fixed when constructed with no subsequent increment in salt removal capacity. To model this case, define:

- \( Y_{it}^\ell \) = \( \ell \)th segment of the piecewise salt removal curve
- \( C_{it}^\ell \) = cost associated with each unit of \( Y_{it}^\ell \)
- \( M_i \) = maximum salt that can be removed at i.
The relationships defining the model then become:

\[
\begin{align*}
\min_{t=1}^{T} & \quad \alpha \left( \sum_{i=1}^{m} \sum_{j=1}^{n} C_{it} Y_{it} + \sum_{i=1}^{m} X_{it} K_{i} + \sum_{j=1}^{n} D_{jt} \left( \frac{S_{jt}}{Q_{jt}} \right) \right) \\
\text{s.t.} & \quad T_{jt} = \sum_{t=1}^{T} \sum_{i \in u_j} \beta_{ij} \sum_{t} Y_{it} = S_{jt} \quad \forall j, t \quad \ldots \quad (6) \\
& \quad \sum_{t} Y_{it} \leq M_{i} X_{it} \quad \forall i, t \quad \ldots \quad (7) \\
& \quad 0 \leq Y_{it} \leq Y_{i} \quad \forall i, t, \ell \quad \ldots \quad (8) \\
& \quad \sum_{i=1}^{T} X_{it} = 1 \quad \forall i \quad \ldots \quad (9) \\
& \quad X_{it} = 0, 1 \quad \forall i, t \quad \ldots \quad (10) \\
& \quad S_{jt} \geq S_{jt} \geq 0 \quad \forall j, t \quad \ldots \quad (12)
\end{align*}
\]

In case 2, additional increments in salt removal capacity may be added in later periods for the additional cost indicated in Figure 2. We have for the cost in the first period at the \( i \)-th project:

\[
\sum_{\ell} C_{i1}^{\ell} Y_{i1}^{\ell} + X_{i1}^{\ell} K_{i1}^{\ell}
\]

\[
0 \leq Y_{i1}^{\ell} \leq Y_{i1} \quad \ldots \quad \ldots \quad \ldots \quad (13)
\]

For the second period,

\[
\sum_{\ell} C_{i2}^{\ell} \left( \sum_{k=1}^{\ell} Y_{i2}^{k} - \sum_{k=1}^{\ell} Y_{i1}^{k} \right) + X_{i2}^{\ell} K_{i2}^{\ell} \quad \ldots \quad (14)
\]

where:

\[
Y_{i1}^{\ell} \leq Y_{i2}^{\ell} \quad \forall \ell,
\]

and for the \( t \)-th period,

\[
\sum_{\ell} C_{i2}^{\ell} \left( \sum_{k=1}^{\ell} Y_{i2}^{k} - \sum_{k=1}^{\ell} Y_{i1}^{k} \right) \]

\[
Y_{it-1}^{\ell} \leq Y_{it}^{\ell} \quad \forall \ell, t \quad \ldots \quad \ldots \quad (15)
\]
Then the model becomes:

\[
\min_{t=1}^{T} \sum_{i=1}^{m} \alpha^t \left[ \sum_{i=1}^{m} \left( C_i + \sum_{k=1}^{\ell} Y_{it}^k - \sum_{k=1}^{\ell} Y_{it}^k \right) X_{it} + K_{it} \right] \\
+ \sum_{j=1}^{n} D_j \left( \frac{S_{jt}}{Q_{jt}} \right)
\]

s.t. \( T \)

\[
\sum_{j \in I-I} \beta_j \sum_{i \in I} Y_{it}^j = S_{jt} \quad \forall j, t
\]

\[
\sum_{k=1}^{\ell} X_{ik} M_i \geq \sum_{i \in I} Y_{it}^j \quad \forall i, t
\]

\[
\sum_{i \in I} X_{it} \leq 1 \quad \forall i
\]

\[
X_{it} = 0, 1 \quad \forall i, t
\]

\[
S_{jt} \geq S_{jt} \geq 0 \quad \forall j, t
\]

\[
Y_{it}^j \leq Y_{it}^j \quad \forall i, t, \ell
\]

\[
0 \leq Y_{it}^j \leq Y_{it}^j \quad \forall i, t, \ell
\]

(16)

The Dynamic Programming Model

An alternate model can be formulated through the framework of dynamic programming. This approach requires projects to have a discrete, rather than a continuous, scale definition. However, several alternate discrete scales for each project could be included. The following notation is necessary for the formulation.

- **i** project index \((i=1, 2, \ldots, m)\).
- **I** subset of projects assumed already established.
- **I-I** set I with \(i \in I\) deleted.
- **C_i** investment cost for project \(i\) (includes allowance for present value of replacement, operating costs).
- \(e^{-rt}\) discount factor from time \(t\) to time 0 at the rate \(r > 0\).
\( \theta_{j}(I, t) \) salinity level for \( j \)th station at time \( t \), assuming all projects in \( I \) operating at \( t \) (implies operating level of projects is not a variable, but effect on salinity can depend on other projects in existence, represented by \( I \)).

\( B_{j}(\theta, t) \) benefit rate at \( j \)th station at time \( t \) given salinity level \( \theta \).

\( B_{j}(I, t) \) benefit rate at \( j \)th station at time \( t \) given salinity level \( \theta_{j}(I, t) \), i.e., \( B_{j}(I, t) = B_{j}(\theta_{j}(I, t), t) \).

\( B(I, t) \) total benefit rate at time \( t \) given projects in \( I \), defined as \( B(I, t) = \sum_{j} B_{j}(I, t) \).

\( i[k] \) project index assigned to the \( k \)th position in a sequence.

\( \{i[k]\} \) complete assignment of project indices for a particular sequence, where \( k = 1, 2, \ldots, m \).

\( I^{*} \) set of all \( m \) project indices.

\( \mathcal{S}_{I^{*}} \) set of all \( m! \) permutations of \( m \) project indices.

\( I_{k} \) set of first \( k \) project indices for a particular sequence, where \( I_{0} = \emptyset \), \( I_{k+1} = I_{k} \cup i[k+1] \) for \( k = 0, 1, \ldots, m-1 \), and \( I_{m} = I^{*} \).

\( \tau_{k} \) establishment time for \( k \)th project in a sequence, where \( \tau_{0} = 0 \), \( \tau_{k} \leq \tau_{k+1} \), and \( \tau_{m+1} = +\infty \).

\( V(I^{*}, \infty) \) total net benefits over the time interval \([0, \infty]\), discounted to time 0, for maximum-benefit sequencing and timing decisions for the set of \( m \) projects.

The general formulation for sequencing and timing projects with the objective of maximizing total benefits is:

\[
V(I^{*}, \infty) = \max_{\{i[k]\} \in \mathcal{S}_{I^{*}}} \max_{\{\tau_{k}\}} \left\{ \sum_{k=0}^{m} \int_{\tau_{k}}^{\tau_{k+1}} B(I_{k}, t) e^{-rt} dt - \sum_{k=1}^{m} C_{i[k]} e^{-r\tau_{k}} \right\}
\]

subject to constraints

\[
\tau_{0} \leq \tau_{1} \leq \tau_{2} \leq \ldots \leq \tau_{m} \leq \tau_{m+1} \ldots \ldots \ldots \ (17)
\]

This formulation allows the possibility of not establishing some projects, since an establishment timing of \(+\infty\) implies indefinite postponement, which is equivalent to eliminating the project from consideration.
Solution approach

The formulation (17) may be solved with the dynamic programming formulation of Erlenkotter and Rogers (4), which is a refinement and simplification of the basic approach in Erlenkotter (2).

Discussion of the approach in a benefit-maximization context related to the one here is given in Erlenkotter and Trippi (5). It is anticipated that the approach can be improved considerably by incorporating bounding procedures into the dynamic programming framework. Several types of bounds have been derived in Erlenkotter (3). Morin and Marsten (8) have also worked on this type of hybrid dynamic programming-branch and bound approach and have promising preliminary results. They, however, are dealing with a much simpler, and less realistic, problem definition than the one considered here.

Discrete vs. continuous-time formulations

For simplicity in notation, we have described a continuous-time formulation here. As noted in (4), an equivalent discrete-time formulation is possible with all the same characteristics. The choice between one or the other is best made on the basis of computational simplicity, depending on whether integration or period-by-period summation of benefit functions is easier for the particular functional forms employed. For the Colorado River model, it appears that a discrete-time model will be best suited to providing flexibility in representing benefit functions and salinity levels over time.

Salinity standards

In addition to the benefit-cost analysis of salinity control proposed here, inclusion of specific salinity "standards" or limits may be desired. This might be desirable to evaluate the welfare loss (if any) entailed by such standards. To incorporate salinity standards, define:
\[ \beta_j(t) = \text{salinity standard for } j^{th} \text{ station at time } t, \text{ specified exogenously.} \]

Impose the constraints

\[ \theta_j(I, t) \leq \beta_j(t) \text{ for all } I, j, \text{ and } t \] \hspace{1cm} (18)

Suppose project \( i \) is to be added to the set \( I-i \), and the unconstrained optimal timing for adding \( i \) is given by \( \tau^*(I, i) \). The optimal constrained timing, taking into account the constraints (18), will now be:

\[ \tau^{**}(I, i) = \min \{ \tau^*(I, i); \min_j \sup_{t=0}^{\infty} \{ \theta_j(I-i, t) \leq \beta_j(t) \} \} \] \hspace{1cm} (19)

This modification sets the optimal unconstrained timing at the earlier of the unconstrained timing or the earliest time at which one of the constraints becomes violated. Note there is no requirement that the standard \( \beta_j(t) \) be non-increasing in \( t \).

If no feasible solution is possible due to the lack of sufficient projects and the tightness of constraints, this would easily be detected by finding the maximum horizon length up to which the standards could be met with all available projects.

Anticipated Work

In keeping with the objectives of this project, these two analytical approaches will be investigated with regard to computational feasibility and the ease with which they accommodate the characteristics of the problem. In performing this investigation, sequencing results will be determined using data available from the studies mentioned above. Since these studies are not drawing to a close as rapidly as had been anticipated, the sequencing results may only be preliminary because the data from these studies are basic input to the sequencing models.

Some interesting results can be obtained even with the preliminary data now available. For example, it will be possible to compute the impact on the investment sequencing schedule, and hence on salinity,
of extra water from, say, weather modification or inter-basin transfer. Another main result will be the shadow cost of the "1972" standards, obtained by solving the sequencing models with and without the "1972" standards imposed. And, the general models will be available for future use as more definitive data becomes available.

Acknowledgments

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References


MODELS AS A MANAGEMENT TOOL

by

J. E. Sarenski

Introduction

Traditionally, water quality management planning was viewed as a technological problem, solved through the construction of more and more sophisticated hardware (treatment plants, pipes, etc.) costing more and more money. Because there are still major water pollution problems even in areas where extensive planning has occurred, it is clear that traditional methods are not getting the job done and new, innovative thinking is called for.

Perhaps a prime reason previous attempts at water quality management planning have failed is the lack of consideration given to the relationships between water quality management and the socio/economic setting of an area as illustrated in Figure 1. As illustrated in the figure, population has a direct influence on the demand for goods and services which stimulates industrial and commercial development. This stimulation creates an employment demand and a general increase in land development through the need for schools, roads, houses, etc. Population and development generate various forms of wastewater which influences the environmental pressures created by the difference between the actual environmental setting and the population's desired environmental setting. This environmental pressure is decreased through environmental management which may influence the character of land development. Stress is constantly generated between land development and environmental management, in part due to the difference generates pressure for increased development, and thus, additional environmental pressure.

* Nelson, Haley, Patterson, and Quirk, Denver, Colorado.
Figure 1. Economic/water quality interrelationships.
The major implication of the economic/water quality feedback diagram illustrated in Figure 1 is that numerous tradeoffs exist between economic development and water quality. Tradeoff elements associated with development are readily translated into monetary terms, such as value added by industry, increased tax revenues, assessed valuation, etc., whereas many elements related to water quality are non-quantifiable. Thus, a classic situation exists in evaluating tradeoffs. Because the thrust of recent water quality management planning programs, including those sponsored under Section 208 of PL 92-500 (Federal Water Pollution Control Act Amendments of 1972), is "implementation," the planning process, whereby the tradeoffs are identified, must be conducted in the public decision-making arena and the final water quality management system negotiated openly.

Developing the full range of tradeoffs implicit in the feedback loops in Figure 1 requires the water quality planner to display the impact on the various management elements of changes in any one element. To this end, Nelson, Haley, Patterson and Quirk, Inc. (NHPQ) has developed a system of integrated mathematical models to assist in the planning process. The following sections contain a brief description of these models as a management tool.

**Physical Systems Planning**

A total of five models are being used by NHPQ in its current water quality management planning efforts. These are:

- Land Use
- GENERATE (Wastewater Generation)
- SEWER (Interceptor Design/Cost)
- TPM (Treatment Design/Cost)
- Water Quality
The land use and water quality models are generic tools, typically supplied by the local planning agency and the U.S. Environmental Protection Agency (EPA), respectively. For purposes of this presentation, the land use and water quality models being used in developing the Colorado Springs Section 208 Plan are used as examples, although GENERATE, SEWER, and TPM are adaptable to a vast variety of land use and water quality models. The land use model developed by the Pikes Peak Area Council of Governments (PPACG) is called PLUM (1) and the water quality model developed by Battelle Memorial Insititue under contract to EPA is named PIONEER (2).

The relationships among the five models are illustrated in Figure 2. Land use forecasts developed from PLUM are converted to wastewater generation by using GENERATE. This prescribes the wastewater flow within a defined geographical area which is the starting point for interceptor locations. Alternative treatment plant sites are selected for wastewater discharges and PIONEER used to define maximum wasteloads for various treatment plant siting schemes. Alternative wasteload allocations became the effluent constraints for TPM and interceptor flow is the influent stream. These two data sets are used to develop alternative feasible treatment systems. Interceptor alignments to transport generated wastewater to the alternative treatment plant sites are analyzed by SEWER.

The input/output of each model is presented below.

**Land use model - PLUM**

PLUM operates with a given set of specified assumptions on growth and development patterns and generates certain land use data for prescribed geographical areas for 5 year increments between 1975 to 2000, inclusive. Originally, the geographic areas were PLUM ZONES, a set of pseudo-homogeneous development areas, but this level of disaggregation proved undesirable for water quality planning. Additionally, the types of land use data originally generated
Figure 2. Interrelationships among Project Aquarius models.
were not sufficiently definitive, particularly in the industrial sector, to permit reasonable wastewater generation forecasts.

In order to make the PLUM model output more compatible with water quality planning, the output was changed by disaggregating the land use forecasts by service district (water and sewer). In areas around existing service districts where growth is anticipated, the development is assumed to be connected to the existing districts. In other areas where no service district exists, the location of anticipated development is illustrated on a map. Additionally, the land use data forecasts now more closely detail industrial growth.

The output from the PLUM model includes the following data disaggregated by present or future service district for 5 year increments from 1975 to 2000:

- Residential population
- Residential acres
- Commercial acres
- Industrial employees by two digit SIC between SIC 20 and 39.

Wastewater generation model - GENERATE

The land use data discussed above serves as input to GENERATE. These data are converted to wastewater flow through a series of transforms as discussed below.

**Residential**

Residential wastewater flow in MGD is computed by:

\[ \text{PEOPLE} \times 10^{-4} = \text{MGD} \]

Residential wastewater flow in PE is equal to PEOPLE.

**Commercial**

Commercial wastewater flow in MGD is:

\[ \text{COMM. ACRES} \times 2(10)^{-3} = \text{MGD} \]

Commercial wastewater flow in PE is:

\[ \text{MGD} \times 10^{4} = \text{PE} \]
Industrial

Calculating industrial wastewater generation is the most complex manipulation of the three. The number of employees in each SIC is multiplied by the corresponding transform presented in Table 1, and the gallons per day are summed over all SIC's and converted to MGD and PE. Thus, the industrial wastewater flow in MGD is:

\[
\sum_{i=20}^{39} EMP_i \times GPED_i \times 10^{-6} = MGD
\]

and in PE is:

\[
MGD \times 10^4 = PE
\]

where:

\[
EMP_i = \text{number of employees in SIC}_i
\]
\[
GPED_i = \text{gallons/employee/day for SIC}_i
\]

Total wastewater flow

Total wastewater flow in either MGD or PE is simply the sum of residential, commercial, and industrial values. An example of the output format for GENERATE is presented in Table 2.

Interceptor model - SEWER

SEWER requires a total wastewater flow and certain physiographic data as input. Total wastewater flow is derived from GENERATE. Profile elevations (ground) are taken from USGS quad-maps. Soil type, urban development, pavement, groundwater level, depth to bedrock, water crossings, etc., are derived from various maps and overlays. The program computes the slope and diameter of required pipe based on the above inputs plus design criteria including minimum/maximum cuts and velocities. Gravity sewer design is straightforward, but where a positive slope of ground profile is indicated, the program checks the cost differential between gravity and pumped flow. The total cost of the system is estimated by adding various surcharges to the basic cost to purchase and deliver reinforced concrete pipe. Surcharges include:
Table 1. Industrial wastewater transforms.

<table>
<thead>
<tr>
<th>SIC</th>
<th>Gallons/Employee/Day</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>1,490</td>
</tr>
<tr>
<td>21</td>
<td>230</td>
</tr>
<tr>
<td>22</td>
<td>810</td>
</tr>
<tr>
<td>23</td>
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<tr>
<td>24</td>
<td>1,370</td>
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<tr>
<td>25</td>
<td>190</td>
</tr>
<tr>
<td>26</td>
<td>14,800</td>
</tr>
<tr>
<td>27</td>
<td>0</td>
</tr>
<tr>
<td>28</td>
<td>3,840</td>
</tr>
<tr>
<td>29</td>
<td>4,110</td>
</tr>
<tr>
<td>30</td>
<td>490</td>
</tr>
<tr>
<td>31</td>
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<tr>
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<td>975</td>
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<td>230</td>
</tr>
<tr>
<td>37</td>
<td>500</td>
</tr>
<tr>
<td>38</td>
<td>380</td>
</tr>
<tr>
<td>39</td>
<td>330</td>
</tr>
</tbody>
</table>
Table 2. Example of output format for generate.

<table>
<thead>
<tr>
<th>SECURITY</th>
<th>POP.</th>
<th>DTON,</th>
<th>15A.</th>
<th>RIC</th>
<th>EMPLOYEE</th>
</tr>
</thead>
<tbody>
<tr>
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<td>5</td>
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<td>75</td>
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<td>50</td>
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<td>0</td>
<td>74</td>
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<tr>
<td>27</td>
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<td>30</td>
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<td>74</td>
<td>30</td>
<td>14</td>
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<tr>
<td>30</td>
<td>100</td>
<td>30</td>
<td>14</td>
<td>32</td>
<td>33</td>
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<tr>
<td>30</td>
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<td>30</td>
<td>100</td>
<td>30</td>
<td>14</td>
<td>32</td>
<td>33</td>
</tr>
</tbody>
</table>

**WASTEATER**

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<tr>
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<th>POP.</th>
<th>DTON,</th>
<th>15A.</th>
<th>RIC</th>
<th>EMPLOYEE</th>
</tr>
</thead>
<tbody>
<tr>
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<td>24</td>
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<td>0</td>
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<td>14</td>
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<tr>
<td>30</td>
<td>100</td>
<td>30</td>
<td>14</td>
<td>32</td>
<td>33</td>
</tr>
</tbody>
</table>

95
- earthwork, $/yd^3
- pavement removal/replacement, $/ft^2
- groundwater control, $/ft
- rock excavation, $/yd^3
- water crossing, $/ft
- congestion, $/ft

Cost data are expressed in terms of:
- capital investment
- annual O/M
- total annual cost
- total annual per capita cost

An example of SEWER output is presented in Table 3.

**Water quality model - PIONEER**

PIONEER is a rather complex steady-state water quality model developed by Battelle for EPA. It allows one to model a variety of conservative and non-conservative constituents/parameters and provides a printed profile by river mile based on selected river mile increments. Inputs to PIONEER are headwater flows and qualities, various point and non-point discharges (quantity and quality) and reaction rates. Several non-conservative constituents, such as nitrogen and phosphorus in various forms, can be modeled by assuming simple reaction rates, e.g., $\text{NH}_3\rightarrow\text{NO}_3$, or by more complex modeling of the entire constituent cycle. The latter approach is rarely taken because of a lack of data.

For the Colorado Springs 208, the available data base limits the use of PIONEER to modeling of:
- BOD
- DO
- $\text{NH}_3\cdot\text{N}$
- $\text{NO}_3\cdot\text{N}$
- Flow
Table 3. Example of output format for sewer.

<table>
<thead>
<tr>
<th>C</th>
<th>INPUT TEST1</th>
<th>000002210</th>
</tr>
</thead>
</table>

**STORE TEST1**

<table>
<thead>
<tr>
<th>STARTING STATION</th>
<th>10.00</th>
</tr>
</thead>
<tbody>
<tr>
<td>DISTANCE: DISTANCE FROM STARTING STATION</td>
<td></td>
</tr>
<tr>
<td>LENGTH: LENGTH OF THE SURCHARGE</td>
<td></td>
</tr>
<tr>
<td>ROWN</td>
<td>DISTANCE 1,8000 LENGTH 0.6000 COST PER ACRE $ 750.00</td>
</tr>
<tr>
<td>PAVEMENT</td>
<td>DISTANCE 7,3000 LENGTH 0.1000 TYPE CONCRETE</td>
</tr>
<tr>
<td>PROFILE</td>
<td>DISTANCE 4.0000 ELEVATION 5997.00</td>
</tr>
<tr>
<td>PROFILE</td>
<td>DISTANCE 3,0000 ELEVATION 5991.00</td>
</tr>
<tr>
<td>PROFILE</td>
<td>DISTANCE 4,0000 ELEVATION 5975.00</td>
</tr>
<tr>
<td>CALCULATE TEST1</td>
<td></td>
</tr>
</tbody>
</table>

**SUMMARY**

<table>
<thead>
<tr>
<th>STATIONS</th>
<th>INITIAL INPUT FLOW</th>
<th>INITIAL DEPTH</th>
<th>MIN. PIPE</th>
<th>END</th>
<th>MANNING</th>
</tr>
</thead>
<tbody>
<tr>
<td>BEGINNING</td>
<td>ENDING</td>
<td>LENGTH (Ft)</td>
<td>4000, Ft.</td>
<td>4000, Ff</td>
<td>6.0t.</td>
</tr>
</tbody>
</table>

**SURCHARGES BETWEEN STATIONS**

| RIGHT OF WAY | 14.40 | 14.40 | 800. |
| PAVEMENT | 18.10 | 18.10 | 500. |
| PAVEMENT | 19.10 | 19.10 | 60. |

**COST** $750.00 ACRE

| COST | $711.00 ACRE |
| TYPE CONCRETE | TYPE CONCRETE |

---

97
Table 3. Continued.

Table: Design/Cost Report for Interceptor Test 3

<table>
<thead>
<tr>
<th>Station</th>
<th>Cut</th>
<th>Elevation</th>
<th>Cut</th>
<th>Elevation</th>
<th>Ground</th>
<th>Pipe</th>
<th>Flow</th>
<th>Velocity</th>
<th>Pipe</th>
<th>Length</th>
</tr>
</thead>
<tbody>
<tr>
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</tr>
</tbody>
</table>

**Gravity System**

<table>
<thead>
<tr>
<th>Station</th>
<th>Cut</th>
<th>Elevation</th>
<th>Cut</th>
<th>Elevation</th>
<th>Ground</th>
<th>Pipe</th>
<th>Flow</th>
<th>Velocity</th>
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<th>Length</th>
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</tr>
</tbody>
</table>

| Excavation & Backfill Cost | 10,000 | 11,800 | $19,400 |

**Gravity System**

<table>
<thead>
<tr>
<th>Station</th>
<th>Cut</th>
<th>Elevation</th>
<th>Cut</th>
<th>Elevation</th>
<th>Ground</th>
<th>Pipe</th>
<th>Flow</th>
<th>Velocity</th>
<th>Pipe</th>
<th>Length</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
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<td></td>
</tr>
</tbody>
</table>

**Pipe Line Cost**

| Total PE | 4,000 |
| Station | 10,800 | To | 19,800 | Length | 4000, Feet |

**Cost Report for Interceptor Test 3**

<table>
<thead>
<tr>
<th>Capital</th>
<th>Cost Category</th>
<th>Cost</th>
<th>D/M</th>
<th>Annual</th>
<th>Total Cost</th>
<th>Per Capita</th>
<th>($/Yr)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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</tr>
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<td></td>
</tr>
</tbody>
</table>

**Total Interceptor Costs**

<table>
<thead>
<tr>
<th>Capital</th>
<th>Cost Category</th>
<th>Cost</th>
<th>D/M</th>
<th>Annual</th>
<th>Total Cost</th>
<th>Per Capita</th>
<th>($/Yr)</th>
</tr>
</thead>
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<tr>
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</tr>
</tbody>
</table>

**Cost**

<table>
<thead>
<tr>
<th>Capital</th>
<th>Cost Category</th>
<th>Cost</th>
<th>D/M</th>
<th>Annual</th>
<th>Total Cost</th>
<th>Per Capita</th>
<th>($/Yr)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
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<td></td>
<td></td>
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<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Page 98**
However, it is anticipated that as the data become available more parameters will be investigated.

The input echo print and output from PIONEER is extremely lengthy and rather complex. A sample of the output format is presented in Table 4.

Treatment model - TPM

Basic input to TPM is:

- Flow, expressed in PE, from SEWER model
- Effluent limits, expressed in PE, for parameters of concern, derived from PIONEER or other effluent limitation specifications. Presently, the parameters of concern are BOD, TSS, Fecal Coliforms, TP, and NH₃-N.
- Land cost, $/acre.

Required percent removals for each parameter are computed from the above. A scan of 30 treatment systems identifies the feasible (in terms of removal efficiencies) alternatives. The user is allowed to prescribe any alternatives not to be considered (e.g., lagoons where land is a constraint) and the program will check to see which alternatives meet certain capacity requirements (e.g., extended aeration plants are considered only below 1,000 PE). The program then uses the feasible alternatives and associated removal efficiencies to back-calculate the actual effluent qualities.

Capital and O/M costs of each feasible alternative are computed and adjusted to any desired ENR Index. Total annual and annual per capita costs are also computed. Based on the size of plant required and character of sludge produced, up to 11 sludge handling systems are costed. The required land area for each the liquid treatment and sludge handling systems is estimated and priced based on dollars/acre data entered as input. All alternatives and the various cost elements of each are then displayed. An example of TPM output is presented in Table 99.
<table>
<thead>
<tr>
<th>Identifier</th>
<th>River Mile</th>
<th>Length (Miles)</th>
<th>Flow Rate (CFS)</th>
<th>Travel Time (Days)</th>
<th>Velocity (Fps)</th>
<th>Depth (Ft)</th>
<th>Flow (Mpg/l)</th>
<th>Depth (Mpg/l)</th>
<th>River Solids (Mpg/l)</th>
<th>Temperature (C)</th>
</tr>
</thead>
<tbody>
<tr>
<td>954</td>
<td>7.162E 00</td>
<td>8.117E 00</td>
<td>8.403E 00</td>
<td>1.20</td>
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<td>7.26</td>
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<td>0.0</td>
</tr>
<tr>
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<td>8.117E 00</td>
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</tr>
<tr>
<td>958</td>
<td>7.632E 00</td>
<td>8.117E 00</td>
<td>8.117E 00</td>
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<td>0.0</td>
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<td>7.63</td>
<td>7.63</td>
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<td>0.0</td>
</tr>
<tr>
<td>960</td>
<td>7.632E 00</td>
<td>8.117E 00</td>
<td>8.117E 00</td>
<td>0.0</td>
<td>0.0</td>
<td>7.63</td>
<td>7.63</td>
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<td>7.66</td>
<td>7.66</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>966</td>
<td>7.661E 00</td>
<td>7.997E 00</td>
<td>7.997E 00</td>
<td>0.0</td>
<td>0.0</td>
<td>7.66</td>
<td>7.66</td>
<td>7.66</td>
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</table>
Table 5. Example of output format for treatment planning/cost model.

<table>
<thead>
<tr>
<th>Flow</th>
<th>10000, PF</th>
<th>10000, PF</th>
</tr>
</thead>
<tbody>
<tr>
<td>HIN</td>
<td>40000, PF</td>
<td>40000, PF</td>
</tr>
<tr>
<td>TSS</td>
<td>5000, PF</td>
<td>5000, PF</td>
</tr>
<tr>
<td>FEC</td>
<td>5000, PF</td>
<td>5000, PF</td>
</tr>
<tr>
<td>TP</td>
<td>5000, PF</td>
<td>5000, PF</td>
</tr>
<tr>
<td>LAND COST</td>
<td>$1500,000</td>
<td></td>
</tr>
</tbody>
</table>

**Removal Matrix**

<table>
<thead>
<tr>
<th>Qnt</th>
<th>HIN</th>
<th>TSS</th>
<th>FEC</th>
<th>TP</th>
<th>WHS</th>
</tr>
</thead>
<tbody>
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<td>95</td>
<td>94.99</td>
<td>90</td>
<td>70</td>
</tr>
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<td>7</td>
<td>97</td>
<td>97</td>
<td>94.99</td>
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</tr>
<tr>
<td>8</td>
<td>98</td>
<td>98</td>
<td>94.99</td>
<td>90</td>
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</tr>
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<td>97</td>
<td>94.99</td>
<td>90</td>
<td>70</td>
</tr>
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The recent focus in water quality management planning emphasizes implementation of the final plan. To achieve this goal, it is imperative that the public decision-making process be aware of the tradeoffs implicit in the relationships between economic development and water quality management. In appreciation of the vast range of alternatives which should be considered including alternative land use plans, physical system configurations, construction phasing, and water uses, a system of integrated models is required to rapidly display the impact of various options to the decision-makers.

In consort with the Pikes Peak Area Council of Governments and EPA, NHPQ has developed a system of five integrated mathematical models to accomplish water quality management planning in the public decision-making arena. These models include: 1) a land use model (PLUM) to specify various elements of future development; 2) a wastewater generation model (GENERATE) to convert land use to wastewater contributions by geographical areas; 3) a water quality model (PIONEER) to estimate the constituent/parameter profile resulting from a specific wasteload allocation; 4) a treatment design/costing model (TPM) to select the feasible alternatives for converting the generated influent to allowable effluent; and 5) an interceptor design/costing model (SEWER) to display the cost of various interceptor configurations.

Advantages of this modeling system are the ability to display the impact of a large number of alternatives and to assess changes in plan alternatives with great speed.

References

1. PROJECTIVE LAND USE MODEL (PLUM), Pikes Peak Area Council of Governments, 1974.
LEGAL AND INSTITUTIONAL CONSTRAINTS
IN THE USE OF MODELS

by

Henry P. Caulfield, Jr.*

Introductory Remarks of the Moderator

The engineering literature of systems analysis with regard to water resources planning and management often brings the ordering of data and their analysis to an abstract, hypothetical "decision-maker." Little, if any, consideration usually appears to be given to the legal, institutional and political context of decisions, the needs for information as viewed by political and administrative decision-makers, or to the process of decision-making which can be a very complex system itself, involving many more than one decision-maker.

The "decision" expected is an affirmation and implementation of the analytic result or a choice among alternatives for implementation that stem from the analysis. Rejection of the analytic result, or failure to make a choice among the choices presented, tends to be presented by the analyst in terms that are not flattering to the decision-maker or to society. And such feelings are often reciprocated.

The purpose of this session is to help develop understanding of the problem of relating systems analysts to decision-makers through discussion of "legal and institutional constraints in the use of models."¹

*Professor of Political Science, Colorado State University.

¹For a more complete version of the Moderator's views on relating systems analysts to decision-making, see "Institutional and Political Constraints," Chapter 2 in Water Resources Planning, Social Goals and Indicators: Methodological Development and Empirical Test, by the Technical Committee of the Water Resources Research Centers in the Thirteen Western States (Utah Water Research Laboratory, Utah State University, Logan, Utah 84322: December 31, 1974, PRWG 131-1).
The decision-makers of concern to use in this context usually are public officials. Thus, fundamentally, what is involved in their decisions is "politics," which here is not taken to be a dirty word. For purposes of this analysis, "politics" can best be said to be the processes by which a society makes authoritative decisions about the allocation of value. ²

The outputs of politics can be said to be "policy" and the value significant effects of the implementation of policy is society. In abstract terms, "policy" is the criteria by which a decision-maker decides what to do or what not to do in a given factual situation. ³ Persons in government are very conscious that criteria external to their own ideas constrain their public decisions and, in principle, they believe this to be appropriate.

More concretely public policy can be seen as a hierarchical system of constraints upon the freedom of decision-makers:

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SPECIFIC DECISION ———> | ←— SPECIFIC DECISION
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³ Adapted from Carl J. Friedrich, Constitutional Government and Democracy, (Ginn and Company, Boston, 1950) p. 362. Somewhat
All of these levels of public policy provide criteria that constrain decisions. The higher levels constrain the lower-level sources of criteria.

Decisions involving the taking of private land into public ownership are constrained by the U.S. Constitutional provision prohibiting the taking of land without just compensation. The U.S. Constitution also constrains water resource plans of the federal government due to the apparent fact that it has no authority to zone flood plains. Authority for flood plain zoning is only available to state and local governments. Also, the federal government cannot directly assess specific lands for benefits received from flood protection storage. Thus, rather than wait for one or more benefitted states to create the necessary local districts to provide reimbursement of some costs (as is the case with respect to federal irrigation costs) the federal government provides the larger flood protection storage works as a non-reimbursable federal expenditure.

Policy embodied in law, the interpretation of which is conditioned by its legislative history, is extensive and becomes very particularized in application. Moreover, extant policy in law has been accumulated over a long period of time. Some, embodied in the common law, was established ages ago. Other extant policy was adopted by statute in the 19th century. Much more statute law still applicable to water and related land resources has been enacted in this century.

Judicial interpretation of law clearly provides decision criteria that executive decision-makers do not ignore. The well-known experiences of federal water agencies since passage of the National Environmental Policy Act in 1969 make this evident.  

Similarly, policy is defined by David Easton in A Systems Analysis of Political Life (John Wiley & Sons, Inc. New York, 1965) as "decision rules adopted by authorities as a guide to behavior..." (p. 358).

The extant Principles and Standards, regulations of the Water Resources Council, are clearly intended to provide criteria consistent with law to guide planners and decision-makers.

The next three levels of criteria are very real, but less uniform and fixed, in terms of their effects upon planning and decision-making. Official "policy" statements are, in effect, calls upon lower officials in the exercise of their discretion to tilt their decisions in accord with the explicit or implicit criteria of the policy statement.

Professional standards derive from intellectual disciplines, training, experience, and professional-society policy. Engineers, economists, biologists, etc., all bring to their work the professional standards of their professions and they are expected to do so.

Finally, the value preferences of the planner or decision-maker, within whatever freedom of decision is left to him, are inevitably involved in his decisions. His values, impacting upon his decisions, can be those that he has long held personally or professionally; or they can be values that he has decided to take into account as a result of public participation in processes of planning and decision-making.

Specific decisions can be said to derive (to continue the metaphor of hierarchy) from criteria imposed from above as well as criteria promoted by public participation from below. Because much that occurs in government depends upon the active interest and substantial concurrence of the affected publics, public participation is an essential element in the realization of plans in terms of actual operations and achievement of effects.

Lead planners and field decision-makers work at the initial interface in a specific factual context between government and what its policy permits, on the one hand, and specific public interests and what these interests need as the planner or they see their needs, on the other. This interface in such a context clearly puts lead planners and field decision-makers in "the middle" in a political (i.e. value allocational) role. Systems analysts need to see this situation objectively
in all of its complexity and then they must plan their potential information-analytic contributions to decision-making in an effective, realistic manner with the means available to them.

In this regard, systems analysts need to view legal and institutional constraints as relatively fixed, or static, part of the real world that they cannot ignore. They may choose for very good and sufficient technical or other reasons not to include explicitly these constraints in their models; but they must recognize then that decision-makers, very appropriately, must view the results of their analysis in the context of appropriate implicit constraints in making their decisions.

Although constraints need to be recognized as relatively fixed, it should also be recognized that constraints can be changed incrementally at politically opportune times. Policy can be viewed dynamically, as well as statically, but not usually in the short run.

The dynamic element can be seen as policy thrusts operating in the historic post, as well as presently, to change policy or to resist change. In the area of water and related land managements (as well as natural resources management generally) three thrusts can be identified:

1. **Development Thrust** -- fostering economic expansion.
2. **Progressive Thrust** -- fostering egalitarian treatment in the distribution of the benefits of expansion.
3. **Conservation Thrust** -- fostering (on the basis of professional and other relatively elite concerns);
   (a) Sustained yield and multiple use
   (b) **Environmental quality** -- water quality control -- preservation of wild and scenic rivers.
In any context of basic policy change, all three of these thrusts can be seen to be operative, but in various mixes of relative strength depending upon the nature of the specific policy-change proposal and the historical period in which the policy development occurs. An extended discussion of this topic cannot be given here. Discussion here will by confined to partial treatments of two currently pertinent policy developments.

In "Colorado River Basin--Policy Goals and Values in Historical Perspective," a general analysis of policy change has been set forth: from a policy dominated by the concept of development of the arid West to a policy of balance, at least, or developmental and perservational interests. The traditional Conservation Movement led by Gifford Pinchot at the turn of the century was, in effect, a complex and changing coalition of the three thrusts bringing about policy developments in law and administrative practice. In this early context the dominant element of the conservation thrust was the concept of "sustained yield" or renewable resources. Interest in "preservation" was present. But it was politically effective only when not frontally challenging development, as in establishment of Yellowstone National Park in 1872, and Yosemite National Park in 1890 and in passage of the Antiquities Act of 1906. But, when directly confronted, preservation had to give way even to ideas of possible future developments, as indicated by the "reservation clauses" in the documents establishing, for example, Grand Canyon National Park, Rocky Mountain National Park and Dinosaur

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6 Paper presented by the Moderator at Symposium sponsored by the Committee on Arid Lands, American Association for the Advancement of Science, at Annual Meeting, AAAS, San Francisco, California, February 28, 1974 (unpublished; copies available from the author).
National Monument. And, of course, when preservation interests were confronted by immediate developmental interest, as in the celebrated Hetch-Hetchy case settled in 1913, the preservation interests lost. Not until the political fight that removed the proposed authorization of Echo Park dam (which would have been located within Dinosaur National Monument) from the Colorado Storage Project Act of 1956 and the proposed authorization of Bridge and Marble canyon dams from the Colorado Basin Project Act of 1968 can it be said that the conservation thrust, in the sense of preservation of natural conditions, came to be really politically potent. But, even then, preservationists had to pay developmentalists politically for their victories, in the first instance, through acceptance of a less than secure solution to the Rainbow Bridge National Monument problem and, in the second instance, by agreeing to a huge coal-fired stream-electric plant in place of the two hydroelectric dams.

Policy change, historically, can be illustrated also with respect to flood hazards. Flood control through channelization and construction of levees and dams, was the first policy approach which developed over many years and became general federal policy in the Flood Control Act of 1936. Of course, this policy, as manifest in engineering works, had the effect not only of reducing flood losses in terms of existing property in flood plains and loss of life of existing occupants, but also of encouraging greater property development and occupying of flood plains. Through the valiant efforts over many years of Professor Gilbert White, as well as other leaders, to demonstrate the futility and costs of this policy in the long run, and with the political aid in recent years of those interested in land-use planning generally, and particularly open-space in flood plains, the policy is shifting from "flood protection" to "flood plain management." 7 Flood plain zoning, flood insurance, open space, flood proofing of buildings, etc.--as

well as public engineering works--are seen as multiple means of such management. The general authorization by the Congress to the Army Corps of Engineers in 1974 to propose flood plain land for public purchase, on the same local reimbursement terms as for local flood protection projects, as a means of flood management, is a key indication of Congressional policy shift. However, the Executive Branch through the Office of Management and Budget, at last reports, is refusing to go along with funds to carry out the initially authorized land acquisition projects that the Congress specifically authorized.

In summary, this introductory overview of the problem of "legal and institutional constraints in the use of models" has sought, first, to emphasize the complexity of the decision-making problem. Second, the political (i.e., value allocational role) of the decision-maker has been highlighted. Third, policy as a hierarchical system of constraints upon decision-makers has been set forth together with the role of public participation in decision-making. Fourth, it has been emphasized that in the conduct of particular modeling efforts, as a part of the planning process, policy needs to be looked upon as relatively fixed, or static. Finally, it was shown that water and related land resource policy over a longer run can be viewed as not fixed, but subject to a dynamic process of incremental change.
Natural scientists--hydrologists, ecologists, geologists, biologists and geochemists--who study some characteristic of the Colorado River Basin are capable of making measurements of considerable exactitude regarding various natural processes taking place in the basin. These measurements provide the basis for statements having high levels of statistical probability concerning the effects of various physical changes associated with management of the river. Thus, they can make predictions concerning Lake Powell with respect to deposition of calcium carbonate, growth of salt cedar and Russian Thistle, the process of eutrophication, and bank storage. Engineers and other scientists may use these data as the basis for making calculations concerning costs of alternative decisions with respect to management or structural or biological changes in the basin. These scientific measurements and conclusions derived from them, plus the costs of dealing with undesirable features associated with existing or predicted conditions, are major constraints on decision-making.

Decision-makers with respect to the basin clearly recognize that there are also social (including political) processes that take place both within and without the basin that provide limits on what can be done. Given the element of volition, it is generally not possible to state these constraints as scientific "laws" or assign them mathematical statements of probability, but these processes are nevertheless of the "if this, then that" variety, relating existing conditions, possible management, structural or biological changes, and likely social and political outcomes. Decision-makers can ascertain costs of alternative strategies, costs that must be measured in economic, social, and political terms.

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Social and political constraints often are more limiting and more costly to overcome than physical and biological constraints. Despite the fact that these constraints are the result of human choice, that they are part of the fabric of human institutions, and therefore, presumably capable of being refashioned at will, human attachments to them and expectations derived from them make them very difficult to alter. The laws, traditions, administrative rules, and judicial decisions—in large part made up of the "Law of the River" in the Colorado River Basin—become battlements behind which various contending interests defend themselves. They constitute legal and ethical imperatives which generate powerful emotions and entrenched political positions.

These social and political constraints do not necessarily lead to optimization of resource use as calculated by the engineer or the economist. They express deeply held values of various populations affected by or dependent on resources. The precise balance struck among those competing values may in fact be in conflict with the optimization values developed by economists and engineers, providing instead a political optimization of values within the basin and within the broader society.

The broad outlines of the decision-making system are found in the basic structures of the American constitutional system. The system tends strongly to decentralize power and to fragment authority. The separation of power systems within the national government and the federal system that divides power between the national government and the states make the formation of national majorities difficult except under extraordinary circumstances such as in the election of a president. Authority for programs—planning, administration, enforcement—is shared among many agencies. Clientele groups attempting to influence diverse agency programs must therefore compete for access in the decision-making process. Local communities, states, and regional organizations are all major nuclei around which form political movements seeking some benefit from the various levels of government. Particularly in dealing with the national government, and especially in the field of water resources policy-making, the states are the major mobilizers of local, state-wide, and regional support.
In such a decentralized political system, the principal mode of decision-making is through a process of bargaining among the major interests concerned with given public policy. Bargaining is an essential element in any democratic process, relying, as it must, on compromise among conflicting interests. In water resource decision-making in the Colorado River Basin, this process takes on unique characteristics or patterns:

1. Coalitions are formed by various local and regional groups having an interest in projects of benefit to their particular locality. The states, through their water resources agencies, play important roles in effecting compromises both within and among the states. These compromises concern priorities among projects, allocation of costs, policies with respect to stream and reservoir management, and project design. Illustrations for this process are found in major legislation over the past 20 years: the Colorado River Storage Project, the Colorado Basin Project Act, and the Colorado River Salinity Control Act.

2. Federal agencies play vital roles in this political process. The Bureau of Reclamation provides technical expertise in project planning, including assistance in the bargaining process. Through the Bureau's reclamation program, local agencies are able to obtain financing for their projects; the Bureau's influence in Congress provides confidence in the legislators in the authorization and financing of the projects. The Environmental Protection Agency has responsibilities for imposing limitations on water use and development in the interest of protecting water quality; it also provides funds for planning, research, and construction of waste treatment plants. Local and regional interests find it necessary to bargain with EPA, particularly with respect to the salinity problem.

3. The availability of federal financing of projects makes such financing, including substantial subsidies, and economic justification central goals of project planners and their supporters. Subsidies have taken several forms: interest-free money for project construction; application of revenues from power production to pay for irrigation benefits; allocation of costs to nonreimbursable purposes. Economic
justification of projects has provided political support through under-
estimation of costs and overestimation of benefits. Financing for salinity
control projects, whether considered federal subsidies or not in view of
the federal lands involved, provides for the federal government to pay
for 75 percent of the cost of the first four projects, with the basin develop-
ment funds providing the remaining 25 percent. Federal financing makes
possible vote trading between members of Congress from the basin and
members of Congress from other areas that seek projects requiring fed-
eral financial support.

4. As a corollary to the above, local and regional interests seek
solutions that minimize burdens on themselves through solutions that put
the burden on nonbasin interests. National assumption of responsibility
for meeting the terms of the Mexican Water Treaty, proposals for im-
portation of water from the Northwest, and the desalination plant at Yuma
to deal with drainage water from the Wellton-Mohawk Project illustrate
this preference. Typically, the major burden is assigned to the national
taxpayer.

5. The existence of legal entitlements to water acquired through
state laws governing appropriations, through Congressional and inter-
state allocations, and through international treaty, makes the bargaining
process uncertain, in that one of the parties may find proposed arrange-
ments sufficiently damaging to their interests that they seek judicial
relief. The threat to do so constitutes a powerful incentive to achieve
agreement through compromise because of the uncertainties of judicial
results and the transaction costs in time and money in reaching judicial
decisions.

Models of the Colorado River Basin must take into account the
political institutions that govern the basin and the goals sought by the
parties who have a stake in the policy output associated with the basin's
water resources. Engineering analysis that seeks to maximize the avail-
able quantity of water or economic analysis that seeks to maximize over-
all economic benefit and minimize overall costs are incomplete if they
do not take into consideration questions of political equity for interested
parties, both with respect to process and to substantive results.
"HOW TO GET PEOPLE TO USE MODELS"

by

Jay M. Bagley*

The question of "how to get people to use models" is a large one having many ramifications. Mr. Holburt has dissected this question and has isolated several major problems which are constraining or interfering with the adoption and use of modeling techniques.

Obviously, there is a large arsenal of technology transfer techniques that can be employed in appropriate ways to transmit results of model development to the practitioner. These techniques range from traditional university classroom settings, to workshops and seminars, to various kinds of reports and papers. Perhaps the most effective technology transfer of all takes place when an individual with the modeling skills moves from a model development environment to a model applying atmosphere. In other words, the employment by action or mission agencies of those trained or experienced in modeling techniques is one of the best ways of carrying this technology into practical use. However, the matter does not end here because modeling is a process which is ever changing. New techniques, improvements, and modifications need to be incorporated from time to time to working models to improve their utility. The practicing modeler, plying his trade with a mission agency, can discover all kinds of bugs which limit effective use of a model. Yet the fire-fighting demands and deadlines associated with carrying out the basic mission may not allow the agency modeler the luxury of probing these difficulties in depth and finding ways to overcome them. The researcher, on the other hand, has both the time and peripheral support to delve into the problems whether they be software or hardware related, and come up with improvements and modifications which lead to more effective application.

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What I am saying is, that getting people to use models and keeping them updated and progressively more useful will always require a continuous two-way information exchange between model developer and model user. I would like to confine my focus to a single thread of this complex tapestry of model development and use to describe an approach that works well in specific instances. It is a workable arrangement that gives rather good assurance that the model will be put to good use. This is an approach in which modelers and model users organize themselves into a working collaboration around a specific problem for which a solution is needed and for which a faithful model could give insightful answers.

There are numerous advantages if participants in both the model development and model use phases can form a "one-on-one" association that continues over the life of a particular modeling problem. With a specific problem, there is a specific set of questions to be answered and these serve to fix the character and resolution of the model itself. The accompanying diagram represents a general description of how research supports action programs. Action agencies commonly identify problems or information gaps which hinder the effective and efficient accomplishment of their mission. The problem itself leads to the identification of the research need; to formulating the research approach; to conducting the research; to summarizing the results and conclusions; and finally, to applying the results in the solution of the problem originally encountered. I would like to refer to this diagram in describing a collaborative mode that has been reasonably successful in "getting people to use models."

Because of organizational, budgetary, and administrative separations, the cycle commonly operates like a "relay race" in which the baton gets passed from user to researcher and back to user. In this kind of transmission process, there is great opportunity for partial or total loss of information (dropping the baton). For example, the planner-manager commonly identifies a problem which may need special study to provide the analytic tools needed if the mission is to be accomplished in the most efficient and cost-effective manner. If there is not full communication when the research need is conveyed to the researcher by
PLANNING AND MANAGEMENT

ENTITIES
- STATE
- LOCAL
- PRIVATE
- FEDERAL

IDENTIFY RESEARCH NEEDS

APPLY RESEARCH RESULTS

FORMULATE RESEARCH APPROACH

REPORT RESEARCH FINDINGS

UWRL RESEARCH
by the user, the researcher may go off by himself to formulate the research approach with a warped picture of what the problem really is. When the results are handed back to the user, they may be good answers to the wrong question. Hence, little likelihood of application will result.

Similarly, even though the problem has been properly identified, the approach properly formulated, and the research completed in timely fashion, the results may be transmitted to the potential user in a way that they cannot be interpreted or adapted to be useful in a real world problem solution. Without elaborating on all the circumstances that can cause this kind of slippage or filtering in the transfer cycle, perhaps it has already become obvious that if the process we are describing could be viewed as a "hurdle race" instead of a "relay race" the chances of dropping the baton could be substantially minimized.

What this says is that a more successful pattern of technology transfer would be obtained if participants in the process, both researcher and research user, could remain in a lock-step through all phases of the cycle thereby reducing the chances of communication breakdown. (This does not mean that the hurdles will not be challenging to overcome.) This opportunity for the research-modeler to team up with the user-modeler and jointly follow the result into testing, adaptation, and final utilization would be valuable for both parties. The researcher would be exposed to viewpoints and ideas that would temper the research approach and give it a more practical orientation. There would be less chance that research would become moribund or myopic. Also a closer interchange would develop in the researcher a better insight into the social, political, and institutional framework within which his results must be implemented. On the other hand, planner-manager personnel would benefit from a better interaction with research through the intellectual stimulation that would reflect itself in more creative expression and innovation in the planning and management arena. A better appreciation and awareness of technological limitations and possibilities would give the planner-manager a more realistic faith in what models can and cannot do and
what practicalities exist in terms of time and budget constraints, assurances of success, etc.

At UWRL, we have had some success in getting models used in actual planning situations of this kind. In fact, the very first hydrologic model attempted nearly 15 years ago was developed within the pattern just described. There was not a lot of deep deliberation that went into the development of the collaborative pattern. It just evolved in an uncomplicated and almost automatic way. This modeling effort was in connection with a Sevier River Basin study under the direction of the Soil Conservation Service. Because the SCS needed to assess the hydrologic consequences of development alternatives that might be proposed, we suggested making an electronic analog model of the river basin which would reproduce the hydrologic flow system and allow the SCS to test the impacts from specific project operations. It was anticipated from the outset that, when and if the model were operational, SCS personnel would want to be able to make independent use of the model in various kinds of analyses that might be found desirable. Consequently, the SCS assigned a bright young man with good basic engineering training to collaborate closely throughout the formulation and verification of the model. This individual attended a special workshop at USU in analog and digital modeling. He spent considerable time at the laboratory, collaborating in the actual development of mathematical equations used to describe the various hydrologic processes and in the important phase of linking the various mathematical components together. As a result of this close working arrangement, SCS personnel were able to utilize the model independently upon its completion. They were fully aware of its limitations, the assumptions inherent, and could use it with judgment and confidence.

Incidently, upon the completion of this river basin study, the young SCS man who had worked so closely with the University modelers, had gained a capability that proved to be quite valuable to SCS. He was called into Washington and given a responsible position where his newly developed technology and talents could be reflected in national programs.
A second example of successful cooperative study between researcher-modelers at UWRL and user-modelers of the Division of Water Resources entailed the development of a simulation model of the hydrology of the Bear River Basin. The Bear River is an interstate stream coursing through the three states of Wyoming, Idaho, and Utah. Through frequent meetings and discussions, the Division and the UWRL jointly conceptualized the modeling approach. A common understanding was reached as to what the problems were, what kinds of answers were required from the model, the time and space resolution to be used in the model, the limitations of available data, and what additional data would be needed. Throughout the course of the development, a Division employee with previous experience in computer modeling spent an average of two days each week at UWRL. Thus, all questions which arose during the study were resolved jointly. When the model was completed, the report and computer program constituted more than a "black box" to the Division, because its personnel fully understood the model including its capabilities and limitations. Since that time, not only has the Division independently applied the model to many planning and management studies involving the water resources of the Bear River Basin, but also has been able to expand and further develop the model and refine it as needed. In this case, a highly effective utilization of research knowledge has been achieved.

A final example involves a cooperative effort between UWRL and the Bureau of Reclamation for the development of a water resources management model of the Provo River Basin in central Utah. This study has involved rather detailed considerations of both surface and groundwater hydrology and has required that other constraints be included, such as water rights and reservoir operating rules for multi-purpose development. Again, counterpart teams of professionals were organized at both the UWRL and the Provo district office of the Bureau of Reclamation. Each team had a prime contact man or principal investigator with other individuals having particular areas of expertise, such as surface water hydrology, groundwater hydrology, and water resources management.
Throughout this cooperative study, there was a high degree of inter­change through numerous meetings and discussions, both at UWRL and at the BOR offices in Provo. Early in the study, the team from UWRL spent several days in the district office discussing various aspects of the model development and in processing and evaluating available data. In the later phases of the study, the BOR team spent two to three days each month at the UWRL in assisting with final development, testing, and debugging of the model. Although the model was developed using hybrid computer facilities, when it was finally completed it was programmed to run on a digital computer to which the BOR has easy access. The teams from UWRL and BOR not only worked closely in developing and testing the model, but continued to collaborate in subsequent management studies involving water resource use and development in the Provo River Basin and the relationship of this resource use to the Central Utah Project. At the conclusion of this cooperative effort, personnel of the BOR team were thoroughly acquainted with the model and were capable of applying it independently to various kinds of management studies. Once again, this study served to demonstrate that close cooperation between researchers and users and the rapid feedback which it promotes can lead to highly effective application of modeling techniques for specific problem situa­tions.
The topic that I will relate to in my part of the panel discussion concerns patterns of group instruction in the use of models.

**Patterns of Group Instruction**

A hydrologist at the University of Arizona became convinced that models using the finite element method were required to predict the movement of artificially recharged groundwater. To fully understand the application of this method, he used a year to obtain the necessary background in mathematics, and then attended a one-week summer course with about 30 others on the finite element method presented by Pinder and Gray at Princeton University.

Workshops conducted by University of Arizona faculty have been used for training dispensers and users of computerized hydrologic data provided from models when questioned at various technical levels.

**Modes of Technology Transfer**

The University of Arizona serves most of the Western Region from its RECON terminal---RECON being a computerized information retrieval system located at the Oak Ridge National Laboratory. For example, presently using the key words, mathematical models and Colorado River, abstracts of seven research projects recently completed or underway are provided.

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*University of Arizona, Tucson, Arizona.*
In Arizona, news bulletins, project information bulletins, and tapes carry information regarding research results and availability of models to over 1,000 researchers and users. There is also a Technical Briefing Note series, which provides the Governor and his staff with research results in simple understandable language.

Successful Conferences, Workshops, Seminars

With regard to successful conferences, workshops, and seminars, one can point to an Evaluation Workshop held in Fort Collins, Colorado, in March of this year wherein it was concluded that a digital computer model can satisfactorily be used to simulate irrigation return flows if sufficient data are available. Researchers and decision-makers from universities, U.S. Bureau of Reclamation, and the U.S. Environmental Protection Agency were involved in the development and evaluation processes.

The, of course, one can point to this seminar with regard to information exchange concerning use of models.

Elimination of Difficulty

One real difficulty in successfully developing and using models for solving problems has been the lack of communications between the groups that gather data, conceive models, and make decisions. In at least one case, this difficulty is being alleviated by having all three groups cooperation in the regional U.S. Office of Water Resources Research and Technology project dealing with salinity management options for the Colorado River; six Universities, the U.S. Bureau of Reclamation, and a Technical Advisory Board from the U.S. Pacific Southwest Inter-Agency Committee are closely involved in the project.

Digital models in particular may be frightening to many people—such models require the learning or understanding of appropriate computer languages, and the output may be staggering and difficult to understand. However, recently with the advent of computer graphics, the output from digital models may be presented in a form readily understandable and intuitive, even to the layman.
In this session entitled, "How to Get People to Use Models," I will focus my remarks on how modeling can provide a device for displaying in a convincing manner the results, values, and alternatives to decision-makers. Further, I would like to cite some experience and recommendations on the general topic.

We can get people to use models by requiring their use through terms of grant conditions, contract requirements, or other regulatory measures. A preferable method for getting people to use models is to demonstrate their value to the point that people will voluntarily use them. We at EPA prefer the volunteer approach and in most cases we are optimistic that models can be "sold" on their merit.

Let me sketch for you a brief history of the involvement of EPA and its predecessor agencies in the water quality modeling area. To my personal knowledge, we have been involved in modeling efforts for over 15 years. Early efforts were, by today's standards, rather simple and straight-forward. Early emphasis was on modeling dissolved oxygen behavior in streams receiving organic waste loads. The early modeling efforts could be characterized as solving deterministic problems through the application of analog and digital computers. From the mid-1960s on, we became more and more involved with increasingly complex modeling efforts. The work that culminated in our report entitled, "The Mineral Quality Problem in the Colorado River Basin," linked three modeling efforts: hydro-salinity, detriment assessment, and total

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economic impact, where the output of one served as the input to the next model. The basinwide impact of continued water use practices with and without control programs were quantified.

Public Law 92-500, The Federal Water Pollution Control Act Amendments of 1972, brought forth many additional requirements for EPA, states, communities, and industry. One area in particular spurred the use of water quality modeling programs. This was in response to requirements in the act to develop waste load allocations for all surface waters in the country to form a basis for establishing discharge permit requirements. Given the short time available to undertake this large task, it was necessary to use simple modeling approaches. We expect, however, as water quality standards are revised and refined, as discharge permits come up for re-issuance, and as we have better data available, more sophisticated modeling techniques will be applied.

The intent of PL 92-500 is to place maximum reliance on state and local government to plan and implement water quality programs. Because of this, EPA is moving toward an overview and assistance role in many areas including modeling aspects of water quality planning.

EPA transferred $525,000 to the U.S. Bureau of Reclamation over a 6 year period to develop and test a basin model that can be used to predict the salinity changes resulting from developing a new irrigation project. Ashley Valley was used as a verification site. The report is expected shortly.

In two additional basin efforts amounting to about $415,000, EPA contracted with and received from this University (1) two models that predict the simultaneous movement of salt and water in soils and their response to changes in quality of irrigation water and management; (2) applications of the above models--expanded to predict plant growth and consider other conditions--to specific farm situations. Reports of these are available from our Ada, Oklahoma office.

We would hope that the results of these and other outputs would prove useful to federal, state, and local entities.
From this experience of EPA and its predecessors, I would like to make several observations:

First, in the early days of modeling and to some extent at the present time, modeling was the domain of technical specialists. Decision-makers had an inadequate understanding of the modeling process and how it could be helpful in resolving policy questions. This factor, combined with unhappy experiences with computers and computer programs by managers, combined to produce a distrust by program managers of modeling efforts. It is my belief that the basic solution to this problem is for technical specialists in modeling to convey to managers explicitly how models can be effectively and economically used. The best way to communicate this understanding is through the demonstration of results. I believe that results are most effectively shown when a set of assumptions are listed along with the resulting set of predictions produced by the model, accompanied by a concise explanation of the approach used in the modeling program. This allows the decision-makers to review options and make decisions. Such an approach avoids, "the computer said the answer is _____" syndrome which is so unsatisfactory to most managers.

As an example of modeling output, I would like to refer again to our report on "The Mineral Quality Problem in the Colorado River Basin." This report included the following information:

1. For the 1942-1961 period of record, water use projections developed by water resource agencies, and salt budgets developed using best available information, total dissolved solids at Hoover Dam were projected to increase from 697 mg/l in 1960 to 990 mg/l in 2010.

2. For the types of agricultural, municipal, and industrial water uses below Hoover Dam in 1960, for the least cost alternative option available to all users of degrading water quality, and for the projections of changes in use developed by appropriate agencies, the direct penalty costs or increase in costs over and above 1960 costs were determined to increase to $16 million annually by 2010.
3. For the type of economic activity and inter-regional conditions existing in 1960, for the changes in the conditions as expressed by input-output matrices and based on OBE-ERS projections, and for the reductions in total gross outputs caused by the penalty costs quantified above, the secondary or indirect penalty costs were determined to increase to $9 million annually by 2010.

The knowledge of the increase in user costs resulting from degrading water quality provides insights regarding how much to spend to ameliorate the projected degradation. Additional effort is needed to determine who pays for any remedial programs.

Another area in which I believe the acceptability of modeling could be enhanced is through the continuity of model development and in adapting models to a variety of uses. Too often in the past, an individual or group of individuals will develop a specific model to solve a specific problem but to be forgotten once the immediate goal is achieved. I think in general terms that it is far more effective to develop basic models then build upon and adapt them to varying situations. This implies, of course, a program of inter-change of information among model developers and users--an effort which we at EPA are promoting.

In summary, we at EPA have used and benefitted from modeling for many years. We are convinced that modeling is a valuable tool in arriving at rational decisions and that the value of models can be demonstrated to "doubters" in most cases. Further, we feel that communication is the largest barrier inhibiting model use and that the most effective communication is through demonstration of results.
HOW TO GET PEOPLE TO USE MODELS

by

Myron B. Holburt

I am one of four who were selected to serve on the panel on "How To Get People To Use Models." In order to avoid repetition, the panel moderator, Ival Goslin, asked each of us to cover specific areas relative to the subject. I will briefly discuss: (1) why mathematical models of the Colorado River System are attractive for use, (2) problems in obtaining general acceptance of the models, (3) the features of some of the available models, and (4) the process that was followed in selecting a model for meeting one major problem within the basin.

Colorado River System and Models

The 242,000 square mile Colorado River Basin encompasses areas with major differences in precipitation, climate, natural and man-made features. The river and its tributaries experience wide variations in flow and in quality on a daily, monthly, and yearly basis, with variations extending over periods of many years.

Present use of water and future plans for use by entities in the seven Colorado River Basin states and Mexico have resulted in many policy, legal, environmental, economic, engineering, and political problems. The complexity of these demands for water with the variations in quality and quantity of flow within the basin makes the use of mathematical models not only attractive but a necessity to evaluate and solve the myriad problems facing the planners, developers, and users of the river system. Use of models of the Colorado River System as a tool to analyze various problems allows consideration of a number of alternatives of the many complex interrelationships existing in the basin. There are a

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number of models which have been developed and used in the Colorado River Basin. It may be that a significant issue which this panel should be addressing, is how to get people to select the correct model from those that have already been developed.

It appears that, in many cases, whenever the need for a new study develops, the investigators develop a new model rather than attempt to modify or improve an existing model. As a result, a large portion of available study funds are used to develop a new model that may differ from one of the existing models only by using a slightly different set of basic equations, a new reservoir mixing assumption, or a different numerical analysis technique.

Time and money could be better utilized in trying to understand the problems and potential solutions to river problems rather than spending time on the mechanics of developing and testing models.

Acceptance of Models

One major reason we do not see a greater acceptance of many models is a lack of understanding and confidence on the part of managers and others who would be using the results of model studies. This lack is fostered when the modeler develops an aura of secrecy and unnecessary complexity around his work by using his own set of specialized terminology, and introducing an unnecessary semantics problem. The modeler often talks in terms of object functions, orders of derivatives, statistical significance, and other specialized terms generated by the simulations technician. While he may feel the need to present his model and its product in such terms in order for it to be accepted by his fellow technicians, it would only require a minimal extra effort for him to present the same material in non-technical language, thus enhancing the public understanding and acceptability of his work.

Also, in order to have the model used by other than the developing agency, enough time must be spent to prepare a complete and understandable user manual.
Another problem that sometimes develops to cloud the acceptability of a model occurs when the modeler fails to understand, or has only a superficial understanding, of the problems he is attempting to solve. The modeler must understand the problem at hand, the physical system to be modeled, as well as the legal and organizational constraints. Too often, the modeler says he understands the problem when he really does not and then proceeds to develop a model which does not simulate the system as it really exists nor does it provide usable results. Frequent meetings between the user and the modeler are required. If the project is large, involving a number of users and developers, an advisory group is one means of providing the necessary inter-communication.

By presenting the model results in tabular form or easily understood graphics, rather than in terms of complex statistics, user acceptability will also increase. Further, the results should be in a form that will facilitate rapid checking for reasonability. No matter how sophisticated the model or how many long, complicated equations are used, it must be explainable in terms understandable by potential users. Unless the modeler can do this, the users will not accept his results and there is even a real question whether the modeler really understands his own model.

**Colorado River Models**

Because of the attractiveness of mathematically relating the complex factors of the Colorado River in order to solve the river's many problems, a number of models have been developed and used in the past, and new ones are continuing to be developed. Several of these are briefly discussed.

**Colorado River Storage Project Model**

This model was originally developed by the USBR for the Colorado River Storage Project and was later modified and used for the studies leading to the establishment of the operating criteria for system
reservoirs. This model not only simulated hydraulic conditions but also evaluated the economics of power production. While it appears to have been well designed for its intended task, it did have shortcomings. One major difficulty in general acceptability of the model was the inability for entities other than the USBR to utilize the model on their own computational equipment for independent studies. This failure was due mainly to the special software packages used by the USBR. Also, the users manual was not complete enough to enable people other than the modeler to independently utilize the model or to understand its operation.

Recently, the USBR has modified the program so that it is now usable with any computational equipment. However, a complete users manual is still lacking. Water quality parameters have also been added to this model but quality predictions can be made only for the portion of the river, Lake Powell, and below.

Hydraulic-salinity flow system within the Upper Colorado River Basin

This model was developed in 1970 by Utah State University. This research tool was developed to: (1) Simulate the relationship between the hydraulic and salinity flow systems, (2) demonstrate the utility of electric analog computers for simulation modeling, (3) improve the understanding of the relationship of the hydro-salinity system, and (4) indicate deficiencies in available quality and quantity data. Its major restrictions are that it is limited to the river system above Lake Powell, and is designed for an analog computer, which severely limits its use by other entities. The model has recently been converted to a digital computer.

Colorado River system simulation model

This is the large scale model that the USBR has been developing over the past 2-1/2 years. It consists of two parts: a data generation phase, and a simulation phase. The data generation portion utilizes synthetic techniques to develop projections of water supply and salt load.
It randomly manufactures water supply and salt load input data which have properties similar to historic data. While the water supply facet is working well, the salt loading portion is experiencing some difficulties due to an inadequate data base. Further analyses of the salinity data base need to be made before the model can be used extensively with confidence.

River network model

This model was developed by Richard Ribbens of the USBR. It is a relatively simple salt routing model well suited to salinity projections, is easily understood, well documented, and provides output in a number of easily understood and usable forms. This model's principal limitation is that it only covers the river system, Lake Powell, and below. Also, it lacks many of the simulation capabilities which are desirable in a full systems model.

Selection and Use of a Colorado River Salinity Model

In preparing testimony for the hearings on the Colorado River Basin Salinity Control Bill, in March, 1974, we found it was necessary to make projections of future salinity in the Lower Basin. We considered the preparation of a model but concluded that time was too short to prepare and properly test a model or to investigate and modify an existing model. We decided to limit the number of salt and water routing studies and to conduct them by hand. A small desk top calculator-computer was used to carry out a year-by-year projection for 15 years in the future. These hand computations required approximately 5 to 8 man-days for each salt and water routing study. Although acceptable for the purpose, the results were subject to potential errors both in performing the computations and in transcribing results.

Salinity projections were also necessary in recent work conducted by the Colorado River Basin Salinity Control Forum composed of water quality and water resource representatives of the seven basin states. In
June 1974, the forum undertook to develop Colorado River salinity standards pursuant to regulations of the Environmental Protection Agency that were in response to requirements of Public Law 92-500, the Federal Water Pollution Control Act Amendments of 1972. In developing the salinity standards and a plan of implementation, the forum conducted an extensive study which included projections of future salinity at a number of points along the river. A small work group was established to conduct the study.

The number of alternatives that the work group planned to study precluded the use of hand studies. Rather than attempting to develop its own model, the work group decided to utilize and possibly modify one of the existing USBR models discussed earlier in this paper.

The work group listened to presentations by the USBR on the CRSP, the Systems Simulation, and the River Network models. The CRSP model's major drawback was it extended only to Parker Dam, while the forum studies required projections at Imperial Dam. Further, there was no complete program documentation that would permit independent use.

The System Simulation Model had a number of drawbacks. It was 10 times more costly to operate than the River Network Model. The data generation portion of the model was not functioning well. Test results gave what appeared to be anomalous conditions on the quality side of the model. There was no agreement among the regional offices of the USBR as to the proper data base to use. These problems, combined with the model's use of synthetic hydrology as its data base, resulted in a lack of confidence on the part of the work group members in this model. Finally, inasmuch as the model developer did not believe the model was fully ready for use, it was concluded that this model would not be used.

The River Network Model was selected because of its reliability of results, simplicity, and ease of understanding of its mathematics, well documented users manual, easy input data preparation, and the work group members had confidence in its results. Some minor modifications were made to the model to conform it to the study's needs. It proved to be an efficient tool in meeting the forum's needs. Ernest M. Weber,
Supervising Geologist on the Board's staff will discuss this forum study in more depth in a case study in this symposium.

The River Network Model is very similar to the salt routing studies which have been computed by hand calculations. However, it contains many additional refinements which were possible only because of the use of the computer. With the model working in a satisfactory manner, it is interesting to compare the time required for one solution of the computer model, including output printing, with performing the same analysis using manual calculators. The model requires between five and ten minutes, while manual computations required five days.

It is apparent that the use of the computer model enabled the forum to look at a wide range of alternative water supplies, water use, and salinity control measures, which could not be done otherwise because of time and money limitations.

Summary and Conclusions

Many of the complex problems which face the Colorado River System and its users can best be understood through the formation and use of mathematical models, including hydraulic salt routing, economic, or other types of models. However, when models are developed without a full understanding of the problems or of the system modeled, the results will not only be of suspected validity but will also not be used. Modelers need to recognize that the model is a tool which is to be used to solve the problems and not an end in itself. Open communications must exist between developers and users of models, which will result in the acceptance of the model and its results. This will benefit all those involved in planning, developing, and using the Colorado River System.
The Colorado River is certainly an appropriate one for a seminar on river basin modeling studies. No major river in the United States is more highly regulated and utilized than the Colorado. Interests and purposes run the entire gamut including municipal and industrial water supply, recreation, fish and wildlife habitat, irrigation, flood control, hydroelectric energy production, salinity levels, and a host of others.

The multitude of purposes and interests served by the Colorado is further complicated by the high variability in runoff of the river. Thus operating criteria need not meet only seasonal demands, but must accommodate long-term drought cycles involving the use of carryover storage for a number of years. Early attempts to solve these complex problems in the Colorado involved, as in other basins, laborious hand-computational operation studies. With the advent of digital computers, operation studies were converted to machine processing. Early versions often involved utilizing the computer for the arithmetic with a majority of the decision-making and other logic handled externally. As the speed and capacity of computers increased, so did the sophistication of river routing programs. However, computer capability is only one limitation on the degree of sophistication that can be incorporated into a river basin model. A more real limitation is the availability of detailed input data and documented operating criteria.

With the number of interests and purposes involved in the operations of the Colorado River, aside from the natural evolutionary process, it is not surprising that a number of models have been developed in an effort to provide answers to meet these needs. Also, the specific purpose of the studies usually dictates the time units to be utilized. For long-range

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*U.S. Bureau of Reclamation, Boulder City, Nevada.*
planning studies average monthly data are usually sufficient. Some real
time operating programs may be dependent upon instantaneous telemetry
for input. Time frames for other programs could fall between these ex-
tremes. Availability of data for the different time units is one factor in
requiring different models to serve different ends. Internal logic require-
ments often vary dramatically. For example, time lag in streamflow may
not be particularly important in annual or monthly time frames. However,
it is usually absolutely essential that this time lag be recognized in daily
or even weekly units.
STREAMFLOW SIMULATION WITH THE COLORADO RIVER SIMULATION MODEL

by

Charles W. Huntley, Robert B. Main, and William L. Lane*

Introduction

This paper discusses a computerized river basin simulation model and its application to the Colorado River Basin. The model was developed to provide the user with the capability of varying demand and hydrologic inputs at points throughout the basin, thus, permitting an examination of the effects of these variations on water availability and salinity concentrations in the basin.

The purpose of the paper is to discuss the concept and capabilities of the model. Although example results of a typical run are included to illustrate capabilities of the model, the purpose is not to present or discuss results of a study. Node structure and reservoir demand, and hydrology inputs are discussed. Salient points of the model operation are summarized. Output options are listed. Special features of the model required for adaptation to the Colorado River Basin are an important part of the paper. Example results of a typical model run and associated costs are included.

This model was used for the U.S. Western Water Plan studies in 1974. Since that time, substantial improvements have been made in the model to enhance usability from the user standpoint, to significantly reduce running time and costs, and to streamline calculation procedures in the model. The model is currently being used in the Engineering and Research Center for examination of a variety of salinity questions on the Colorado River.

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The model utilizes the node concept, each node representing a specific reach of river. The node structure which the user sets up for the model forms the pattern for all other inputs and model computational order. Computations were made on a monthly time basis.

Four groups of inputs are required: Node structure; reservoir operational data; demand data; and hydrology data. These are illustrated on the block diagram shown in Figure 1.

Desired output is written as the computations are made or written on a file and extracted after the run is complete depending on the options specified at the beginning of the run.

**Node Control Structure**

The node control structure defined in the input data is designed to allow mathematical representation of a river basin. Node structure refers to the sequence and arrangement of nodes within the basin to be modeled. A specific reach of river is modeled by each node. The simulation model presently has capability to handle 25 nodes. The node structure currently set up for the Colorado River is shown in Figure 2.

Each node is set up to compute flows and salinity at sequence points in the node, the values representing flows and salinities in the river. This computation is made using inflows and outflows and their respective salinities at these sequence points.

A single node can include a maximum of 10 inflow points and 10 demand points. A typical node is shown in Figure 3. A node can include one reservoir which requires one of the ten inflow points. Inflows include such items as inflow from rim areas or intervening areas from the hydrology data file, main streamflow from an upstream node, or return flow from a demand on the river. Demand points (outflow from the node) include diversions from the river. As will be described under "Demand Input," a separate program is available to combine information from up to 10 "users" into the value for one demand sequence point.
Figure 1. River basin simulation model block diagram.
Figure 2. Node diagram for Colorado River simulation model.
Figure 3. Typical node, Colorado River simulation model.
The following table summarizes the potential number of nodes, inflow sequence points, demand sequence points, and users and shows the number of each presently used.

Table 1. Node setup.

<table>
<thead>
<tr>
<th>Item</th>
<th>Number Potential</th>
<th>Number Used(^a)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nodes</td>
<td>25</td>
<td>25</td>
</tr>
<tr>
<td>Inflow sequence points</td>
<td>250</td>
<td>154</td>
</tr>
<tr>
<td>Demand sequence points</td>
<td>250</td>
<td>78</td>
</tr>
<tr>
<td>Users</td>
<td>2,500</td>
<td>129</td>
</tr>
</tbody>
</table>

\(^a\)Currently used for Colorado River model runs.

The node control structure includes three sets of control information. The first set defines node order, node identification, and destination of node outflow. Node order is set up so that calculations begin at the top of the river basin and proceed down the basin to the bottom. Calculations also proceed from the top of a node to the bottom of the node. Thus, flow at all upstream nodes is handled prior to any downstream node it may affect.

The second set defines the sequence of inflow points, demand points, and the reservoir point within the node. Sequence numbers are assigned point by point from top to bottom; positive for inflows, negative for demands, and zero for a reservoir. Care must be taken in assigning inflow and demand points so that basin structure is modeled as closely as possible. This will help to prevent the calculation of negative flows in the river.

The third set of control information defines the upstream node numbers which can provide water for demands within the node.
Reservoir Operational Input

Reservoir operational relationships represented by polynomial equations in this model are as follows: (1) Reservoir elevation-area-capacity; (2) horsepower at full gate versus head; (3) flow rate versus head; and (4) tailwater elevation versus flow rate.

The polynomial is of the form

\[ Y = a_0 + a_1 x + a_2 x^2 + a_3 x^3 + \ldots + a_n x^n \]

The \( a_n \) coefficients are determined by least squares fit and entered as input data.

Other reservoir operational inputs are: (1) Target capacity values for each month called rule curves; (2) bank storage coefficient; (3) evaporation rates for each month; (4) capacity at normal water; (5) maximum and minimum reservoir capacities; (6) maximum and minimum outlet capacities; and (7) beginning reservoir storage and salinities.

Demand Input

The simulation model requires input data on a node and demand sequence point arrangement as described in the section on node control structure. However, demand input is usually set up on a detailed "user" basis and put through a separate program which prepares the information for the simulation model. The user basis model is called Simulation Model Demand Input Data (SMDID).

The user basis allows a more detailed breakdown of demands and allows them to be identified by state and function. SMDID can take demand information from up to 10 users and combine it into the total demand at a single demand sequence point within the node.

Types of information required in setting up the demand data for a node are listed as follows: (1) Withdrawals for a given user at a demand point; (2) depletions (or return flow); (3) year of withdrawal and depletion; (4) base year (coordinated with hydrology modified flow base);
salt pickup; (6) user function (irrigation, municipal and industrial (M&I), etc.); (7) state; (8) node, sequence point, user number; and (9) node and sequence number for return flow location.

An important feature of SMDID is the handling of withdrawal and depletion data as a step function or a linear trend function depending on a flag extended by the user. For example, if data are entered for 1970 and 1980, the intervening years would use the 1970 value and jump to the 1980 value in 1980 if the step function flag is set. If the linear function flag is set, the 1970 value would be incremented each year in a linear fashion until it reached the 1980 value.

Another capability of SMDID is summarizing demand information into various report-type forms. Five different report forms can be produced at the option of the user. These are summarized as follows:

1. Shows information on a biennial basis at the user level with accumulated values shown at the demand sequence point.

2. Shows information by function for nine functions, (irrigation, M&I, fish, etc.) for each node. Data are shown for the first and last year and each decade in between.

3. Shows information by state, upper and lower subbasins, and combined basin totals.

4. Shows information by node and function for the first and last year and intervening decades for each individual state.

5. Shows information by state and function for the first and last year and intervening decades for the two subbasins and total basin.

Hydrology Input

Hydrology inputs to the simulation model are of two forms: (1) Flows and salinities at major rim stations around the periphery of the basin; and (2) intervening flows and salinities between the rim stations and major downstream stations. Downstream flows and salinities are calculated by adding intervening values to rim station values. The intervening values may be either positive or negative.
Hydrology input data are prepared separately and stored on a disk file in the computer, then read from the disk and used in the simulation model computations. The hydrology base presently used in the model is synthetically generated with a separate program (GENHYD). This program uses statistics developed from historic data with appropriate adjustments to the statistics to bring them to the 1970 depletion level. The purpose of using synthetic hydrology was to allow model operation on a large number of hydrologic traces, thereby testing a wide range of flow conditions. The model will also accept historic flows or modified historic flows or virgin flows so long as rim and intervening flows are prepared on the proper node setup and coordinated with the base level of demand data.

The philosophy behind the synthetic hydrology approach used in this application is to define historical streamflow characteristics as completely as possible with cyclical and regressive mathematical relationships. The remaining unexplained or random part of the streamflow variation is treated with probability concepts.

Statistics needed for each rim or intervening flow input to be synthetically generated are as follows: (1) Monthly means; (2) monthly standard deviations; (3) coefficient for Markov model; (4) coefficients for polynomial fit of frequency distribution of residuals; and (5) regression coefficients between flows at this location and flows at other locations.

These statistics are obtained through analysis of historic data with proper regard to changes in streamflow characteristics due to development in the basin. A separate set of computer programs for data analysis are used to analyze the historic data and obtain the necessary information for generation of synthetic flows.

A detailed discussion of the data analysis procedures and synthetic generation procedure is given in the report "Application of Stochastic Hydrology to Simulate Streamflow and Salinity in the Colorado River," by William L. Lane and Albert E. Gibbs, May 1975 (2).
Simulation Model Operation

At the beginning of a run reservoir operational data, initial demand data, and the first year of hydrology inputs are read in. The one year of hydrology data is placed in a temporary file. This temporary file is then accessed during the streamflow forecast procedure and each month for new hydrology data.

The model simulates river basin flows on a monthly time frame starting at the top of the basin and proceeding completely through the basin to the bottom. Simulation is done node by node in the order specified in the node control structure input. Within a node, computations are made from the upstream to downstream end. The general operation of the simulation model is shown schematically on Figure 4.

Riverflows are calculated at each inflow, demand, and reservoir sequence point. All calculations for river flow are based on the continuity equation:

\[
\text{Flow at next sequence point} = \text{Flow from preceding sequence point} + \text{Inflow} - \text{Demand}
\]

When there is not enough flow in the river at a sequence point to supply a demand, a search is made of upstream reservoirs for the additional water needed. If there are upstream reservoirs and they have sufficient water in storage to meet the demand, the additional increment needed is released and routed through the system to the point in need. If adequate water is found, the demand is met and the calculations proceed to the next point downstream. If this second demand cannot be met, the amount of shortage is computed and printed and calculations proceed.

All calculations for reservoir operations are based on another form of the continuity equation:

\[
\text{Change in storage} = \text{Inflow} - \text{Outflow} - \text{Evaporation} - \text{Bank storage}
\]

A reservoir is operated to meet a target end-of-month contents. A release is determined by either a minimum release rate, a flood space storage requirement, demands which draw from the reservoir,
Figure 4. Operational schematic of river basin simulation model.
power releases to meet generation requirements, or demands which draw from upstream reservoirs and move water through the reservoir. There also may be spills from the reservoirs. Evaporation, bank storage change, and power production are calculated each time water is moved through a reservoir.

When calculations have proceeded through the last sequence point of the last node, flows are in balance throughout the entire basin. Salinities are then computed throughout the system by a mass balance accounting procedure. Computations are then complete for the monthly time frame.

Before calculations are started for the next month, hydrology inputs for the new month are updated from the temporary hydrology file. If it is the beginning of a new year, demand and/or reservoir operational input data are updated. Simulation of basin operation for the new monthly time frame is then repeated. This process is repeated through the final month of the last year specified in the input data.

Special Colorado River Basin Features

Several features have been incorporated into the general river basin model to reflect specific Colorado River operations. These include use of snowmelt-runoff forecasts for January-July reservoir operations, distribution of water between the Metropolitan Water District of California (MWD) and the Central Arizona Project (CAP), water splitting between the Upper and Lower Basins and storage requirements of the Upper Basin described in section 602(a) of Public Law 90-537, and flood operations.

The model has a procedure to provide a forecast of spring runoff with the same error properties as actual forecasting during actual operation. In the model, the spring runoff (from the month under consideration through July) is summed from the disk file containing one year of hydrology data. An error term is applied each month which reflects the historical accuracy of the forecast in that month. Thus, in any
month the flow used as the forecasted value will be high or low to the same degree as actual forecasted flows are in field operations.

The legal constraints which govern the flow of water between the Upper and Lower Basins are incorporated in the model through a special subroutine. This aspect of the basin operation uses the forecast flows to determine the monthly release from Lake Powell and Lake Mead. Before the Upper-Lower Basin analysis is run, the Upper Basin reservoirs are analyzed and their rule curves established.

To determine whether Powell will release more or less than the normal 8.23 million acre-feet (10,151.71 million cubic meters) (from a monthly schedule), an estimate is made of Powell and Mead contents for the upcoming October. The forecasted inflow to Lake Powell (from the current month through July) plus the average August and September inflow forms the expected total Powell inflow. The total release from Powell expected through September is calculated as 8.23 MAF minus the amount delivered up to the current month. The October Powell contents are then the current contents plus the expected inflow, minus the planned releases. Similarly, the October contents of Lake Mead can be estimated from the current contents plus expected inflow (same as Powell releases plus gain between Powell and the Grand Canyon gage) minus the planned releases.

Once the October contents of Lake Mead are estimated, a Lower Basin shortage or surplus may be declared. If Mead's ending water surface elevation exceed 1190 feet above sea level (362.7 meters), water above that level is declared surplus and distributed to the Central Arizona Project (CAP) and the Metropolitan Water District of California (MWD). If Mead is below elevation 1124 (342.6 meters), a shortage is declared and CAP is reduced below its normal demand. Between elevations 1,124 and 1,190, the normal release pattern associated with an 8.23 MAF/year total release is followed. The declaration of a shortage or surplus in the Lower Basin does not depend upon the status of Lake Powell. However, water splitting between Powell and Mead may increase the supply to the Lower Basin and affect the declaration.
Water splitting is based upon three criteria: (1) The Upper Basin has adequate water in storage to meet paragraph 602(a) requirements, (2) Lake Powell has more water in storage than Lake Mead, and (3) Lake Mead does not have surplus water.

The water needed to meet 602(a) storage requirements is based upon future Upper Basin depletions. The water currently available to meet 602(a) storage is calculated as the active storage of all Upper Basin Colorado River Storage Projects (CRSP) reservoirs. If there is insufficient water available to meet the requirement, releases from Lake Powell will not exceed those of the 8.23 MAF schedule regardless of the contents of Lake Mead. If 602(a) storage is available, then an additional release can be made from Powell to the Lower Basin provided Mead is storing less than Powell and is not expected to exceed elevation 1190 in October.

The model considers these constraints when assessing the potential for water splitting. It restrains Powell from releasing too much water and violating 602(a) storage requirements, and also releases water from Powell only to the point where Powell and Mead are at equal contents. As this is done, the October contents are reestimated to reflect possible changes for demands by CAP and MWD.

It is possible that in a particular month the Lake Mead flood criteria could force a release greater than that planned by the water splitting analysis. In this case, CAP and MWD demands are increased as much as possible to utilize the excess flows. Water not used by Lower Basin demands is passed on to Mexico and is in excess of the 1.5 MAF (1850.2 million cubic meters) annual delivery.

The flood operation of Lake Mead is intended to follow closely the regulations specified in the Corps of Engineers Flood Control report (1). Essentially three types of criteria govern the releases from Mead: (1) Minimum flood storage space in Lake Mead, (2) specified minimum rates of release determined by release tables, and (3) operations based on inflow forecasts. Each of these will be described separately and related to the mechanics of the computer model.
Flood storage space is defined on the first of each month from August to January. The minimum flood control storage space below elevation 1229 (374.6 meters) is as follows:

<table>
<thead>
<tr>
<th>Date</th>
<th>Min. space required</th>
<th>Million cubic meters</th>
</tr>
</thead>
<tbody>
<tr>
<td>August 1</td>
<td>1,500,000</td>
<td>1,850.2</td>
</tr>
<tr>
<td>September 1</td>
<td>1,500,000</td>
<td>1,850.2</td>
</tr>
<tr>
<td>October 1</td>
<td>1,500,000</td>
<td>1,850.2</td>
</tr>
<tr>
<td>November 1</td>
<td>2,675,000</td>
<td>3,299.6</td>
</tr>
<tr>
<td>December 1</td>
<td>3,963,000</td>
<td>4,888.4</td>
</tr>
<tr>
<td>January 1</td>
<td>5,350,000</td>
<td>6,599.2</td>
</tr>
</tbody>
</table>

Storage space in upstream CRSP reservoirs may apply towards the above requirements within the following limits: (a) Lake Mead must have at least 1,500,000 acre-feet of available space and (b) the maximum creditable storage space shall not exceed the values below.

<table>
<thead>
<tr>
<th>Reservoir</th>
<th>Maximum creditable space</th>
<th>Million cubic meters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lake Powell</td>
<td>3,850,000</td>
<td>4,749.0</td>
</tr>
<tr>
<td>Navajo</td>
<td>1,036,100</td>
<td>1,278.0</td>
</tr>
<tr>
<td>Blue Mesa</td>
<td>748,500</td>
<td>923.3</td>
</tr>
<tr>
<td>Flaming Gorge</td>
<td>1,507,200</td>
<td>1,859.1</td>
</tr>
</tbody>
</table>

The difference between maximum storage space and the current storage is used as creditable space up to the values shown above.

From August through September a rule curve is specified which maintains 1,500,000 acre-feet of flood storage space. In October, November, and December the model applies all creditable upstream space to the storage required. It then checks Lake Mead to provide at least 1,500,000 acre-feet of space in Lake Mead.

Beginning in January and continuing through July, the forecast values are used to regulate releases and contents of Lake Mead. The flow of the Colorado River at Grand Canyon is forecasted and summed from January to July, February to July, March to July, etc., until July is reached. The objective is to operate Lake Mead to handle all
inflows such that 1,500,000 acre-feet of flood storage is available on August 1. When inflows are not extremely high or the reservoir is well below flood levels, normal releases to meet demands in the Lower Basin are adequate. In years of high spring runoff, the Corps of Engineers report specifies the minimum average monthly releases to be made for January through June.

To determine the minimum release two quantities are calculated. First, the maximum runoff from the current month through July is computed as Lake Mead natural inflow adjusted for effective storage space in upstream reservoirs excluding Lake Powell. Effective storage space is calculated as the smaller of either the actual space available or the difference between the minimum forecasted inflow and normal releases. The minimum forecasted inflow is obtained by reducing the previously forecasted value by 1.645 times the standard error of estimate for each reservoir. This produces a flow value which can be expected to be exceeded 19 out of 20 times. The second quantity computed is the space in Lake Mead below elevation 1229 (374.6 meters) plus space in Lake Powell below elevation 3700 (1127.8 meters) at the first of each month. These quantities are calculated as the maximum reservoir capacity minus the previous end-of-month contents. A function subroutine is called with the above quantities and computes the release as specified by the Corps.

In each month of the model calculations from January through July, the criteria above are analyzed to determine the operation of the reservoirs. This operation plan is used in the model either as a required release or a control on the final reservoir contents. When these constraints are met the basin simulation proceeds.

Output Options

A variety of output options are available on the river basin simulation model. These are as follows:
1. Forecast Summary. The summarized results of the forecast procedure are shown for each month a forecast is made.

2. Point-to-Point Flows and Salinities. A print is made of main-stream flows and salinities, and demands on input flows and salinities for every demand and inflow sequence point in every node. This information can be obtained for a specified month in a specified year, or all months in the year, or all months in all years.

3. Monthly Reservoir Summary. A monthly summary is printed of reservoir operation results for each node including inflow to and outflow from a node. This summary can be obtained for any month or all 12 months.

4. Annual Summary. An annual summary and statistics of flow and flow weighted salinity is printed for key stations in the Colorado River Basin. Upon completion of a number of runs the annual values at a specific station can be summarized and statistics computed.

5. Tape Edit. This option writes the monthly results of an entire run for every sequence point in every node on a disk file. A separate program called TAPEDT extracts information from the disk file and puts it in a form specified by the user. Information that can be extracted is: a. Flow and salinity input to the mainstream; b. demands requested and actually diverted; c. consumptive use and shortage criteria; d. reservoir operation results and criteria; e. flow and salinity in the main stream; and f. powerplant operation results and criteria.

The following operations can be done on this information: a. Perform math transformations (add, subtract, multiply, or divide by a constant); b. compute and print statistics; and c. compare one parameter against another.

The information extracted and the results of the TAPEDT operations on the information can be output in the following forms: a. Print on highspeed printer; b. punch data cards; and c. plot on microfilm.

Careful consideration of output options and their use is essential since computer costs go up rapidly with increased output.
Associated Programs

Programs associated with the river basin simulation model are summarized in this section in the order of use for a complete study.

1. Data Analysis Programs. These programs are used to analyze historic data if hydrology inputs to the simulation model are to be synthetically generated. These programs do time series analysis and removal of cyclistic components, analysis of probability distributions, and multiple regression analysis. Coefficients are computed which are input to the synthetic hydrology generation program.

2. Synthetic Hydrology Generation Program. This program uses the coefficients prepared in the data analysis sequence and recreates a streamflow sequence ready for input to the river basin simulation model.

3. Simulation Model Demand Input Data Program. This program takes demand data in a detailed "user" form, summarizes it into any of several report type forms for easy examination and report presentation, and accumulates "user" data into a composite value for a demand sequence point and in the required format for input to the river basin simulation model.

4. River Basin Simulation Model. This program uses hydrology, demand, and reservoir operational inputs, all based on a node-sequence point configuration to compute flows in the mainstream of the basin. Special features of the Colorado River operation are included as special subroutines. A variety of output options are available to fit the user's needs.

5. Tape Edit Program. This program analyzes a special output disk file from the simulation model run if it has been written. A variety of parameters can be printed, punched on cards, or plotted on microfilm.
Example Model Run Results

Results of one series of model runs are included for illustrative purposes. These runs used demand inputs from the Westwide Studies, with some modifications. These demands totaled 3.0 million acre-feet (3,700.5 million cubic meters) depletion in the Upper Basin in 1970 and increased to 5.5 million acre-feet (6,784.2 million cubic meters) in year 2000. Lower Basin demands totaled 6.1 million acre-feet (7,524.4 million cubic meters) in 1970 and increased to 7.5 million acre-feet (9,251.2 million cubic meters) in 1988 and remained at 7.5 million there after.

Table 2 is an example of the summary of annual values of flow and salinity at selected stations for 1 of 30 hydrologic traces. Flows are in 1,000 acre-feet increments and salinity is in milligrams per liter \((1,000 \text{ acre-feet} \times 1.2335 = \text{million cubic meters})\).

Typical Model Run Costs

The river basin simulation model is presently set up to run on the Bureau of Reclamation's Control Data Corporation CYBER 70/Model 74-28 computer at the Engineering and Research Center, Denver, Colorado. The model requires 150,000 octal words of storage.

A typical breakdown of central processor unit time used for major functions in the model is shown in Table 3. This time breakdown relates to one 26-year run which printed only the summary table of annual values of flows and salinity at the selected stations.

Simulation model costs associated with several typical runs are shown in Table 4. These costs vary with the amount and type of output desired.

When synthetic hydrology data are used, one trace is generated at a time and all sets of demand data are run through the model before going to the next hydrologic trace. Generation costs are thus reduced.
Table 2.

<table>
<thead>
<tr>
<th>YEAR</th>
<th>FLOQ</th>
<th>QUAL</th>
<th>FLOW WEIGHTED QUALITY (IN/ML) AND REGULATED FLOWS (THOUS. ACRE FEET) ON A YEARLY BASIS AT SELECTED POINTS</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>TISD</td>
<td>GREATH</td>
<td>GREEDR</td>
</tr>
<tr>
<td>FLOW 1975</td>
<td>1523.</td>
<td>1337.</td>
<td>2836.</td>
</tr>
<tr>
<td>QUAL 1977</td>
<td>591.</td>
<td>949.</td>
<td>531.</td>
</tr>
<tr>
<td>QUAL 1979</td>
<td>591.</td>
<td>333.</td>
<td>346.</td>
</tr>
<tr>
<td>FLOW 1979</td>
<td>3112.</td>
<td>2953.</td>
<td>3413.</td>
</tr>
<tr>
<td>QUAL 1980</td>
<td>651.</td>
<td>865.</td>
<td>341.</td>
</tr>
<tr>
<td>FLOW 1980</td>
<td>3715.</td>
<td>1631.</td>
<td>5820.</td>
</tr>
<tr>
<td>QUAL 1984</td>
<td>532.</td>
<td>413.</td>
<td>437.</td>
</tr>
<tr>
<td>FLOW 1984</td>
<td>3972.</td>
<td>1652.</td>
<td>5713.</td>
</tr>
<tr>
<td>FLOW 1985</td>
<td>5625.</td>
<td>1845.</td>
<td>5211.</td>
</tr>
<tr>
<td>FLOW 1986</td>
<td>2747.</td>
<td>1239.</td>
<td>2509.</td>
</tr>
<tr>
<td>QUAL 1987</td>
<td>735.</td>
<td>419.</td>
<td>409.</td>
</tr>
<tr>
<td>FLOW 1987</td>
<td>7598.</td>
<td>2737.</td>
<td>5522.</td>
</tr>
<tr>
<td>QUAL 1989</td>
<td>683.</td>
<td>367.</td>
<td>419.</td>
</tr>
<tr>
<td>QUAL 1990</td>
<td>517.</td>
<td>385.</td>
<td>454.</td>
</tr>
<tr>
<td>FLOW 1990</td>
<td>5366.</td>
<td>724.</td>
<td>2792.</td>
</tr>
</tbody>
</table>
Table 2. Continued.

<table>
<thead>
<tr>
<th>YEAR</th>
<th>CIZCO</th>
<th>GREEN-D</th>
<th>GREEN-RA</th>
<th>ALUFF</th>
<th>INDIANPO</th>
<th>AT-LEES</th>
<th>A-MOGUA</th>
<th>A-DAVIS</th>
<th>A-PARKA</th>
<th>A-WPUL</th>
</tr>
</thead>
<tbody>
<tr>
<td>FLOW 1991</td>
<td>4.399</td>
<td>3.896</td>
<td>3.845</td>
<td>3.174</td>
<td>1.312</td>
<td>0.834</td>
<td>2.907</td>
<td>2.763</td>
<td>2.616</td>
<td>3.594</td>
</tr>
<tr>
<td>QUAL 1993</td>
<td>775.</td>
<td>454.</td>
<td>724.</td>
<td>142.</td>
<td>940.</td>
<td>960.</td>
<td>787.</td>
<td>760.</td>
<td>765.</td>
<td>935.</td>
</tr>
<tr>
<td>FLOW 1993</td>
<td>2.272.</td>
<td>0.22</td>
<td>1.063</td>
<td>0.236</td>
<td>4.081</td>
<td>3.249</td>
<td>3.994</td>
<td>2.132</td>
<td>2.173</td>
<td>2.101</td>
</tr>
<tr>
<td>FLOW 1996</td>
<td>0.722</td>
<td>0.525</td>
<td>0.434</td>
<td>1.194</td>
<td>1.213</td>
<td>0.829</td>
<td>9.936</td>
<td>9.912</td>
<td>8.933</td>
<td></td>
</tr>
<tr>
<td>FLOW 1997</td>
<td>0.132</td>
<td>0.262</td>
<td>0.315</td>
<td>0.173</td>
<td>0.583</td>
<td>0.833</td>
<td>0.975</td>
<td>0.912</td>
<td>0.666</td>
<td>0.573</td>
</tr>
<tr>
<td>FLOW 1998</td>
<td>0.058</td>
<td>0.113</td>
<td>0.327</td>
<td>0.173</td>
<td>0.583</td>
<td>0.833</td>
<td>0.975</td>
<td>0.912</td>
<td>0.666</td>
<td>0.573</td>
</tr>
<tr>
<td>FLOW 1999</td>
<td>0.014</td>
<td>0.049</td>
<td>0.321</td>
<td>0.131</td>
<td>0.251</td>
<td>1.592</td>
<td>9.914</td>
<td>9.915</td>
<td>9.915</td>
<td>9.915</td>
</tr>
<tr>
<td>FLOW 2000</td>
<td>0.014</td>
<td>0.049</td>
<td>0.321</td>
<td>0.131</td>
<td>0.251</td>
<td>1.592</td>
<td>9.914</td>
<td>9.915</td>
<td>9.915</td>
<td>9.915</td>
</tr>
</tbody>
</table>

26 YEAR AVERAGE ANNUAL VALUES

<table>
<thead>
<tr>
<th>FLOW WEIGHTED QUALITY IN MG/L</th>
<th>662.</th>
<th>423.</th>
<th>441.</th>
<th>598.</th>
<th>583.</th>
<th>658.</th>
<th>776.</th>
<th>792.</th>
<th>812.</th>
<th>951.</th>
</tr>
</thead>
<tbody>
<tr>
<td>AVERAGE SALT MASS IN 1002 TONS/YEAR</td>
<td>4.334.</td>
<td>411.</td>
<td>2526.</td>
<td>1022.</td>
<td>988.</td>
<td>1511.</td>
<td>1033.</td>
<td>1033.</td>
<td>8129.</td>
<td>8129.</td>
</tr>
<tr>
<td>AVERAGE FLOW (THOUS. ACRE FEET)</td>
<td>4314.</td>
<td>1422.</td>
<td>4211.</td>
<td>1261.</td>
<td>1679.</td>
<td>1472.</td>
<td>9792.</td>
<td>9643.</td>
<td>7369.</td>
<td>6343.</td>
</tr>
</tbody>
</table>
Table 3. Typical central processor unit (CPU) time break-down by job function.

<table>
<thead>
<tr>
<th>Function</th>
<th>CPU sec</th>
<th>Percent of total</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Initialize</td>
<td>3.6</td>
<td>9.5</td>
</tr>
<tr>
<td>2. Read hydrology tape</td>
<td>2.2</td>
<td>5.9</td>
</tr>
<tr>
<td>3. Surplus analysis</td>
<td>5.1</td>
<td>13.5</td>
</tr>
<tr>
<td>4. Read scratch hydrology</td>
<td>.3</td>
<td>.8</td>
</tr>
<tr>
<td>5. Balance the system</td>
<td>17.7</td>
<td>46.8</td>
</tr>
<tr>
<td>6. Route salt</td>
<td>7.6</td>
<td>20.2</td>
</tr>
<tr>
<td>7. Write to tape edit</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>8. Print summaries</td>
<td>.9</td>
<td>2.3</td>
</tr>
<tr>
<td>9. Read transaction cards</td>
<td>.3</td>
<td>.7</td>
</tr>
<tr>
<td>10. Print shortage messages</td>
<td>.1</td>
<td>.3</td>
</tr>
<tr>
<td>Total time used</td>
<td>37.8</td>
<td>100.0</td>
</tr>
</tbody>
</table>

Table 4. Typical model run costs for 26-year run.

<table>
<thead>
<tr>
<th>Output Option</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>(a) Annual values of flow and salinity</td>
<td>$12</td>
</tr>
<tr>
<td>(b) Option (a) plus writing all information for the tape edit analysis</td>
<td>$17</td>
</tr>
<tr>
<td>(c) Write all output information plus tape edit file</td>
<td>$50</td>
</tr>
<tr>
<td>(NOTE: Writing all output is very costly - this output is normally used only for debug purposes.)</td>
<td></td>
</tr>
</tbody>
</table>

The cost for generating a typical 30-year trace for the present node and sequence point setup is less than $4. The cost for a typical tape edit program run is about $10.
Development Problems

A number of problems have arisen as the simulation model and associated programs were developed. The most significant of these are listed here hopefully to provide guidance to others who are developing large complex computer programs and simulation models.

Adequate program documentation was not accomplished during model development. With the turnover of personnel associated with this modeling effort, continuity of work was thus made quite difficult. It is recommended that for future computer program development of this type that documentation be carefully written as a first step and the program then developed to accomplish the desired result.

Because of an operational application to the Western U.S. Water Plan Studies both program documentation and systematic debugging efforts were delayed. The operational application was an excellent learning and debugging process for that set of conditions, but did not test the range needed to adequately debug the model.

A number of technical problems associated with generation of synthetic streamflows were solved. Significant problems still remaining relate to analysis of the historic data base for statistics to be used in generation, generation of salinities for intervening inflows, and correlation of these intervening flows with other flows in the basin.

Analysis of the historic data is complicated by development in the basin which may cause different periods of streamflow records to be incomparable. Corrections must be made to bring the data to a uniform depletion level. Also the length of available records varies greatly from station to station within the basin.

Generation of salinities within a reach of stream are based on statistics derived from net values of flow and net values of salinity. These net values are made up of both historic inflows and historic outflows (demands) from a reach. If inflow and outflow from a reach are nearly the same the net inflow to the reach is essentially zero. There may be a movement of salt into or out of the reach but it is difficult to
model because the net flow is zero and thus, this implies no carrier for the salt. Also correlation of the net values with other streamflows is difficult.

**Present Model Status**

The river basin simulation model is nearing the end of the current phase of refinement and is being documented. Model runs are being made for the Water Quality Office in the Engineering and Research Center.

The synthetic hydrology generation program has been revamped to streamline computation procedures and significantly reduce running time. This alteration is essentially complete and documentation of the program is partially complete.

The demand input data program for summarizing detailed input data is operational and a user's manual is available.

The tape edit program presently is not operational because of a change made in the format of the large output disk file written by the simulation model. Documentation is partially complete. The program will be made operational in the near future.

The data analysis programs presently are not operational on the Bureau's CYBER 70 computer, not having been converted from the CDC 3800 status when used for the Western U.S. Water Plan Studies. Conversion, streamlining, and program documentation for these data analysis programs will be done in the near future.

**Future Model Improvements**

The studies accomplished to date have shown areas where refinements and improvements should be made on the model and associated programs. These are listed in two groups, the first relating to the synthetic hydrology generation program, and the second relating to the river basin simulation model.
Suggested improvements for synthetic hydrology generation:

1. Reanalyze the historic data base, incorporating as much of the data for the basin as possible.

2. Add, as an option, the capability of generating total flow at downstream stations, but retain the present capability of generating intervening inflows if so desired. The present treatment of intervening inflows as net values should be reexamined for the possibility of separating the inflow and outflow components.

3. Investigate additional techniques for computing mathematical and statistical parameters from historic data and for regeneration of flows in an effort to better preserve the characteristics of historical flows.

Suggested improvements for the river basin simulation model:

1. Additional streamlining of calculation procedures including checking of input data for missing or bad data.

2. Activating and completing the capability of handling demands on a priority basis.

3. Completing and improving the power generation computation capabilities presently in the model.

Conclusions

Complex problems arising from past and proposed river basin development can be studied over a wide range of conditions using simulation models and large capacity computers. River basin simulation models, such as described in this paper, present the opportunity to examine the results of a series of solutions. The use of synthetically generated data allows analysis of the effects of a large number of flow sequences and development of information on a probability basis. Some problems have arisen during the development of this model and associated programs and data base inputs. When documentation is complete the river basin simulation model will be usable by others.
Acknowledgments

The river basin simulation model described herein was developed by the Engineering and Research Center, Bureau of Reclamation, Denver, Colorado, with assistance from the Bureau of Reclamation Regional Offices in Salt Lake City, Utah, and Boulder City, Nevada, and the Hydrology and Water Resources Program at Colorado State University, Fort Collins, Colorado.

The initial concepts were developed and early work on the application of synthetic hydrology to Bureau of Reclamation studies was performed by Albert E. Gibbs and Eugene A. Cristofano, both with the Engineering and Research Center, Denver. As others have become involved in the effort, Mr. Gibbs and Mr. Cristofano have played a major role in guiding the work and in providing training. Current work to improve usability of the simulation model is being accomplished by the authors under the guidance of Mr. Gibbs. Mr. Main is presently improving the simulation model by streamlining calculation procedures, finding methods to reduce costs per run, and making other refinements to make the program easier to use. Dr. Lane is improving calculation procedures and running costs of the synthetic generation program. Individuals who have contributed significantly to the development of the model and associated computer programs but are no longer involved are: Alden L. Briggs, Lower Colorado Region, Bureau of Reclamation, Boulder City, Nevada; R. Wayne Cheney, Upper Colorado Region, Bureau of Reclamation, Salt Lake City, Utah; Dr. Keith Eggleston, formerly of the Engineering and Research Center, Bureau of Reclamation, Denver, Colorado; Dr. John D. Hendrick, formerly of the Engineering and Research Center, Bureau of Reclamation, Denver, Colorado; Brent W. Paul, Engineering and Research Center, Bureau of Reclamation, Denver, Colorado; and Dr. V. Yevjevich (consulting basis), Colorado State University, Fort Collins, Colorado.
References


COLORADO RIVER STORAGE PROJECT MODEL

by

Wayne Cheney *

Background

During the planning phase of the Colorado River Storage Project, all of the river operation and reservoir sizing studies were done manually. After considering the many alternative plans studied, it is clear that a model for the Colorado River has been necessary for some time.

After the by-pass tunnels at Glen Canyon were closed in 1963, periodic operation plans were made. Operation plans became very important as the reservoirs gained significant power head. It wasn't long until 5 or 6 people dropped everything they were doing for about 10 days every month to do operations studies. By 1965, the first version of this model became functional out of sheer necessity. At first it was little more than an automated hand study. Gradually, the basic structure took shape. Decision blocks were manipulated from time to time as experience was gained and engineering judgment could be applied. During 1969 and early 1970, formal operating criteria were developed. The model was used to test these criteria and provide a basis for their acceptance by the states and other interested agencies. The formalization of the operating criteria gave the model solid footing for making operating decisions. For the first time it was capable of making its own operating decisions. During the past five years, the model has shown us some areas of the operating criteria that need more definition. It is hoped that the pending operating criteria review will result in this additional definition.

*U.S. Bureau of Reclamation, Salt Lake City, Utah.
Structure

The name "CRSP Model" is really a misnomer. As can be seen in Figure 1, it not only encompasses the Colorado River Storage Project, but the entire basin. It ranges from the headwaters of the Green River, Colorado River mainstream, Gunnison River and the San Juan, through the tremendous storage features of Lake Powell and Lake Mead, to the last major diversion point within the boundaries of the United States.

Physical features of the system are described as accurately as mathematics allow. Distance and travel time are not applicable because the smallest time increment is one month. All other physical properties are incorporated in the mode.

Parameter Matrix

The different parameters can be discussed with the aid of Figure 2. They have been separated into three groups: water group, salinity group, and power group. The entire salinity group is not used for reservoirs above Glen Canyon. The model has capability to operate salinity in these upper reservoirs, but as yet, the effort required to develop data for those response points will not be compensated by the expected increase of information. Sediment is used only in those reservoirs that trap significant sediment loads. Imperial Dam is not operated as a reservoir but only as terminal response point.

Water Supply

The water supply is taken from the 1906-1972 period of flows modified to the 1968 level of depletion. Thirteen different hydrologic traces may be chosen from the base data. The first of these traces equates 1906 flows with the starting study year; the second equates 1911 flows with the starting study year, and so on, each shifting the flow data by 5 years until 1966 is the starting study year. (See Figure 3.) In order
to make the cycle complete, the flow data for 1906 and thereafter is assumed to follow that of 1972. Ideally, each hydrologic trace should differ by a shift of only one year, resulting with 65 traces. Choosing 13 traces to represent all 65 assumes a linear difference which could be approximated by interpolation. Computation time is reduced 500 percent and any loss of accuracy is not thought to be significant.

Future basin depletions are subtracted from the water supply at appropriate times and locations. Therefore, as future basin development occurs, it is affected by a corresponding reduction in the water supply.

**Limitations**

The CRSP model is clearly an operations model. It will not describe project effects, such as Animas-La Plata, Dallas Creek, Central Arizona, Central Utah, etc., except to show their effects on the whole basin. It is not convenient nor practical to add new features or additional parameters. Its main purpose is to demonstrate the ability of the Upper Basin to meet the Lee Ferry commitment.
Figure 1.
## C.R.S.P. Model Parameter Matrix

### Time Frame - Monthly

<table>
<thead>
<tr>
<th>RESPONSE POINT</th>
<th>Water Group</th>
<th>Salinity Group</th>
<th>Power Group</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1000 AC. FT</td>
<td>1000 AC. FT</td>
<td>1000 TONS</td>
</tr>
<tr>
<td>Fontenelle</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Flaming Gorge</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Blue Mesa</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Morrow Point</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Crystal</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Navajo</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Glen Canyon</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Hoover</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Davis</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Parker</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Imperial</td>
<td>X</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
WATER SUPPLY

Figure 3.
The need to assess the impact of various salinity control projects and to determine the effect of future developments on the salinity of the Colorado River led to development of a simple accounting or salt routing model. It was developed prior to completion of the stochastic river model to provide interim results. The model does not consider power generation and was designed primarily as a project planning tool.

Descriptions of available existing models were used to determine their suitability to meet study requirements. Although none satisfied all requirements, ideas and concepts embedded in them were borrowed freely for incorporation into a model for use on the Colorado River.

During model design, special effort was made to develop a general model applicable to other river basins. The resulting model was programmed in Fortran IV for solution by a large second generation computer. It has been run on a Control Data Corporation (CDC) 3800 computer and is presently operational on the Bureau of Reclamation Cyber 74-28 system which employs CDC-6400 and CDC-6600 central processors.

The program was applied to the Colorado River (Ribbens and Wilson, 1973). It has since been used by the Colorado River Board of California for use on the Colorado River and by the Water Resources Research Institute of the University of Wyoming for application to the North Platte River. It is currently being used by the Bureau of Reclamation.
Lower Missouri Regional Office for application to front range projects of Colorado.

The program is completely general in the sense that all inputs are specified by the user through punched card input. There are no special control or driver routines required. Special subroutines or changes to existing ones are also unnecessary. Emphasis is on ease of input and flexibility on output. Results can be obtained in both printed and graphical forms. Extensive error checking of inputs is included to help the user avoid useless runs. Experience has shown this to be valuable to the occasional user. A user's manual is available (Ribbens, 1973).

**Program characteristics**

This program may be referred to as a river network model or a flow and salt accounting model. Although the program incorporates several features common to a simulation, it is more accurately described as an accounting or bookkeeping system. It does permit operation of a river system, routing flows and salts through a system which can include reservoirs and a branching network. Reservoir operations are fairly flexible, being specified by the user. A basic time frame of one month is used.

The quality system is treated as a conservative one. Thus, the precipitation and dissolution of salts are not explicitly simulated. They may be roughly accounted for by proper use of the water and salt inputs, but this presupposes that the user has this information at his disposal. This input requirement, placed upon the user, highlights the difference between the accounting and simulation methods. If the chemical reactions and pathways were simulated, the output information from the simulation would be precisely the inputs required for the accounting model. However, the simulation model would also require additional inputs to define soil and water chemistry characteristics.
The term salinity means the total dissolved solids. Salinity units for most inputs are specified by the user. For example, if imports are included, either the mass of salts in 1,000 tons or the concentration in parts per million (ppm) or tons/acre-foot can be specified. Similarly, output quality units may be selected as any combination of these.

Input data include the system configuration; reservoir characteristics, parameters, initial conditions, evaporation rates, and operating criteria; upstream and downstream boundary inputs; water use inputs including import/exports, exports, irrigation, and variable exports whose magnitudes depend on the availability of water; and run and output options.

Output includes printed and cathode ray tube (CRT) plots of results at various river locations and reservoirs. Initial input data, detailed month-by-month results, and a concise summary for the entire run including simple descriptive statistics can be printed. Computed flows and salinity at the downstream boundary may also be written on magnetic tape and saved for future use.

Input Data and Definitions

System definition - node and element concepts

The configuration of the river system is defined by means of element data cards. Only tree-type branching systems are permissible, resulting in a single downstream boundary. Model nodes which correspond to an exact geographical location on the river are first located. They are chosen to coincide with existing gaging stations, to locate points at which model output is desired, to delimit portions of the river system such as reservoirs or river reaches, and to subdivide lengths of the river to obtain the desired resolution for water use inputs. Nodes are consecutively numbered using integer values starting at 1. However,
there is no restriction on their order: Node 1 can be downstream from Node 20 and upstream from Node 7.

Elements are defined as that portion of the river system between geographically adjacent nodes. The only exception is for upstream boundary elements which have no upstream node. Consequently, nodes represent a single location and elements embody the dimensions of length or area. By convention, elements assume the number of the downstream node. All inputs for an element must reference this element number.

Five types of elements are allowed, with the type specified by the numeric code ITYPE on the element data card. They are:

<table>
<thead>
<tr>
<th>ITYPE</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Upstream boundary</td>
</tr>
<tr>
<td>2</td>
<td>Downstream boundary</td>
</tr>
<tr>
<td>3</td>
<td>River reach</td>
</tr>
<tr>
<td>4</td>
<td>Junction</td>
</tr>
<tr>
<td>5</td>
<td>Reservoir</td>
</tr>
</tbody>
</table>

Both water and salt are introduced to the system at the upstream boundary elements. These generally provide the main driving force or input to the system. Desired flows at the downstream boundary are specified as target values which may or may not be realized. River reach elements are assumed to have no storage properties (surface or bank) and merely route flows from the upstream to downstream nodes encompassing the element. Reservoir elements provide for storage of water (and salt) and may be operated in a variety of ways as specified on input. Both evaporation and bank storage are included. The confluence of several river branches is accomplished through use of the junction elements.

Provisions are also included to identify each node by a descriptive 24-character label. Each upstream boundary, junction, and reservoir element can also be identified by a label.

Element data completely define the system configuration. It includes the element number and type plus the number of the element
immediately downstream. The only exception is for the downstream boundary element which has no element below it. Additional information contained on the junction data card includes the number of upstream branches entering the junction as well as the element numbers. This information provides a cross-check on the element data.

Figure 1 illustrates the node locating and numbering procedures as well as the numbering and typing of elements for a hypothetical river basin.

Only river reach and reservoir elements can have inputs to account for additions or withdrawals of water and/or salt.

Node numbers are used for internal subscripting purposes. Element numbers are considered as external and are employed by the user for all inputs referencing a given element. However, the program internally assigns subscripts for each element type, starting with 1 for the first element referenced by any input card. They are cross-referenced to the external element numbers. Consequently, rearranging the input deck can result in different internal subscripts, although results should be the same. The internal subscripts appear on the listings when initial inputs are requested to be printed. These are useful for debugging purposes.

Reservoir inputs

Reservoir inputs include basic parameters such as area-capacity curves; outlet works capacities, and bank storage coefficients; evaporation rates; initial conditions; and criteria for single and multiple reservoir operations. Rather than using elevations for items such as the top of the inactive storage pool or the spillway crest, the program requires the corresponding reservoir surface storage volume.1

1The term reservoir surface storage volume or surface volume is used to signify the volume of water contained in the reservoir proper and excludes bank storage.
Figure 1. Sample problem system, configuration and inputs.
Consequently, when the term reservoir level is used, it is considered synonymous with the surface volume.

Volumes for all inputs must be placed on a consistent basis. Normally, all volumes are referenced to an empty base condition and include dead storage. However, it is permissible to subtract dead storage from all volumes, using the top of the dead storage pool as the reference level. However, in this case water will only be removed by evaporation and reservoir inputs down to the dead storage level. Further depletions are not permitted.

The individual input items are now discussed in detail.

**Reservoir inputs - basic parameters**

1. **Bank storage coefficient.** The bank storage coefficient is defined as the ratio of the volume of water stored in or released from the soils and aquifers surrounding the reservoir to the corresponding change in the surface storage volume. Mathematically:

   \[
   \text{BANK} = \frac{V_B}{V_S} \quad \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \quad (1)
   \]

   where **BANK** is the bank storage coefficient (dimensionless).\(^2\)

   - \(V_B\) is the change in volume of water in bank storage \(\quad (L^3)\)
   - \(V_S\) is the change in volume of water in reservoir surface storage \(\quad (L^3)\)

   The total change in storage is:

   \[
   V_T = V_B + V_S
   \]

   or

   \[
   V_T = (1+\text{BANK}) V_S \quad \ldots \ldots \ldots \ldots \ldots \ldots \quad (2)
   \]

   where **\(V_T\)** is the total change in storage \(\quad (L^3)\)

---

\(^2\)Units for each variable are indicated in parentheses by the symbols \(L\) for length and \(T\) for time, and dimensionless if there are no units.
In practice, $V_T$ is either the inflow to or release from a reservoir. For input-output purposes, the surface storage volume is used and can be obtained by rearrangement of Equation (2).

The bank storage coefficient is assumed to be a single valued, nonnegative constant. It is independent of whether the reservoir is filling or lowering. This assumption may be very poor for the initial filling cycle since a significant portion of the water entering dry formations is lost to water retention by the soil particles. It is also assumed that water enters or leaves the bank formation within the basic month time frame. Consequently, volume changes at the end of the time frame are essentially instantaneous.

As a consequence of these assumptions, if there is no change in surface storage during a month, there will be no flow into or out of the banks. In reality, this may not be true because of the dynamics of groundwater flow in the aquifers. The hydraulic nature and geographical location of the aquifer boundaries, transient effects of previous operations, and the slow drainage of soil materials may all be significant.

Consequently, the program approach must be considered a crude abstraction of reality. The bank storage coefficient should be evaluated in terms of an effective or active bank storage component. Thus water lost to specific retention and to aquifer storage great distances from the immediate reservoir are excluded. If a detailed accounting of these losses is desired, they can be included in an approximate fashion. For example, for the initial filling of a reservoir, water lost to remote portions of the aquifer can be included as an export if either field data, past studies, or a detailed aquifer simulation provides estimates of the quantities involved. An initial run would be required to determine the filling cycle, with a subsequent run including the losses of water and salt by adding an export of appropriate magnitude.

(2) Area-capacity relationship. Area-capacity curves are required to define the reservoir surface area for evaporation computations.
The program uses the fourth-order polynomial:

$$A = C_1 + C_2 V_S + C_3 V_S^2 + C_4 V_S^3 + C_5 V_S^4$$  \hspace{1cm} (3)

where $A$ is the reservoir surface area \( (L^2) \)

$C_1$ to $C_5$ are coefficients in the polynomial \( \text{various units} \)

The coefficients in the equation must be determined outside the program using one of the multiple regression programs available on most computer systems and programmable desk calculators. They are computed by entering pairs of values of the surface storage volumes and corresponding surface areas. Coefficients that are insignificant or unused may be left blank on input.

(3) Contents and outlet works capacities. Three levels or volumes are used to divide the reservoir into four zones with associated minimum and maximum outlet works capacities, as shown in Figure 2. They are:

Zone I - The DEAD STORAGE VOLUME marks the top of the dead storage, conservation, or inactive storage pool. When levels are in this zone, no water can be passed through the outlet works. The extent of this zone may be modified through use of variable constraints as described later.

Zone II - The SPILLWAY CREST VOLUME marks the top of the normal operating zone which is immediately above the dead storage zone. When the water level is in this zone, all releases are assumed controlled and the minimum required release is zero. The maximum allowable release is based on the MAXIMUM OUTLET CAPACITY.

Zone III - The top of the spill zone is given by the DAM CREST VOLUME. When the water level is in the zone between the spillway and dam crests, provision is

---

3 For example, the well-known BMD programs.
Figure 2. Contents and outlet capacities.

<table>
<thead>
<tr>
<th>ZONE</th>
<th>RESERVOIR LEVEL</th>
<th>OUTLET CAPACITY</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>Base equal to or above</td>
<td>Minimum: 0, Maximum: 0</td>
</tr>
<tr>
<td>II</td>
<td>Dead Storage less than Spillway Crest</td>
<td>Controlled releases: Maximum Outlet</td>
</tr>
<tr>
<td>III</td>
<td>Spillway Crest less than Dam Crest</td>
<td>Minimum Flood: Maximum Flood</td>
</tr>
<tr>
<td>IV</td>
<td>Dam Crest no limit</td>
<td>All Water Passed: No Restriction</td>
</tr>
</tbody>
</table>
made for a minimum required released based on flows through uncontrolled spillway works. These are based on the MINIMUM FLOOD CAPACITY, which may be zero. The maximum allowable release, based on the MAXIMUM FLOOD CAPACITY, includes controlled and uncontrolled spills along with all other outlet works operating at full capacity.

Zone IV - When water levels are above the dam crest, it is assumed that the dam is overtopped and the entire volume is routed downstream. In this case the minimum capacity is assumed sufficient to accommodate the entire volume. There is no maximum limit.

It should be noted that specification of a single number for the minimum and maximum capacities in a zone ignores head effects. This is only a first approximation to the true situation. Consequently, the value used should be realistic in terms of the needs of the study.

All capacities are specified with the units 1,000 acre-feet/day. The required minimum or allowable maximum release for a month is then based on the daily capacity and the number of days in the month. Leap years are ignored and a perpetual calendar assumed.

Reservoir inputs - evaporation data

Evaporation rates are time dependent requiring a specific value for each month. A monthly distribution may be used in conjunction with an annual rate to yield the monthly values or they may be read directly. Rates are in acre-feet/acre/time frame (month or annual). Values may be gross or net (difference between evaporation and precipitation).
Reservoir inputs - single reservoir operation - demand array

Details of the reservoir operating procedures are contained in the Computation section. Briefly, the program works from the upstream to downstream boundaries. When a reservoir is encountered, the demand array, IDMD(I), is used to determine how the reservoir is to be operated. The options are:

IDMD(I)=0 - A specified release will not be made, but releases are allowed to satisfy downstream demands or boundary target flows as required.

IDMD(I)=1 - A specified release is to be made. Specified releases are time dependent. The amount may be given as an annual volume which is distributed using the specified monthly distribution or as monthly values. Further releases to meet demands are not allowed, although uncontrolled spills or overtopping the dam may result in additional releases of water.

IDMD(I)=2 - A specified release is made in a manner identical to that for IDMD(I)=1, but in addition further releases are permitted to meet downstream demands.

Demands occur when the upstream flow in a river reach is less than the net depletion. The difference represents a demand for water that has not been met. Similarly, if depletions in a reservoir exceed the total water in storage or target flows at the downstream boundary are not met, a demand exists.

The demand array is time dependent requiring monthly values. A single annual cycle can be used, or values can be changed for each year. It is also noted that the demand array pertains to releases of water from storage and not to flows. Thus, flows may be routed through a reservoir even though IDMD(I)=1.
Reservoir inputs - segment data

Whenever a demand exists, the program uses segment data to determine which reservoirs are normally available to make releases to meet the demands. Segment data includes a segment number, the number of reservoirs available, and the element numbers of the available reservoirs. It is noted that the term normally available is used because even though the segment data specifies a given reservoir, capacity restrictions or the value specified by the demand array may prevent releases. Finally, the percent of the demand to be released from each reservoir is also included on the segment card and used when the release rule code is set to 1 (see following subsection).

Segment numbers, ISEG, for use with each element are specified on the element data card. They are only meaningful for river reaches, reservoirs, and the downstream boundary. They are ignored for upstream boundaries and junctions. Segments are numbered starting at one going up to the maximum. Figure 1 illustrates the segment concept for a hypothetical river basin. As shown, more than one element may reference the same segment.

Reservoir inputs - multiple reservoir operations

When the segment data and reservoir status results in at least two available reservoirs, the problem of allocating water from storage from each to meet demands arises. This is resolved by using the release rule code, IRRCD. The following options are available:

- IRRCD=1, percentage allocation. The percentages contained on the segment card are used to distribute releases among the reservoirs. If for any reason one of the reservoirs cannot

---

Availability involves both physical (reservoir must be upstream and political or legal (operation policy and water rights must permit drafts to be made on the reservoir).
When the above process is completed, the computed releases are made from each reservoir and routed downstream to the point of demand. If the full release cannot be made from a reservoir or cannot be routed through a downstream reservoir to the demand point because of capacity constraints, the reservoir making the release is removed from the list of available reservoirs.

The portion of the demand that was not met is accumulated and is used as the new demand. The entire procedure is then repeated using the available reservoirs. Should the entire demand remain unsatisfied when there are no longer any available reservoirs, a deficit flow results in the element containing the demand.

A little consideration shows that the above method of computing the RATIO and $\overline{RATIO}$ is applicable when $V > V_{SC}$. Although in this case the ratio will be greater than 1, there is no restriction on the method.

IRRCD=3, control zone operation. This method is similar to that used for balanced operations. However, rather than dealing with a single zone extending from the top of the dead storage pool to the spillway crest, up to five levels may be specified to divide the reservoir into as many as six zones. Releases are adjusted so that water levels are in the same zone in each reservoir, and so that the same portion of the zone is filled in a manner similar to balanced operations.

Refering to Figure (3a), the bottom zone extends from the top of the dead storage pool to the first control level, VOLCT(1). The second zone extends from VOLCT(1) to VOLCT(2), etc. The highest zone uses the highest control zone level as a lower limit but has no upper limit.

Adjacent levels may be identical, thus defining a zone of zero volume as shown in Figure (3a). In addition, the lowest
Zone 3 has zero volume.

(a) EXAMPLE OF RESERVOIR WITH FIVE CONTROL VOLUMES

(b) SIMILAR OPERATION OF RESERVOIRS A & B

(c) RESERVOIR A GENERALLY HIGHER THAN B

Figure 3. Control zone volume examples.
level may coincide with the top of the dead storage pool. When a logical analysis of the input data is requested, levels below the top of the dead storage pool are set equal to that level.

Letting $Z_1$ denote the zone number whose upper limit is denoted by control volume $V_{Z_1}$ and lower limit by $V_{Z_1-1}$, the ratio for the zones intermediate between the lowest and highest is given by:

$$\text{RATIO} = \frac{(V-V_{Z_1-1})}{(V_{Z_1}-V_{Z_1-1})}$$

For the lowest zone (Zone 1) the corresponding equation is:

$$\text{RATIO} = \frac{(V-V_{DS})}{(V_{Z_1}-V_{DS})}$$

For the highest zone (Zone $Z_{IM+1}$), the ratio corresponds to the quantity of water above $V_{Z_{IM}}$ since there is no upper limit:

$$\text{RATIO} = (V-V_{Z_{IM}})(1+BANK)$$

When water is in this zone for several reservoirs and exceeds the demand, individual releases are based on taking the same percentage of water in this zone from each reservoir. For all lower zones, the procedure used for balanced operations is applied successively to each zone.

Since the control zone volumes may be specified in various ways for each reservoir, a great deal of flexibility in reservoir operations exists. For example, in Figure (3b), the two reservoirs would operate in similar ways. However, in Figure (3c), Reservoir A would exhibit small fluctuations in content relative to Reservoir B when contents are high but would experience large variations when contents are low.

It should be noted that the release rule code is specified for a reservoir and is not associated with the segment data. Consequently, if one segment references A and B while another references A, B, and C, it is impossible to use balanced operations for A and B operating
together and control zone volumes for A, B, and C. Thus, all reservoirs operating conjunctively must have the same release rule code. When a logical analysis of inputs is requested, this condition is checked after any zero or blank release codes are set to 2. Otherwise, the program uses the release rule code of the first available reservoir it encounters.

Reservoir inputs - control options

Six reservoir options are available to provide additional flexibility in reservoir operations. They are specified through use of the control option array, ICON(I).

ICON(1), variable constraint option. Through the use of this option, variable constraint volumes may be specified which act as lower limits for releases from storage. These constraint volumes are time dependent, being specified for each month. When the variable constraint volumes are less than the dead storage pool, they have no effect on operations. When they are greater they prevent releases from storage whenever the reservoir level falls below them. In this case, the maximum capacity is not altered and flows may be routed through the reservoir providing sufficient capacity exists. If the variable constraint volume exceeds the spillway crest volume, it is reduced to the crest volume.

Proper use of this option prevents premature seasonal lowering of the reservoir that could result in shortages during subsequent months.

ICON(2), flood zone override. A flood zone volume, marking the lower limit of the flood zone which extends upward from it, is specified. It is considered a single valued constant which is independent of time. When releases are made using the percentage allocation (IRRCD=1) it has no effect. However, when
either balanced operations (IRRCD=2) or control zone operations (IRRCD=3) are used, it acts to override other operations criteria.

When water levels are in the flood zone, the quantity of water in the flood zone (including bank storage) is accumulated for all reservoirs operating conjunctively. Should this quantity exceed the demand, the same percentage of water in the flood zones of each reservoir is released. If the quantity is less, water in the flood zones is released and the unsatisfied demand is met using the normal methods established by the release rule code.

ICON(3), spill option. After all demands are met and the downstream boundary has been reached, the program checks the status of each reservoir for additional spills. Starting with the upstream reservoirs and working downstream, the level in each reservoir is used to determine the size of any required minimum releases. If these are not satisfied, additional releases or spills are required.

If the level is above the dam crest, the entire quantity of water above it must be released. Similarly, when levels are between the spillway and dam crests, the minimum uncontrolled spills must be satisfied. The additional spills are then the total of the unsatisfied minimum uncontrolled spills and the quantity above the dam crest.

If the spill option is not used, the program checks if the excess water option (subsequently discussed) is to be used. If

---

If the program finds water in the zone between spillway and dam crests, it assumes that water was in this zone for the entire month. The required minimum uncontrolled spill volume is then the product of the daily rate and the number of days in the month. If the release volume is less, then the difference is the additional uncontrolled release to be made. Now it may occur that only a portion of this amount would put the reservoir below the spillway crest. The program checks for this condition and only releases that portion. Obviously, the state of the reservoir during this transition depends on details during the month. The basic monthly time frame is not adequate and the program approach constitutes an approximation of the true situation.
it is, the quantity of spills is added to any other excess waters which are then released according to excess water rules. When neither the spill or excess water option is used, the total quantity of water is released and routed to the next downstream reservoir or to the downstream boundary if there is none.

When the spill option is used, spills are routed to downstream elements containing variable exports. As many as five variable exports may be specified. Allocation of water between them is based on the size of the spills using a percentage concept. As many as five separate rules may be used. Details are contained in the subsection, "Reservoir Excess Water and Spill Options - Use of Variable Exports."

ICON(4), excess water option. This option is similar to the spill option but does not depend on the availability of spills for water to allocate to the variable exports. Instead, time dependent monthly volumes are specified, above which water is considered as excess. The number of rules, variable exports, and variable export numbers as well as the rules are given on input. Details, almost identical to those for the spill option, are contained in the subsection, "Reservoir Excess Water and Spill Options - Use of Variable Exports."

ICON(5), flag volumes. This option provides a means of flagging the extreme reservoir conditions:
- when the reservoir exceeds the maximum level specified as the maximum flag volume
- when the reservoir falls below the minimum level specified as the minimum flag volume

At the end of each month's computations, levels in each of the reservoirs using this option are compared to the input values. Required flag messages are then printed on the common output unit. They include the month number, reservoir element number, reservoir identification, surface volume, and the message:
RESERVOIR VOLUME GREATER THAN MAX. FLAG VOL. =
or RESERVOIR VOLUME LESS THAN MINIMUM FLAG VOL. =
followed by the corresponding flag volume.

ICON(6), reservoir balancing option. Through the use of this option, two reservoirs may be brought into balance at the end of a month after normal operations are completed and prior to excess water and spill operations. This option is most appropriate for use with reservoirs using balanced operations (IRRCD=2) but can be used with either the percentage allocation (IRRCD=1) or control zone operations (IRRCD=3). It can also be used for reservoirs that are operated independently during normal operations to meet demands.

The element number of the downstream reservoir which is to be balanced with the reservoir for which this option is selected is given by IDRBAL on input. The program checks to see if this is a reservoir element when a logical analysis of inputs is required. If logical analysis is not used, an error message is generated and execution is terminated when an attempt is made to use the reservoir balancing option.

IDRBAL must be located downstream so that releases can be made and routed to it. If the downstream boundary is reached first, a fatal error results with an appropriate error message being written.

Operations are based on the balanced operation philosophy. The percentage of the normal operating zone's capacity that is full is computed for the upstream and downstream reservoirs. If the upstream reservoir has a higher percentage, releases are made (subject to outlet capacity restraints) from the upper reservoir, routed to the lower reservoir, and stored there, thus bringing the reservoirs into balance. When the percentage of the upper reservoir is equal to or less than the lower, no operations occur.
Reservoir inputs - initial conditions

The initial state of the reservoirs must be specified. Included are the surface storage volume in 1,000 acre-feet plus the total dissolved solids. Salinity units, specified by the salinity code, may be:

- Total mass of salts in the surface reservoir and the banks in 1,000 tons
- concentration in ppm or tons/acre-foot. The program assumes the same concentration for water in surface and bank surface. Consequently, this concentration applies to the total volume of water.

Reservoir inputs - excess water and spill
options - use of variable exports

The computation of the quantity of additional uncontrolled spills and/or excess water was discussed in the subsection, "Reservoir Inputs - Control Options." Allocation of this water to variable exports requires the same type of information for either the spill or excess water option. This includes the:

- number of variable exports to which water will be allocated (maximum number is 5)
- variable export numbers
- number of rules to use in allocating the water (maximum number is 5)

Each rule is referenced by an integer rule number. The distribution of water among the variable exports is specified by percentages on the rule card. Associated with the rule is a constraint volume which represents the upper limit for applying the rule. The first increment of water to be allocated up to the constraint volume for the first rule is done so using percentages for the first rule. The second increment of water, corresponding to the difference between the second and first rules constraint volumes, is allocated using percentages for the second rule. The process carries up to the last rule.

In applying the rules, water is allocated by successive rules until the entire amount is either allocated or the last constraint volume
is exceeded. Thus, the program chains through successive rules, starting with Rule No. 1. To illustrate, consider the following example in which three variable exports are to receive water using four rules:

<table>
<thead>
<tr>
<th>Rule No.</th>
<th>Constraint volume (1,000 acre-feet)</th>
<th>Percent allocation to variable export</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>1</td>
</tr>
<tr>
<td>1</td>
<td>10</td>
<td>20</td>
</tr>
<tr>
<td>2</td>
<td>50</td>
<td>20</td>
</tr>
<tr>
<td>3</td>
<td>90</td>
<td>50</td>
</tr>
<tr>
<td>4</td>
<td>150</td>
<td>0</td>
</tr>
</tbody>
</table>

If the quantity of water to be allocated were 100 (1,000 acre-feet), application of each rule results in the following values for each export:

<table>
<thead>
<tr>
<th>Rule No.</th>
<th>Quantity allocated to each export (1,000 acre-feet)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1</td>
</tr>
<tr>
<td>1</td>
<td>2</td>
</tr>
<tr>
<td>2</td>
<td>8</td>
</tr>
<tr>
<td>3</td>
<td>20</td>
</tr>
<tr>
<td>4</td>
<td>0</td>
</tr>
</tbody>
</table>

Total 30 43 27 100

When a logical analysis of the input data is requested the program checks that:

- there is at least one variable export, but not more than the allowable maximum
- the variable numbers are greater than zero, but not more than the allowable maximum
- the number of rules is not more than the allowable maximum
- the percentages sum to 100 for any given rule (plus or minus the program tolerance of 0.1)
- successive constraints must be equal to or greater than the one for the last rule (must be greater than or equal to zero for the first rule)

Variable exports should be located downstream from the reservoir, or within the reservoir itself. There are no logical checks to verify this prior to the monthly computations. However, in routing
allocations downstream, an error message is generated and execution is terminated if the downstream boundary is encountered before the element containing the variable export is reached.

Associated with each variable export is a constraint volume which acts as a capacity limitation. The total quantity allocated to a given variable export from all reservoirs using both spill and excess water options cannot exceed the constraint volume. Constraint volumes are time dependent and can be specified for each month of the study.

Because of variable constraint volumes, capacity limitations within the given reservoir or at intervening downstream reservoirs, the total quantity of water allocated to a given export may not reach the export's diversion point. When this occurs, values for the other exports remain the same, being established by the rule percentages.

The actual monthly quantities allocated, routed, and diverted by each variable export are automatically printed at the end of the run. Descriptive statistics are included.

River reach and reservoir element inputs

River reach and reservoir element inputs refer to those inputs representing water and/or salt additions and depletions to the river system. They permit an accounting of activities within the system and are only allowed for river reach and reservoir elements. All inputs are time dependent and may be specified using either monthly values or annual ones which are distributed using a monthly percentage distribution.

Four types of inputs are allowed:

Import/exports. Both the quantity of water (1,000 acre-feet/month) and salt are specified. Salinity may be expressed as a concentration (tons/acre-foot or ppm) or as a mass (1,000 tons/month). A positive mass signifies the addition of water or salt to the river system while a negative denotes a depletion or removal. When the salinity
is a concentration, the product of the signs of the flow and concentration determines the sign of the salt mass. (Negative concentrations are permitted.)

This type of input may be used to account for a variety of activities in the basin. For example:

<table>
<thead>
<tr>
<th>Item</th>
<th>Flow</th>
<th>Salt</th>
</tr>
</thead>
<tbody>
<tr>
<td>Import</td>
<td>+</td>
<td>+</td>
</tr>
<tr>
<td>Irrigation</td>
<td>-</td>
<td>+</td>
</tr>
<tr>
<td>Export</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Consumption</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Desalting</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

It is noted that the combination of a positive flow and negative salt mass is not apt to occur. One possible though unlikely situation would be a desalting plant disposing of brine in an isolated deep aquifer, using water from a shallow aquifer for blending purposes.

Exports. Only the quantity of flow (1,000 acre-feet/month) removed or exported from the river basin is specified. For river reaches the concentration of the exported water is that of the upstream inflow to the element containing the export prevailing at the given time. For reservoirs, export concentrations equal the reservoir concentration at the start of the month. Consequently, concentrations depend on initial conditions, boundary inputs, river reach and reservoir inputs as well as operating policies.

The word export already denotes the removal of water from the river and all exports should be positive quantities. However, there are no program checks. Negative exports will result in the addition of both water and salt to the system.

Exports may be used to account for diversions to locations outside the basin, or to temporary storage facilities such as aquifers or offstream surface storage.

Variable exports. These are identical to the exports described above except their magnitude is not firmly set. Instead, maximum or
limiting values are specified and the actual size depends on the availability of water in the reservoirs. Either the excess water or spill option described in the preceding subsection, or both, may be used to determine availability.

Irrigations. An irrigated acreage (1,000 acres), quantity of water added or subtracted from the system, plus a salt pickup rate (tons/acre/month) are specified to account for irrigation projects.

A positive flow denotes an increase of water in the river corresponding to the case where diversions are less than return flows while a negative flow signifies a decrease in streamflow. If the pattern of diversions and returns are neglected, the flows may be set equal to the consumptive use to account for water depletions. It should be noted that the salinity of diversions and returns are not explicitly considered. However, a net water depletion does produce an increase in concentration in the river.

The salt pickup rate is multiplied by the acreage to yield the addition or removal of salt due to chemical processes in the soil system. A positive rate results in the addition of salt to the river due to dissolution and leaching while a negative rate results in a depletion due to precipitation and storage within the soil system. Pickup rates and the resulting quantities of salt are independent of the magnitude and sign of the flows.

Inputs of each type are numbered consecutively using integers starting with one and proceeding up to the maximum. The numbers are used by the program for subscripting purposes. An input may be given a number in the consecutive string even if it is not used or its value is zero. The element in which an input is located is specified on the identification card for that input.

Inputs located in reservoir elements withdrawing water are assumed to do so directly. Consequently, the outlet works capacity does not apply and there is no restriction on the size of the depletion. Water can be withdrawn from the reservoir until it is dry, after which the remaining demand must be satisfied using reservoirs specified by the segment data. If withdrawals are to be met by releases from several reservoirs, a dummy river reach should be added immediately downstream from the reservoir and the input located in it.
When the logical analysis of inputs is requested, the program checks all inputs for errors. However, even if the analysis is not requested, several items are still checked when the program establishes internal arrays. Consequently, logical error messages may be produced even though this option was not specified.

**Boundaries**

Boundaries include those upstream at which both flow and salinity are specified as well as the downstream boundary at which desired or target flows are specified. Values at the upstream boundaries normally represent the main supply or driving force of the river system. If the computed downstream boundary flow is less than the target value, the difference is treated as a demand which may be met by releasing water from those reservoirs indicated by the segment data. Upstream flows and salinities and downstream target flows should always be positive quantities, although there are no program checks to assure this.

**Upstream boundary.** Values of the volume and salinity can be input as monthly values or as annual ones distributed using monthly percentages.

**Downstream boundary.** Inputs include the target volumes specified as monthly or annual values.

**Computations**

**Computation procedure for quantity**

The basic program approach is outlined so that the user may better appreciate the limitations and assumptions involved in using the program. Standard internal units for flow are 1,000 acre-feet/month and for reservoir volumes 1,000 acre-feet, with the corresponding salinity units of 1,000 tons/month and 1,000 tons (i.e., mass only). Only standard units are allowed for flow and volume inputs. Output is
normally in 1,000 acre-feet but the user has the option of obtaining 100's, 10's, or 1's of acre-feet depending on the field width. Salinity inputs and outputs may also be expressed as concentrations: either ppm or tons/acre-foot. Nonstandard input units are converted to standard ones prior to computation. Standard ones are converted to requested nonstandard units prior to output.

It is assumed that the river flows are independent of salinity. The program first computes flows and reservoir releases, and then computes the corresponding salinities.

Computations commence with the first upstream boundary referenced on input (internal subscript 1) and proceed downstream until a junction is encountered. The computation procedure is now restarted at the second upstream boundary, proceeding as before until a junction is encountered. Since this may not be the same junction initially encountered, the program checks to see if all upstream branches into the junction have been entered. If they have, computations proceed downstream until either another junction or the downstream boundary is reached. If all branches were not entered, the next upstream boundary is used as the starting point for continued computations.

When a river reach element is encountered, the net effect of the import/export, irrigation, and export volumes is computed. The sequence of inputs within the reach is unimportant with only their aggregate value considered. This limits the obtainable resolution of the results. If more detail is necessary, the reach must be subdivided into smaller reaches containing the appropriate inputs.

An internal sign convention in which a depletion of flow is negative and an accretion is positive is attached to the aggregate value. This is algebraically added to the upstream inflow to the reach resulting in the computed downstream outflow. If the outflow is negative, the total streamflow has been depleted. The negative outflow now represents the additional demand placed upon the system.

If a demand exists, segment data are used to determine which reservoirs, if any, are available to meet the demand. The purpose
of a reservoir is to regulate natural inflow hydrology and convert it to a desired outflow hydrology to meet the demands. To accomplish this in a general program requires procedures that facilitate duplication of the operational policies and rules, at least in a gross manner. Certain details are inevitably lost. If their loss is unacceptable, special programs must be written for the specific system. However, future uncertainties in water supply, demands, operational policies, and project development may be of greater significance.

When a reservoir element is encountered the first time during the monthly computation cycle, the program first computes evaporation for the current month. The volume at the start of the month is used with the area-capacity relations to compute the corresponding surface area. Evaporation rates for the month are then used to compute the total evaporation volume for the month. In general, a more accurate approach requires use of an average area during the month. Since the ending area for the month is unknown until computations are completed, an estimated value is required. Computations would be carried out, the average area found and compared with the estimated, and a new estimate of the average area made based on these results. The entire process would then be repeated until the estimated and computed areas are within a desired degree of accuracy. However, this iterative approach is assumed an unnecessary refinement in terms of other model assumptions and inputs.

When the reservoir is initially encountered, any inflow is assumed to raise the level of the reservoir with a portion of the water entering the banks and a portion increasing surface storage. Internally, all volumes are in terms of the surface storage. Indeed, flag levels, constraints, control zones, area-capacity curves, etc., are in terms of surface storage.

In a manner similar to that for the river reach, the net effect of the inputs within the reservoir are computed. Additions are made directly to the reservoir and depletions removed directly from it.
The outlet works capacity does not apply and water may be withdrawn until the reservoir is dry. If additional water is required, segment data are used to determine which reservoirs, if any, can be operated to meet the demand.

The operation of an individual reservoir is based on the demand array. Three options are available:

1. Make a specified release only
2. Make a specified release and meet any downstream demands
3. Do not make a specified release but meet downstream demands

Before proceeding downstream, the reservoirs demand array is used to determine if a specified release is requested. If it is, the release is made subject to capacity restraints. The computational procedure is identical to that used whenever releases are made from storage to meet downstream demands. No water may be released if the level is within the dead storage pool since the corresponding capacity is zero. When levels are above the dam crest, there is no restriction on the capacity. Between the spillway and dam crests, the maximum flood capacity controls. Below the spillway crest and above the dead storage pool the normal maximum capacity is used. When the variable constraint option is used the variable constraint level supersedes the dead storage level as the level below which the capacity is zero as far as releases from storage are concerned. Should the variable constraint level exceed that of the spillway crest, the program essentially reduces it to the spillway crest.

The program determines in which of the previous zones the water level is located, computes the amount of water available as bank and surface storage in that zone, adds it to the releases made thus far, and then compares the total with the corresponding capacity. If the capacity exceeds or equals this amount, available water in the next lower zone is computed and handled in the same way. If the
capacity would be exceeded should all available water be released, the available water is reduced until the maximum capacity is just reached.

The available water is now compared to the demand (or required release). If it exceeds the demand, releases are made to just satisfy the demand. If it is less than the demand, all of the available water is released and a portion of the original demand remains unsatisfied.

After the specified releases are made, computations proceed downstream from the reservoir. If an unsatisfied demand is encountered, segment data are used to determine which upstream reservoirs can be operated to satisfy the demand. However, some of these reservoirs may not be available because the demand array may allow only specified releases. Since the demand array is time dependent, the actual number of reservoirs available to meet demands may vary from month-to-month.

When more than one reservoir is available, their conjunctive operation is based on the release rule code which permits three methods of operation:

1. Release water using fixed percentages specified by the segment data
2. Base releases on balanced operations
3. Base releases on control zone operations

When the fixed percentages are used and some of the reservoirs are unavailable, only that portion of the demand satisfied by the remaining reservoirs is supplied. Similarly, if any of the remaining reservoirs cannot release their required share due to capacity restraints or if intervening downstream reservoirs restrict routing the releases to the element containing the demand, shortages will result. It is emphasized that releases from the available reservoirs are not increased to reduce this shortage.

When either balanced or control zone operations are employed the program first checks if any of the available reservoirs are using
the flood zone override option. In using this option, the bottom of the flood zone is specified. Should water levels enter this zone, the reservoir is given preference over all other reservoirs operating conjunctively, regardless of other criteria. When more than one reservoir in a segment is using this option the total volume of water in all the flood zones is computed. If this amount exceeds the demand, the same percentage of the water in each flood zone is released. For example, if the demand is 100 and the flood zone volume in Reservoir A is 150 and in B is 50, the percent to release is $100/(150+50)$ or 50 percent. Then the release from A is $150 \times 0.5$ or 75 and from B $50 \times 0.5$ or 25.

If there are no flood zone releases or if they do not satisfy the demand, the remaining unsatisfied demand is met using normal operating procedures. The technique is to calculate the quantity of water to be released through a series of steps. Actual releases and routing are only performed when releases equal the demand (if possible).

For balanced operations, water is released from the reservoir having the highest percentage of water in the active or normal operating zone down to the next highest reservoir if necessary. Should additional water be required, water is released from both reservoirs down to the next highest reservoir (or to the dead storage or variable constraint levels). For control zone operations, the procedure is similar except water is released from the reservoir with water in the highest zone at the highest percentage.

After the required releases are determined, each reservoir is operated with the releases made and routed to the element containing the demand. During these operations, two situations may arise which prevent meeting the full demand:

1. The required release cannot be made due to capacity constraints
2. A portion of the actual water released did not reach the demand element because an intervening downstream reservoir restricted routing due to capacity constraints
In either case, the reservoir making the release is removed from the list of available reservoirs. That portion of the demand not met becomes the new demand which is to be satisfied by operating the remaining available reservoirs. This procedure is continually repeated until the demand is satisfied, or there are no reservoirs remaining.

When flows are routed through a reservoir, only the normal zones and capacities apply. Thus, even if water is below the variable constraint level, water may be routed through the reservoir if sufficient capacity exists. Similarly, the demand array only pertains to making releases from storage and not to routing. It should also be noted that although this discussion talks of routing water through a reservoir, in reality water is released from storage at the originating reservoir to exactly replenish water released from the lower reservoir to meet downstream demands. The routing concept is a computational expedient.

When the downstream boundary is reached, the computed flow is compared to the target value specified on input. If the computed value is less than the target value, the difference is treated as a demand. Segment data are then used in the usual way.

During the normal computation process the demand in a river reach or reservoir may not be met, or the downstream boundary target flow may not be satisfied. The unsatisfied demand or target flows are retained as a variable called the deficit flow on output. Subsequent spills may occur, which if used properly, could reduce or satisfy the deficit flow. However, when reservoirs are operated in a logical manner, this is unlikely. It should also be noted that there may be outflow from a reach even if there is a deficit flow since releases from a reservoir above the reach to elements below it may be specified.

After computations for the downstream boundary are completed, each reservoir is checked to see if the reservoir balancing option has
been selected. The procedure commences with the upstream boundary and is applied progressively in the downstream direction. If a reservoir is using this option, the balanced operation philosophy is applied to determine the portion of the normal operating zone that is full in both the upstream reservoir using this option and the specified downstream reservoir with which it is to be balanced. If the upstream reservoir storage is greater, the required release to balance the reservoirs is computed. An attempt is then made to make the required release and to route and store it in the downstream reservoir. Balanced conditions may not be achieved because the upstream or intervening reservoirs restrict flow due to capacity restraints.

The status of each reservoir must now be checked to determine if any uncontrolled spills are required to satisfy minimum capacity constraints. In addition, the status of reservoirs requesting the excess water option must be examined for excess water which may be allocated to variable exports. If the spill option is used, spills may also be allocated to variable exports. The computational order is illustrated by the flow chart of Figure 4.

Computations commence with an upstream reservoir and proceed downstream through successive reservoirs. Any water above the dam crest must spill. In addition, it is assumed that if water is in the spillway-dam crest zone at the end of the month, the required minimum daily spill must occur during each day of the month. The total volume for the month is then compared to the total releases made during normal operations (exclusive of water above the dam crest). If it exceeds the releases, the difference is the additional release necessary to satisfy the minimum flow constraint. This additional release is then compared to the volume of water which would lower the reservoir to the spillway crest level. If this volume is less than the additional release, the

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6 This is obviously an approximation to reality. The actual number of days during which uncontrolled spills occur is a detail lost because the monthly time frame is too large.
Figure 4. Computational order for spills and excess water.
release is set equal to the volume. Under no circumstances will uncontrolled spills lower the reservoir below the spillway crest.

The total uncontrolled spills are the sum of the additional release to satisfy minimum spills and the water above the dam crest. The same procedure is used when there is no water above the dam crest. If the spill option is used, the spills are allocated to variable exports, released from the reservoirs, and routed to the element containing the export. Since only a portion of the total spill may be allocated, the remainder is computed and saved as the spills remaining to be released. If the spill option is not used, the remaining spills equal the originally computed total.

The program now determines if the excess water option is being used. If it is, the amount of excess water exclusive of any spills is computed and added to any remaining spills. The total is then allocated to the variable exports, released, and routed. Finally, the excess water allocated and released is compared to the remaining spills to determine if all the required spills were satisfied. If they were not, sufficient water is released to satisfy the minimum flow constraint. When the excess water option is not used the additional release equals the remaining spills. Spills that are not allocated are routed to and stored in the first downstream reservoir.

It should be noted that certain details are lost and replaced by approximations established by program assumptions whenever levels fluctuate between zones during a given month. Thus, transitions between the normal operating zone and the zone above the spillway crest or below the top of the dead storage pool produce approximate results. Refinement would require shorter time increments and capacity constraints continuous with reservoir levels.

**Computation procedure for quality**

With river flows and reservoir releases determined, the corresponding salinities can be computed. The general approach is nearly
identical to that used in computing flows. Computations commence with the first upstream boundary referenced on input and proceed downstream until a junction is encountered. Computations are then restarted at the next upstream boundary, proceeding as before until a junction is encountered. If each upstream branch into the junction has been entered, computations proceed downstream. Otherwise, another upstream boundary is used in continuing the process.

When a river reach is encountered, the net salinity effect of the import-exports, irrigation, and exports is computed. Only their aggregate effect and not their sequence within the reach is important. For the import/exports either the concentration or mass of salts is specified. Irrigation effects are computed using the per acre pickup rate per month and the specified number of acres. Exports are diverted at the computed upstream inflow concentration to the reach.

An internal sign convention is employed in which a negative aggregate value denotes a removal of salt from the river while a positive quantity denotes salt has been added. The aggregate effect is added to the mass of salts entering the upstream end of the reach to determine the salt outflow. Since the mass of salts specified on input may exceed the mass in the river, a negative sum may result. Clearly, this attempt to remove more salt than exists is an impossible situation. Consequently, the salt outflow is set to zero and the negative value is retained as a variable called delta salt on output.

A second situation that can produce a value for delta salt arises when the water outflow from the reach is zero. Since the salt outflow may not be zero, the outflow concentration is undefined and meaningless. For program purposes, it is assumed that without water there is no vehicle to transport the salt. The actual salt outflow is set to zero and the delta salt variable to the computed value. By convention, the program assumes these salts are lost from the river system rather than remaining in the river bed to be picked up by subsequent river flows. The corresponding outflow concentration is set to zero so that exports in the next reach divert no salts.
In either of the above cases, the delta salt value represents the computed salt outflow which was reset to zero because of an impossible physical situation. The positive sign indicates computed salts would have been carried downstream if flows existed while the negative sign denotes an attempt to remove an excess of salt from the river.

Inputs specify either the concentration or mass of salts for upstream boundaries. This mass is routed downstream with the output from one element acting as input to the next. At junctions, the salinity outflow is equal to the sum of salt inputs. Only river reaches and reservoirs can contain element inputs to account for imports, exports, and irrigation projects. Treatment of river reaches has already been discussed and attention is now turned to reservoirs.

A number of situations may arise in operating a reservoir. As described below, each situation requires a different method of computing salinities:

a. Normal procedure. When reservoir releases for the month are less than 25 percent of the total water in storage at the beginning of the month (bank plus surface storage) and the reservoir does not go dry during the month, normal procedures are followed. The concentration at the start of the month is used to compute salt removed by releases and exports from the reservoir during the month. At the end of the month, the mass of salts stored in the reservoir is equal to the algebraic sum of the starting mass, export salts, downstream release salts, and the net effect of import/export and irrigation inputs. If the mass is negative, it indicates an attempt to remove more salt than exists. The salt content is set to zero and the negative value retained as a delta salt. A linear mix of all salts is assumed, resulting in a uniform concentration of salts throughout the surface and bank storage volumes.

b. Excessive releases. When reservoir releases for the month are equal to or greater than 25 percent of the total water in storage at the beginning of the month and the reservoir does not go dry during the
month, use of the initial concentration may be inappropriate. In partic-
ular, as the releases approach 100 percent of the initial content, out-
flow concentrations would be expected to show some effect of the in-
flows. To include the weight of present inflows, concentrations for
exports and releases are based on a linear mix of the starting salt
mass and volume with the inflow of salt and water. At the end of the
month the reservoir salinity is updated and checked as it is under nor-
mal procedures.

c. Special cases of empty reservoirs. Three cases can arise
depending on whether the reservoir is empty at the start or end of the
month:

(1) Reservoir empty at start and end of month. Concentrations
for computing salt removed by exports and releases are equal
to the inflow concentration for the month. The mass in storage
at the end of the month is then updated according to normal
procedures. In this case, it must be exactly zero since there
is no water remaining as storage. If a positive or negative value
results, the mass is set to zero and the nonzero value saved as
the delta salt for the reservoir.

(2) Reservoir empty at end but not at start of month. Concen-
trations for exports and releases are computed using the same
linear mix technique employed for the case of excessive releases.
The salt mass in storage at the end of the month is computed using
methods for the normal procedure. Delta salts are defined in
the same manner as for Case c-1 above.

(3) Reservoir empty at start but not at end of month. In this
case, there are no salts in storage at the start of the month. Con-
centrations used for exports and releases are those of the entering
inflows. At the end of the month, salts are updated and checked
exactly as they are using normal procedures.

It should be noted that whenever a reservoir goes dry, any positive
delta salts are assumed lost from the system. As for deficit salts in
a river reach, they do not remain within the reservoir until it again fills or flows past through it. Evaporating water is assumed to carry no salts.

References


A major effort to ascertain the economic impact of changes in salinity levels of the Colorado River is contained in the Colorado River Regional Salinity Research Project cosponsored by the Office of Water Research and Technology (Project B-107-Utah). Leadership for the project is by Dr. Jay C. Andersen, Utah State University, and Dr. Alan P. Kleinman, Bureau of Reclamation. Cooperating institutions include the University of California, University of Arizona, Colorado State University, and the University of Colorado. In addition, considerable time contributions have been made by personnel of The Metropolitan Water District of Southern California (MWD).

Research efforts are focusing on three primary water uses: agricultural, municipal, and industrial. Management alternatives available to upper basin water users which might mitigate the salinity burden of the river also are being analyzed.

Agricultural yield decrements and alternative management practices which might occur as salinity levels increase are being evaluated by the University of Arizona and the University of California. These physical data are then used as inputs to a linear programing profit maximization model, wherein the optimal farmer response to salinity change is delineated. From this optimization for salinity levels from 900 to 1,400 milligrams per liter, a damage function is defined for each impact area. This linear programing work is being carried out by the Bureau of Reclamation. The agricultural areas now being modeled are all in the lower

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*Head, Economics Section, Engineering and Research Center, USBR, Denver, Colorado.
basin: San Diego coastal area, Coachella Valley, Imperial Valley, Yuma area, Palo Verde Irrigation District, Colorado River Indian Reservation, and the Central Arizona Project (CAP) service area.

The agricultural damage estimates will then be used in conjunction with the Colorado River Simulation Model. This will provide the capability to observe the major economic impact of salinity changes, as the simulation model is run under varying assumptions and conditions. Such economic evaluation will provide the basis both as a measure to evaluate salinity mitigation proposals and for negative external impacts of future water resource development projects as required by the Office of Management and Budget.

Continuing work is expected to encompass all agricultural and M&I users in both the upper and lower basins as well as the most promising salinity mitigation measures, in order to provide guidance as to the optimal development pattern for handling salinity in the basin.

Research Objectives

The objective of this portion of the research is to make an economic evaluation of the impact of increasing salinity in the Colorado River on agriculture in the lower basin. Specifically, it is desired to project changes in cropping patterns, physical output for each crop, changes in farm management, and dollar impacts in terms of net profit.

The Linear Programming Model

The linear programing routine (APEX-I), utilized for analysis, is a program supplied by Control Data Corporation and run on the CDC Cyber 74/28 system of the Bureau of Reclamation in Denver. This LP package has sufficient capacity and flexibility to allow modeling of all sizes of irrigation districts. Though models of necessity are tailor-made for each area investigated, the work accomplished on the Imperial Irrigation District will be used as the example.
The model is designed to maximize net returns to all farmers in a
district above variable production costs and new capital investments
subject to resource and production constraints. Detailed enterprise
budgets for 13 crops representative of conditions in the Imperial Irriga-
ted area were used to develop the linear programing model.

The crops used were alfalfa hay, cotton, sugar beets, sorghum,
wheat, barley, lettuce, tomatoes, asparagus, onions, watermelon,
carrots, and cantaloupe which account for about 90 percent of the acreage
of the Valley. Each of these crop activities was defined on four soil types;
very poorly drained, poorly drained, moderately well drained, and well
drained. The combination of each crop with each soil type was then
defined for six irrigation activities which include variations in frequency
of water application as well as partial and full sprinkler systems. Avail-
able to each of the above combinations was a number of management
activities. These activities were options open to the manager which he
might employ, at a cost, in the face of rising salinity to mitigate the
detrimental influence upon net returns. These activities include ditch
lining, land leveling, deep plowing, tiling, special bedding practices,
and leaching irrigations. Various combinations of crops were defined to
allow more than one crop on each acre per year.

The program was then run for six salinity levels from 900 to 1,400
mg/l with the difference in the value of the objective function indicative of
the damage associated with the salinity change.

Model Constraints

The number of acres available for crop production was limited to
the available land including double cropping and excluding the historical
pattern to fallow land. The quantity of water available for crop use had
an upper limit associated with the water rights. Various categories of
labor were constrained or simply accounted for to provide labor use infor-
mation. Fertilizer rows were utilized as well as rows for new capital
investment. Existing management improvements were inserted as data
in the model such as land presently tilled. In order to restrict the production of high valued specialty crops, constraints were applied to total production of each commodity which serves as a proxy for the magnitude of market demand.

Results

The decrease in net profit available to farmers as a result of salinity impacts is estimated through repeated running of the linear programming model for Imperial Valley. The estimated impacts are given below:

<table>
<thead>
<tr>
<th>Irrigation District Farm Profit (Dollars)</th>
</tr>
</thead>
<tbody>
<tr>
<td>mg/l</td>
</tr>
<tr>
<td>------</td>
</tr>
<tr>
<td>900</td>
</tr>
<tr>
<td>1,000</td>
</tr>
<tr>
<td>1,100</td>
</tr>
<tr>
<td>1,200</td>
</tr>
<tr>
<td>1,300</td>
</tr>
<tr>
<td>1,400</td>
</tr>
</tbody>
</table>

Average decrease per mg/l = $28,167.35

Table 1 shows a comparison of the 1974 season prices actually received by Imperial Valley farmers with the long-term trend prices used in the model. Because of the dynamic price movements experienced in 1974, some of the prices used are significantly different than actually realized.

In order to indicate the predictive ability of the model, a comparison of selected factors is given in Table 2. The approximation of the existing situation by using 900 mg/l shows a very good correlation between historical trend and model results.

Table 3 shows on a crop by crop basis comparison between actual data and model results for yields, acres, and production.
Table 1. Price Comparisons ($/Ton).

<table>
<thead>
<tr>
<th>Crop</th>
<th>1974 Season</th>
<th>LP Model</th>
</tr>
</thead>
<tbody>
<tr>
<td>Asparagus</td>
<td>888</td>
<td>917</td>
</tr>
<tr>
<td>Cantaloupe</td>
<td>222</td>
<td>196</td>
</tr>
<tr>
<td>Carrots</td>
<td>94</td>
<td>171</td>
</tr>
<tr>
<td>Alfalfa</td>
<td>62</td>
<td>56</td>
</tr>
<tr>
<td>Tomatoes</td>
<td>192</td>
<td>387</td>
</tr>
<tr>
<td>Watermelon</td>
<td>108</td>
<td>88</td>
</tr>
<tr>
<td>Barley</td>
<td>120</td>
<td>118</td>
</tr>
<tr>
<td>Wheat</td>
<td>130</td>
<td>137</td>
</tr>
<tr>
<td>Sugar Beets</td>
<td>51</td>
<td>24</td>
</tr>
<tr>
<td>Lettuce</td>
<td>88</td>
<td>137</td>
</tr>
<tr>
<td>Onions</td>
<td>106</td>
<td>166</td>
</tr>
<tr>
<td>Sorghum</td>
<td>128</td>
<td>112</td>
</tr>
<tr>
<td>Cotton (lb)</td>
<td>.50</td>
<td>.49</td>
</tr>
</tbody>
</table>

Table 2. Selected factor comparison historic and LP Model 900 mg/l.

<table>
<thead>
<tr>
<th>Factor</th>
<th>Historic</th>
<th>LP Model</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water use - acre-feet</td>
<td>2,838,558</td>
<td>2,692,167</td>
</tr>
<tr>
<td>Total acres in crops</td>
<td>384,530</td>
<td>384,530</td>
</tr>
<tr>
<td>Acres double - cropped</td>
<td>122,698</td>
<td>122,698</td>
</tr>
<tr>
<td>Gross output in dollars</td>
<td>284,242,000</td>
<td>296,822,804</td>
</tr>
<tr>
<td>Acres tiled</td>
<td>288,325</td>
<td>288,325</td>
</tr>
<tr>
<td>Sprinkler to establish stand</td>
<td>56,600</td>
<td>69,973</td>
</tr>
<tr>
<td>Full time sprinkler system</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Fallow land</td>
<td>28,675</td>
<td>28,675</td>
</tr>
</tbody>
</table>
Table 3. Comparison of actual conditions for Imperial Valley in 1974 with Solution at 900 mg/l.

<table>
<thead>
<tr>
<th></th>
<th></th>
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<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Asparagus</td>
<td>1.53 tons</td>
<td>±.16</td>
<td>4,533</td>
<td>6,568</td>
<td>2,963</td>
<td>4,170</td>
<td>1.63</td>
<td>7,500</td>
</tr>
<tr>
<td>Alfalfa</td>
<td>7.45 tons</td>
<td>±.33</td>
<td>1,072,288</td>
<td>1,203,934</td>
<td>150,726</td>
<td>176,051</td>
<td>9.00</td>
<td>1,089,000</td>
</tr>
<tr>
<td>Watermelon</td>
<td>9.80 tons</td>
<td>±.42</td>
<td>29,846</td>
<td>25,777</td>
<td>3,046</td>
<td>3,192</td>
<td>7.25</td>
<td>29,000</td>
</tr>
<tr>
<td>Tomato</td>
<td>7.68 tons</td>
<td>±.85</td>
<td>19,018</td>
<td>16,951</td>
<td>2,529</td>
<td>2,401</td>
<td>12.93</td>
<td>38,800</td>
</tr>
<tr>
<td>Onion</td>
<td>13.70 tons</td>
<td>±.41</td>
<td>81,752</td>
<td>64,846</td>
<td>5,967</td>
<td>4,231</td>
<td>12.00</td>
<td>36,000</td>
</tr>
<tr>
<td>Carrot</td>
<td>14.00 tons</td>
<td>±.42</td>
<td>67,254</td>
<td>56,462</td>
<td>4,804</td>
<td>4,657</td>
<td>18.86</td>
<td>111,300</td>
</tr>
<tr>
<td>Cantaloupe</td>
<td>5.88 tons</td>
<td>±.59</td>
<td>77,504</td>
<td>61,866</td>
<td>14,028</td>
<td>10,567</td>
<td>7.53</td>
<td>62,500</td>
</tr>
<tr>
<td>Sugar Beets</td>
<td>22.00 tons</td>
<td>±.36</td>
<td>1,459,281</td>
<td>1,615,143</td>
<td>66,331</td>
<td>69,193</td>
<td>26.80</td>
<td>1,742,000</td>
</tr>
<tr>
<td>Sorghum</td>
<td>2.25 tons</td>
<td>±.27</td>
<td>91,101</td>
<td>100,934</td>
<td>67,736</td>
<td>50,417</td>
<td>2.30</td>
<td>74,000</td>
</tr>
<tr>
<td>Barley</td>
<td>1.90 tons</td>
<td>±.21</td>
<td>52,606</td>
<td>95,500</td>
<td>27,687</td>
<td>51,766</td>
<td>2.14</td>
<td>12,000</td>
</tr>
<tr>
<td>Wheat</td>
<td>2.14 tons</td>
<td>±.29</td>
<td>131,182</td>
<td>125,191</td>
<td>61,300</td>
<td>51,477</td>
<td>2.53</td>
<td>263,000</td>
</tr>
<tr>
<td>Cotton</td>
<td>2.43 tons</td>
<td>±.80</td>
<td>100,182</td>
<td>74,722</td>
<td>41,199</td>
<td>36,625</td>
<td>2.38</td>
<td>215,800</td>
</tr>
<tr>
<td>Lettuce</td>
<td>10.83 tons</td>
<td>±.01</td>
<td>6,411,159</td>
<td>515,815</td>
<td>59,202</td>
<td>42,771</td>
<td>11.65</td>
<td>571,000</td>
</tr>
</tbody>
</table>
The results of all model runs are then used to define a damage function to be used in conjunction with the Colorado River Simulation Model at the node for Imperial Dam. Alternative functional forms are shown in Table 4 and the data are shown graphically in Figure 1. As can be seen, the quadratic form provides a close approximation of the data generated by the model runs but deviates rather widely outside of the range of the data.

Similar functions have been generated for major areas of agriculture and M&I water use. Upon completion, these models will provide, at a very low cost, information relative to the economic impact of any number of alternative operating, management, and structural policies which we may wish to evaluate in order to provide guidance for the "best" solutions to the salinity problems of the Colorado River.

Table 4. Agricultural damage function estimates Imperial Valley (1,000's of Dollars).

<table>
<thead>
<tr>
<th>mg/l</th>
<th>Model Estimate</th>
<th>Linear Fit 1/</th>
<th>Quadratic Fit 2/</th>
</tr>
</thead>
<tbody>
<tr>
<td>1,000</td>
<td>1,906</td>
<td>267</td>
<td>2,145</td>
</tr>
<tr>
<td>1,100</td>
<td>2,702</td>
<td>3,186</td>
<td>2,246</td>
</tr>
<tr>
<td>1,200</td>
<td>4,294</td>
<td>6,105</td>
<td>4,226</td>
</tr>
<tr>
<td>1,300</td>
<td>7,539</td>
<td>9,024</td>
<td>8,085</td>
</tr>
<tr>
<td>1,400</td>
<td>14,084</td>
<td>11,943</td>
<td>13,822</td>
</tr>
</tbody>
</table>

1/ Estimated by the equation: \( D = a + bx \) where \( a = -28,925,121 \) and \( b = 29,192; \; r^2 = .87 \)

2/ Estimated by the equation: \( D = a + bx + cx^2 \) where \( a = 104,465,155, \; b = -196,257, \) and \( c = 93.94; \; r^2 = .96 \)
Figure 1. Imperial Valley Damage Function
AUTOMATIC GENERATION CONTROL

by

Larry R. Ruggles*

This paper describes in general the generation and marketing of power from the United States Bureau of Reclamation (USBR), Colorado River Storage Project (CRSP), with emphasis on the present Automatic Generation Control (AGC). The AGC program is in a General Electric 412B Computer located at the Power Operations Office, which is in Montrose, Colorado.

The Colorado River Storage Project has hydro-electric generator units on the Colorado, Gunnison, and Green Rivers and the AGC extends into five states. Each unit is controllable by the AGC which transmits raise or lower pulses to increase or decrease turbine gate openings which regulate the level of power being generated.

The Power System Dispatcher at Montrose selects the unit and enters into the computer the mode of generation as "AUTOMATIC," "MANUAL," or "OFF." In the Automatic Mode the unit participates in control area regulation. In Manual Mode the unit is base loaded at some base point determined by the dispatcher. The unit level may be changed by entering a new base point into the computer. In the Off Mode no control pulses are transmitted to the unit.

The CRSP Control Area extends into five states with generating capacity far in excess of the load within the control area; hence, most of the power is exported to customers outside the control area. Power scheduled to these customers is usually finalized prior to each hour and then the computer, through the AGC program, maintains proper generation to fulfill the schedules. The total generation equals the sum of the

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*United States Bureau of Reclamation.
internal load and losses within the control area, plus the sum of the power scheduled to all customers outside the control area. The computer controls to a deviation known as Area Requirements.

To perform the AGC function, the computer needs to obtain several quantities of information about the control area.

**Area Requirements**

\[
AR = PSI - PAI + 10B* (SF - AF)
\]

**AR** - Control Area Requirement (positive AR indicates under-generation)

**PSI** - Power Scheduled Interchange (sum of all schedules)

**PAI** - Power Actual Interchange (sum of all control ties)

**B** - Control Area Bias Factor in MW per tenth Hertz

**SF** - Scheduled Frequency (normally 60 Hz)

**AF** - Actual Frequency

**Total Generation Desired** (Units in Automatic Mode)

\[
TGD = TGA + AR
\]

**TGD** - Total Generation Desired

**TGA** - Total Generation Actual

**AR** - Area Requirement

**Participation Factor** (Units in Automatic Mode)

\[
PF(i) = \frac{NR(i)}{SNR}
\]

(i) - Unit Index

**PF** - Participation Factor

**NR** - Nameplate Rating of Unit

**SNR** - Sum of all Units Nameplate Rating
Unit Generation Desired (Units in Automatic Mode)

UGD(i) = PF(i) * TGD
UGD(i) - Unit Generation Desired
PF(i) - Participation Factor for Unit (i)
TGD - Total Generation Desired

Unit Control Error (Units in Automatic Mode)

UCE(i) = UGD(i) - UGA(i)
UCE(i) - Unit Control Error
UGD(i) - Unit Generation Desired
UGA(i) - Unit Generation Actual

AGC looks at each unit's control error, then determines the number of raise or lower pulses required to reduce each unit's control error to zero. AGC then transmits the pulses to the units.

Many factors directly or indirectly cause the Area Requirement to fluctuate constantly, and the function of AGC is to keep the Area Requirement to a minimum by adjusting generation to meet system requirements. The AGC function is repeated every four seconds as presently set and is variable from two to six seconds.

The control tie quantities, system frequency, and generator quantities are received over a microwave system and most are analog quantities. The raise and lower pulses are transmitted over the same microwave system.

Example of AGC Function

4 Units in Manual Mode at 100 MW each
2 Units in Automatic Mode
Internal Loads and Losses ≈ 50 MW
Automatic Unit #1 - Nameplate Rating (NR = 150 MW)
Automatic Unit #2 - Nameplate Rating (NR = 75 MW)

PSI = 500 MW
PAI = 470 MW
B = 20
SF = 60 Hz
AF = 60 Hz
TGA = 150 MW (Units in Automatic Mode Only)

Unit #1 = 100 MW
Unit #2 = 50 MW

AR = PSI - PAI + 10 * B * (SF - AF)
AR = 500 - 470 + 10 * 20 * (60.0 - 60.0)
AR = 500 - 470 + 0 = +30 MW

TGD = TGA + AR
TGD = 150 + 30 = 180 MW

PF(i) = \frac{NR(i)}{SNR}

PF_1 = \frac{150}{225} = .667
PF_2 = \frac{75}{225} = .333

UGD(i) = PF(i) * TGD

UGD_1 = .667 * 180 = 120 MW
UGD_2 = .333 * 180 = 60 MW

UCE(i) = UGD(i) - UGA(i)

UCE_1 = 120 - 100 = +20 MW
UCE_2 = 60 - 50 = +10 MW

At this point the AGC program would determine the number of raise pulses needed to increase each unit generation sufficiently to reduce their respective Unit Control Error (UCE) to zero.

There exists a correlation between water flow and generation of power for each unit. Therefore, one can determine fairly accurately how much water has been released per power generated or at what level to generate to obtain the desired water releases.

One area of improvement in AGC is to use digital telemetering of control tie and generation quantities. The CRSP existing analog telemetering has a maximum of 2 percent error at full scale. CRSP is presently investigating digital telemetering.
MODELS APPLIED TO SALINITY PROJECTION

by

Ernest M. Weber, Christopher S. Donabedian and Merlin B. Tostrud*

The Colorado River Basin covers an area of 242,000 square miles, approximately one-twelfth of the conterminous United States, and 2,000 square miles in Mexico. It extends 1,400 miles from the Continental Divide in the Rocky Mountains to the Gulf of California.

The Colorado River Basin has a population of about 2.25 million and, through export projects, its water provides either full or supplemental supplies to an additional 12 million persons in the Southern California, Denver, Salt Lake City, Cheyenne, and Albuquerque areas. With the completion of the Central Arizona Project now under way, the Phoenix and Tucson areas will also be served from the lower mainstem.

Within the basin the regional economy is based on irrigated agriculture, mining, forestry, manufacturing, oil and gas production, and tourism. Approximately 2.4 million acres are irrigated within the basin, and hundreds of thousands more acres are also irrigated with water exported from the basin. In Mexico, about one-half million persons and 425,000 irrigated acres are served with Colorado River water.

Historically, the river, from both natural causes and man's activities, has carried a large dissolved mineral load resulting in salinity concentrations higher than those for most other major rivers.

Salinity concentrations increase throughout the length of the river. This increase is the result of two basic processes - salt loading and salt concentrating. Salt loading, which is the addition of mineral salts by both natural and man-made sources, increases the salinity concentration by increasing the total salt load carried by the river. Salt concentrating is the result of evapotranspiration or the diversion of water from the

*Engineering Geologist, Colorado River Board of California; Hydraulic Engineer, Colorado River Board of California; Civil Engineer, Colorado River Board of California; respectively.

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river, which causes a concentration of the salt load in a lesser volume of water.

In the mid 1960's the concern over the quality of the nation's water supply brought forth new legislation covering water quality.

The Environmental Protection Agency has interpreted Public Law 92-500, "Federal Water Quality Control Act Amendments of 1972" as requiring the establishment of numerical salinity criteria for the Colorado River. Consequently EPA has promulgated regulations that set forth the salinity control policy, procedures, and requirements for establishing water quality standards for salinity in the basin. In essence the regulation's policy is that the flow weighted average annual salinity in the lower mainstem of the Colorado River must be maintained at or below the average level found in 1972. Numeric criteria are to be adopted, along with a plan of implementation to achieve compliance with the criteria. The salinity problem is to be treated as a basin-wide problem that needs to be solved to maintain lower mainstem salinity at or below 1972 levels while the basin states continue to develop their compact apportioned waters.

The basin states in response to EPA's requirements, and in consideration of several other questions that were generated relative to certain sections of PL 92-500, formed the "Colorado River Basin Salinity Control Forum." The forum consists of representatives of water development and water quality control agencies. A work group was appointed by the forum to develop the numeric criteria and a plan of implementation of control measures to meet the criteria. The major part of the work group's activity was the future salinity projections made through the use of a river network model. Myron Holburt, Chief Engineer of the Colorado River Board is a member of the work group and Ernest Weber is his alternate. The salinity projections were made by Weber, Donabedian, and Tostrud for the work group. This is a report on how the projections were made.
River Network Model

A series of salt routing studies were conducted to provide estimates of future salinity levels at selected points in the basin under different assumptions as to both the available water supply and future water use. The studies were designed to provide estimates of salinity conditions with and without salinity control measures during the period 1974 through 1990.

The river network model developed by Richard W. Ribbens, of the Bureau of Reclamation-Engineering and Research Center was used. Basically the model is an accounting system with only limited simulation capabilities. River flow and salinity are routed through the river system using a time frame of one month.

Total dissolved solids (TDS) are used as the quality parameter. Since mass balance concepts are used, such items as chemical precipitation, dissolution and reactions of individual constituents are not considered but are included by appropriate inputs.

The reservoirs in the system may be operated in a number of ways and may be considered individually or conjunctively.

Program input includes the system configuration (network), reservoir characteristics, storage conditions, evaporation rates, operating criteria, upstream and downstream boundary values, and water use options. Various types of output from the program can be selected, including printed and cathode ray tube plots at various river locations and reservoirs. In addition, initial input data, detailed monthly results and summaries, as well as simple statistics, can be printed.

For this study, all known natural and existing man-made water use and salt loadings were identified for the river reach extending from Lake Powell to Imperial Dam. No attempt was made to model the river system above Lake Powell. Identification of individual uses and salt sources in the Upper Basin are not required for a study of their impact on salinity in the lower mainstem. Consequently, only the sum of the individual uses and salt loading were used.
The river below Lake Powell was divided into distinct reaches to determine future salinity levels. Estimates of future water use and salt loading for each appropriate reach of the river below Lake Powell and the accumulative effect above Powell were superimposed upon historic conditions for each year of the study. The changes were routed downstream with the accumulated impact reflected at downstream stations. The studies were made on a monthly basis using a range of water supply conditions and future depletion rates.

Input - Assumptions and Estimates

Operating criteria

"Criteria for Coordinated Long-Range Operation of Colorado River Reservoirs"(1), governs operation of basin reservoirs. A great deal of judgment is required to implement the criteria, as many factors must be considered, including environmental.

Because of the difficulty of trying to model these factors, only two main criteria were used in the study:

1. The first criteria requires a minimum yearly release of 8.23 million acre-feet from Glen Canyon Dam.

2. The second criteria calls for equalization of storage in Lakes Mead and Powell unless Lake Powell storage must be drawn below Lake Mead's in order to achieve the first criteria.

Mixing in reservoirs

For this study, the assumption was made that any salt or water entering a reservoir was instantaneously mixed with water already there. Thus, water anywhere in a reservoir was always of the same quality. Such was the assumption used by Ribbens (2) when he developed the model used in the study.

In a recent report by Hendrick (3) it was shown that a complete mixing model gave results equal to, or better than, any other model tested. Retention time in Hendrick's study was longer than a year, as it is with Lakes Mead and Powell.
Storage conditions

Reservoir parameters had to be set for model execution. In the study only Lake Powell and Lake Mead were modeled. The combined capacity of these reservoirs is about 85 percent of the basin's total usable capacity.

Area-capacity curves and monthly evaporation rates were used by the program to compute monthly evaporation from Lakes Mead and Powell.

It was assumed that water would not be drawn below the elevation at which power could no longer be produced. This was a capacity of about 12.4 maf in Lake Mead, and 6.1 maf in Lake Powell.

Beginning conditions for the two reservoirs consisted of the average salinity concentration of water in storage during calendar year 1973, and the volume of water in storage at the end of calendar year 1973.

Two other Lower Basin reservoirs, Lakes Havasu and Mohave are used, respectively, as a pumping forebay, and as a regulating facility to even out the fluctuating hourly releases made from Hoover power-plant upstream. Storage in these two reservoirs is relatively small and fluctuates very little from month to month. Consequently, these reservoirs were treated as river reaches.

The basin reservoirs above Lake Powell were not included since no attempt was made to model the system above Lake Powell. However, a yearly consumptive use of 110,000 acre-feet was depleted from Upper Basin supply to cover estimated evaporation loss from the reservoirs in that portion.

Water supply

To evaluate future possible salinity levels a number of water supply conditions were considered. Five water supply conditions were employed--a virgin flow of 12, 13, 14, 15, and 16 million acre-feet per year at Lee Ferry, Arizona. (The 1896-1974 average annual virgin flow is 14.9 million acre-feet.)
After reviewing historic hydrologic conditions, it was decided that within the time frame of this study, the next 15 years, this range of flows would most likely encompass the actual future flow.

Since it was necessary to develop average annual salinity projections to develop a plan for salinity control, no attempt was made to use a series of historic virgin flows or synthetic hydrology to predict future salinity. Because of the need for average values, basically the same end was achieved by using a constant water supply through each year of the study.

It should also be noted that the erratic flows of the Colorado River have been regulated by the construction of large volume reservoirs storage which is currently at 75 percent of full capacity. This reservoir system will dampen the variation in both the annual flow and salinity in the lower mainstem.

Water use

Predicting future water use under any set of circumstances is difficult. Within the Colorado River Basin, several factors made predicting even more difficult:

1. Over two-thirds of the river's supply is being consumed now, and competition for the remaining supply is keen.

2. There is keen competition as to what projects will be built in the future and when they will be built.

3. A significant portion of the unused supply will most likely be used to develop the basin's vast energy supply.

A number of different figures for total 1973 use were available from a number of different agencies. Following consultation with each of the basin states a base year value was determined for each state by category of use. (The term "use" means water consumed in a process.)

There were three recent studies available which predicted future water use in the basin.

There predictions were made by the Colorado River Board of California, the Pacific Southwest Interagency Committee, and the
U.S. Bureau of Reclamation. In addition, each state made an estimate of its own future water needs.

Originally the plan was to use one future depletion schedule for input to the model. However, it was apparent that agreement could not be reached by the basin states on just one schedule of depletions because each state had its own view concerning future development. Consequently, a range of future water use was developed, consisting of three possible rates of depletion: low, moderate, and high. Utilizing a range of depletion rates allows for greater flexibility in the study and indicates the extremes that could be encountered. The range encompassed all estimates from prior studies and the state estimates. Future depletions used as model input were estimated by a subcommittee of the work group consisting of a representative from the Upper and Lower Basin.

The depletion estimates were made on a project by project basis, some 150 projects and uses in all, as a required input item to the model. For presentation in this paper, the estimates were grouped by category of use.

The 1973 base year uses as well as the future increase in use over the 1973 base, by category, are shown in Table 1, and the total use is summarized in Figure 1.

Agriculture is predicted to use a major portion of presently unused water in the river system. Most of the agricultural water will be consumed by two projects: the Central Arizona Project and the Navajo Indian Irrigation Project. These two projects are under construction.

Total export out of the basin is expected to be reduced. Some new exports will be taking place, but these will be more than offset by California's reduction in diversions.

There are only about 75,000 acre-feet of water being consumed at present for coal development, which includes coal-fired electrical power generation. By 1990, the amount is expected to increase to 480,000 acre-feet.

Oil shale development for which only miniscule amounts of water are presently being used, will jump to 130,000 acre-feet of use by 1990.
Table 1. 1973 Water use and estimated increase in use over 1973 base, Colorado River Basin. (Thousands of acre-feet per year.)

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Low</td>
<td>Moderate</td>
<td>High</td>
<td>Low</td>
</tr>
<tr>
<td>UPPER COLORADO RIVER BASIN (Depletions)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Out of Basin Exports</td>
<td>651</td>
<td>170</td>
<td>220</td>
<td>375</td>
</tr>
<tr>
<td>In Basin Agricultural Use</td>
<td>2,175</td>
<td>100</td>
<td>135</td>
<td>250</td>
</tr>
<tr>
<td>In Basin Coal Development (Including electrical power generation)</td>
<td>59</td>
<td>160</td>
<td>190</td>
<td>305</td>
</tr>
<tr>
<td>In Basin Oil Shale</td>
<td>0</td>
<td>15</td>
<td>20</td>
<td>65</td>
</tr>
<tr>
<td>Other In Basin Uses (Fish &amp; Wildlife &amp; other M&amp;I Uses)</td>
<td>91</td>
<td>5</td>
<td>35</td>
<td>50</td>
</tr>
<tr>
<td>Total Upper Colorado River Basin</td>
<td>2,976</td>
<td>450</td>
<td>600</td>
<td>1,045</td>
</tr>
<tr>
<td>LOWER COLORADO RIVER BASIN (Diversions less returns)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Out of Basin Exports</td>
<td>4,538</td>
<td>-500</td>
<td>-450</td>
<td>-300</td>
</tr>
<tr>
<td>In Basin Agricultural Use</td>
<td>1,461</td>
<td>80</td>
<td>125</td>
<td>170</td>
</tr>
<tr>
<td>In Basin Coal Development (Electric power generation)</td>
<td>15</td>
<td>20</td>
<td>20</td>
<td>25</td>
</tr>
<tr>
<td>In Basin Oil Shale</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Other In Basin M&amp;I Uses</td>
<td>90</td>
<td>70</td>
<td>95</td>
<td>120</td>
</tr>
<tr>
<td>In Basin Fish &amp; Wildlife &amp; Recreation Uses</td>
<td>39</td>
<td>0</td>
<td>20</td>
<td>45</td>
</tr>
<tr>
<td>Total Lower Colorado River Basin</td>
<td>6,143</td>
<td>-330</td>
<td>-190</td>
<td>60</td>
</tr>
</tbody>
</table>
Figure 1. Projected water use from Colorado River—excluding all main-stream losses and deliveries to Mexico.
Other in-basin uses include water for recreation, fish and wildlife, and municipal and industrial needs not associated directly with any of the other major uses. M & I water for Central Arizona Project is an example of such use.

The water use projections used in the salt routing studies, represent, what are thought to be the best available information at this time.

Upper Basin

The annual average inflow of salt to Lake Powell under 1973 conditions of development for the five average annual virgin flow levels considered in the analysis was estimated in the following manner.

A relationship between flow and salt load at Lee Ferry was established using records for the 24-year period 1929, (when water quality measurements began), through 1962, just prior to closure of Glen Canyon Dam. During this period, the relationship between annual streamflows and annual salt loads was nearly stable, as indicated by a USGS study (4).

The relationship between flow and salt load was established by means of a least squares plot. Both linear and parabolic plots were tried, and the linear plot was thought to represent the best fit. The least squares equation derived for the plot was found to be \( S = 2,989,000 + 0.4856F \) where \( S \) is the annual salt load in tons and \( F \) the annual depleted flow in acre-feet.

Because considerable development has occurred in the Upper Basin since 1962, salt load amounts obtained from the above relationship had to be adjusted to reflect the impact of those developments. The USBR (5) has estimated what the salt load into Lake Powell would have been for the 1941 through 1970 period if the 1970 level of development had prevailed throughout the entire period. USBR's estimated average annual salt load exceeded the amount derived from the above equation by about 350,000 tons for an average annual depleted flow of 10,812,000 acre-feet. This difference was attributed to the increased level of Upper Basin development that has occurred during the period 1962 to 1970.
It was assumed that this difference would be the same for the range of virgin flows considered in this study. Also, because conditions of development in 1973 were not significantly different from those in 1970, it was further assumed that the difference in salt load would also be the same for the period 1962 through 1973. Thus, the values obtained from the relationship equation were adjusted by adding 350,000 tons to the left of the equation which then became \( S = 3,339,000 + 0.4856F \).

Using the adjusted equation, the inflow of salt to Lake Powell under 1973 conditions of development was estimated for the five average annual virgin flow levels as shown in Table 2.

Table 2. Estimated average annual inflow of salt to Lake Powell under 1973 conditions of development.

<table>
<thead>
<tr>
<th>Virgin Flow (1000 Acre-Feet)</th>
<th>Depleted Flow (1000 Acre-Feet)</th>
<th>Salt Load (1000 Tons)</th>
</tr>
</thead>
<tbody>
<tr>
<td>16,000</td>
<td>12,914</td>
<td>9,610</td>
</tr>
<tr>
<td>15,000</td>
<td>11,914</td>
<td>9,120</td>
</tr>
<tr>
<td>14,000</td>
<td>10,914</td>
<td>8,640</td>
</tr>
<tr>
<td>13,000</td>
<td>9,914</td>
<td>8,150</td>
</tr>
<tr>
<td>12,000</td>
<td>8,914</td>
<td>7,670</td>
</tr>
</tbody>
</table>

Lower Basin

Little of the tributary inflow of water and salt between Lee Ferry and Hoover Dam is measured. Consequently, the estimate was based on a study made by the USBR (5). The USBR, using the 30-year period 1941 through 1970, has estimated that the average annual net tributary inflow of water (excluding evaporation losses in Lake Mead) between the two points under 1970 conditions of development is 709,000 acre-feet and the corresponding net salt gain is 1,904,000 tons.

The USBR's estimates, after verification by an independent analysis made by the Colorado River Board, were used in this study for the 14,000,000 acre-foot/year virgin flow level of supply at Lee Ferry. Tributary inflow for the four remaining levels of virgin supply were
estimated assuming that the tributary inflow of water and salt varied directly as the virgin flow. These estimates are shown on Table 3.

Table 3. Average annual net tributary inflow of water and salt between Lee Ferry and Hoover Dam. a

<table>
<thead>
<tr>
<th>Virgin Flow (1000 Acre-Feet)</th>
<th>Tributary Inflow of Water (1000 Acre-Feet)</th>
<th>Tributary Inflow of Salt (1000 tons)</th>
</tr>
</thead>
<tbody>
<tr>
<td>16,000</td>
<td>810</td>
<td>2,176</td>
</tr>
<tr>
<td>15,000</td>
<td>760</td>
<td>2,040</td>
</tr>
<tr>
<td>14,000</td>
<td>709</td>
<td>1,904</td>
</tr>
<tr>
<td>13,000</td>
<td>658</td>
<td>1,768</td>
</tr>
<tr>
<td>12,000</td>
<td>608</td>
<td>1,632</td>
</tr>
</tbody>
</table>

a Includes stream losses but does not include evaporation losses in Lake Mead.

Inflow-outflow conditions below Hoover Dam require a more delicate balance than above that point. Under present conditions, a change in flow of 100,000 af (other conditions being the same) produces a 15 mg/l change in salinity at Imperial Dam. A change in salt loading of 100,000 tons produces a 12 mg/l change at Imperial.

Because the stretch of river from Hoover Dam to Imperial Dam is so sensitive, losses or gains of salt and water were analyzed very carefully. Most studies done on this stretch have used the mass-balance method. Unfortunately, losses determined by this method are within the ±5 percent flow gaging accuracy that can be expected at Hoover Dam. As a result, the estimates are in considerable variance. Each of the studies conducted on this subject was considered and the decision was made to take a weighted average of them. More weight was placed on recent studies by Ribbens and Wilson (5) of the USBR and by the USGS. (4)

The resulting numbers used in the analysis are:
Reach | Water (af) | Salt (tons)
---|---|---
Hoover Dam to Parker Dam | 300,000 | 320,000
Parker Dam to Imperial Dam | 250,000 | 0

*Palo Verde Irrigation District, Colorado River Indian Reservation, and Metropolitan Water District of Southern California.

Included in these figures are tributary inflow, mainstream evaporation, phreatophyte consumption, and minor man-made uses along the river as of 1973.

Upper Basin

The total salt load now contributed by sources in the Upper Basin is included in the salt load entering Lake Powell under 1973 conditions of development. Because this quantity of salt is an input item to the model, it was necessary to estimate only future changes in salt loads as regards the Upper Basin. The changes were then superimposed on the salt load entering Lake Powell under 1973 conditions.

The salt load per acre contributed by future irrigation projects was assumed to be similar to the salt load contributed by lands now under irrigation near each proposed project. Information on present salt pick-up rates was obtained from a study conducted by the Environmental Protection Agency. (6) For the proposed Upper Basin projects included in this study, estimated salt pick-up rates ranged from a low of 0.3 ton per acre to 3.5 tons per acre.

It was anticipated that most future Upper Basin industrial development will be devoted to the oil shale, coal gasification, and electric power generation industries. It was assumed that these industries, together with their associated municipal uses, will consumptively use all the water diverted and will dispose of their wastes in such a manner as would preclude the return of any salts to the river. Consequently, the net effect on the salt load entering Lake Powell would be a reduction equal to that contained in the water used. The quantity of salt removed from the system was determined by choosing a most probable source
of water for each future project and assuming that the present concentration of each source would remain roughly unchanged during the period of study. Then a weighted average for each of the three industries was computed. The weighted average salt removal rates in tons per acre-foot of water diverted amounted to 0.51 for the oil shale industry, 0.35 for coal gasification, and 0.47 for electric power generation.

Transmountain diversions export water out of the basin for use elsewhere and, consequently, remove salt from the system. The rate of salt removal by each project was determined by assuming that the present salt concentration of each diversion point would remain roughly unchanged during the study period. Salt removal rates varied from 0.06 to 0.19 ton per acre-foot.

**Lower Basin**

A salt pick-up rate of 0.5 ton per acre was used for all irrigated areas in the Lower Basin. This pick-up rate was based on information developed by the Environmental Protection Agency (6).

Present and projected urban water uses in the Lower Basin are very small when compared with other uses. Consequently, the salt load from this source is also small. It was estimated that urban uses contribute about 0.5 ton of salt per acre-foot water diverted. This estimate was based on a brief analysis of the City of Needles, assuming 0.07 ton per capita salt pick-up plus an arbitrary increase based on the fact that the waste water, with its salt load, infiltrates and picks up additional salts on its way back to the river.

The amount of salt removed by water exported out of the Lower Basin, such as diversions by the Metropolitan Water District, is not a model input. The amount of salt for each export item is computed and accounted for internally by the model.

**Salinity control measures**

To comply with the proposed numeric salinity criteria, the forum work group considered a number of salinity control measures.
that could be implemented to reduce the salt load of the river and to minimize future increases in loading. The salinity control measures consist of: (1) no salt return for electrical generation, coal development, coal gasification and oil shale industries; (2) construction of 16 Federal salinity control projects specified in P.L. 93-320 (the "Colorado River Salinity Control Act"); and (3) reformulation of three authorized Upper Basin water development projects.

A schedule for implementation of salinity control measures was determined. The control of industrial salt return was based on the water use projections. The schedule for construction of the 16 salinity control projects and for reformulation was obtained from a preliminary schedule of the Bureau of Reclamation. The estimated salt removed in 1990, listed by category of salinity control measures is:

<table>
<thead>
<tr>
<th>Control Measure</th>
<th>Salt Removed (1000's Tons)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Low</td>
</tr>
<tr>
<td>No industrial salt return</td>
<td>278</td>
</tr>
<tr>
<td>Four authorized salinity control projects</td>
<td>514</td>
</tr>
<tr>
<td>Twelve future control projects</td>
<td>1,130</td>
</tr>
<tr>
<td>Project reformulation</td>
<td>121</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>2,043</strong></td>
</tr>
</tbody>
</table>

**Results of Salt Routing Studies**

As has been described, there are over 150 projects that will affect future salinity of the river. It was decided, almost from the outset of the study, that effects of each individual project on salinity at a number of points under different flow conditions could not be studied. Condensation was necessary. As was described earlier, five different Lee Ferry virgin flow conditions (12 maf to 16 maf) were analyzed.

It was decided to have the computer print salinity calculations for five points along the river. In this section, results are presented for only two of those points—Hoover Dam and Imperial Dam.
The first step in determining the effects of salinity control projects was to estimate future salinities without any control projects whatsoever. This is represented in the top line of Figures 2 to 9. Therefore, 15 computer runs were made with no salinity control projects as a base condition, using the combinations of 5 virgin water supplies and 3 depletion rates.

Next, studies were run in which salt from industrial projects was not returned. Salt within water removed for power plant cooling, coal gasification plants, and oil shale development was not returned to the river. It was assumed such devices as evaporation ponds would effectively retain all salt. The results of these studies are presented in the second line down on Figures 2 to 9.

In the next set of studies, salt removed by the four authorized salinity control projects as identified in PL 93-320 in addition to the salt of industrial projects, was taken out. The results of these studies are shown in the third line down on Figures 2 to 9. Thus, the difference between lines two and three reflects the effects of the four authorized salinity control projects.

In a like manner, effects of the 12 additional salinity control projects were determined. These projects are under investigation, but have not, as yet, been authorized. Such a project is Glenwood-Dotsero Springs Unit in Colorado which could remove 200,000 tons of salt per year. Results of computer runs with salt from industrial projects and the 16 salinity control projects removed are shown in the bottom line of Figures 2 to 9.

A small reduction in salinity is anticipated by making changes in three authorized Bureau of Reclamation projects in the Upper Basin. The proposed changes include a shift from agriculture to M & I water use along with changes in areas of proposed irrigation to less saline soils. The effects of such changes, referred to as "project reformulation," were also analyzed. However, because their effects were small, a complete set of computer runs was not executed. The analysis was limited to only one supply and depletion estimate (15 maf with a moderate depletion rate), and estimates were made for the other supply
Figure 2. Projected salinity at Hoover Dam.
Figure 3. Projected Salinity at Imperial Dam.
Figure 4. Projected Salinity at Hoover Dam.

247
Figure 5. Projected salinity at Imperial Dam.
Figure 6. Projected salinity at Hoover Dam.
15 M. Af. / Yr. Supply
MODERATE DEPLETION RATE

No Industrial Salt Return
Four Authorized Salinity Control Projects
12 Additional Salinity Control Projects
Project Reformulation

1972 Av. Salinity

Excess Flows At Imperial Dam Due To Reservoir Spillage

Flow in Excess Of Demand (1,000 A.F.)

Figure 7. Projected salinity at Imperial Dam.

250
Figure 8. Projected salinity at Hoover Dam.
Figure 9. Projected salinity at Imperial Dam.
and depletion schedules. A hash mark below the bottom line of Figures 2 to 9 shows the effect of "project reformulation."

The bottom line of Figures 2 to 9 (which may be lowered slightly to account for "project reformulation") represents expected salinity concentrations if all the salinity control projects are developed on the time schedule anticipated, and all other study assumptions are met.

Actual 1974 salinity concentrations were used as the starting points in plotting Figures 2 to 9. The curves were then constructed by superimposing annual salinity changes computed by the model on 1974 salinity values. The 1972 flow-weighted salinity concentrations, which are the numeric criteria are plotted on the figures as a point of reference.

In addition to the above described runs a number of runs were made to study other aspects of activities effecting the river's salinity. Of particular interest was the impact of each of the categories of control at four key stations; Lee Ferry, Hoover, Parker, and Imperial Dams.

Each of the industrial activities--power plant cooling, oil shale and coal gasification--were evaluated in separate model runs. Table 4 shows the reduction in projected salinity due to the categories of control measures. The values will differ depending on the depletion rate and supply schedule used.

The impact on salinity of projected depletions for fish and wildlife enhancement was also tested. In making this run it was assumed that water not depleted for this purpose would remain in the river and not be allocated for other uses. It was found that the increased water use for enhancement, under a 15 million acre-foot supply and a moderate depletion rate, would increase salinity at Imperial Dam by 7 mg/l by 1990.

The model was used to answer several questions not directly related to predicting future salinities. One such question concerned the lag time between effects of a project at Lake Powell and effects of the same project at Imperial Dam. It was known that reservoir mixing and retention would cause a lag. But for how long? Special runs were
Table 4. Projected reduction in salinity due to salinity control measures.\textsuperscript{a}

<table>
<thead>
<tr>
<th>Control Measures</th>
<th>Lee Ferry (Concentration)</th>
<th>Below Hoover Dam (Concentration)</th>
<th>Below Parker Dam (Concentration)</th>
<th>Below Imperial Dam (Concentration)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>No Industrial Salt Return</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Power Plant Cooling (Incl. attendant coal devec.)</td>
<td>6</td>
<td>4</td>
<td>6</td>
<td>18</td>
</tr>
<tr>
<td>Coal Gasification Industry</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Cil Shale Industry</td>
<td>5</td>
<td>3</td>
<td>4</td>
<td>4</td>
</tr>
<tr>
<td><strong>Subtotal</strong></td>
<td>12</td>
<td>8</td>
<td>11</td>
<td>23</td>
</tr>
<tr>
<td><strong>Salinity Control Projects</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Four Authorized Projects</td>
<td>26</td>
<td>32</td>
<td>33</td>
<td>39</td>
</tr>
<tr>
<td>12 Additional Projects</td>
<td>59</td>
<td>49</td>
<td>51</td>
<td>64</td>
</tr>
<tr>
<td><strong>Subtotal</strong></td>
<td>85</td>
<td>81</td>
<td>84</td>
<td>103</td>
</tr>
<tr>
<td><strong>Reformulation</strong></td>
<td>4</td>
<td>2</td>
<td>1</td>
<td>2</td>
</tr>
<tr>
<td><strong>Totals</strong></td>
<td>101</td>
<td>91</td>
<td>96</td>
<td>128</td>
</tr>
</tbody>
</table>

\textsuperscript{a} Concentration reduction reflects dilution within system reservoirs, variations in completion dates of control measures, and salt removed with out-of-basin diversions and non-return uses.
made in which 500,000, 750,000 and 1 million tons of salt were re-
moved from the Upper Basin starting in 1976. To determine the lag,
these runs were compared to a base run in which no salt was removed.
By so doing, it was found that an equal percentage change in concentra-
tion occurred at Imperial Dam about three years after it occurred at
Lake Powell. In addition, this series of runs demonstrated that the
effect of salt removal from the Upper Basin on Lower Basin salinity
amounts to 0.09 mg/l per 1000 tons of salt at Imperial Dam in 1990.

By analyzing all of the computer runs it was possible to determine
which sets of conditions would meet the numeric salinity criteria. Annua
average salinity levels can be maintained at or below 1972 levels at
Hoover and Imperial Dams if the following conditions exist:

1. Full implementation of salinity control measures.

2. Virgin flow at Lee Ferry of 14 million acre-feet/year or more
with a low depletion rate and 15 million acre-feet per year or more with
a moderate depletion rate.

Summary and Conclusions

As part of the forum's efforts to establish numeric criteria for salinity
and a plan of implementation, a salt routing was employed to make a
number of future salinity projections. The U.S. Bureau of Reclamation
Ribbens model used was well suited to the needs of the forum. It was
simple, understandable, and well documented. Thus it was easy to use
and gave results which could be utilized with confidence.

The salinity of the river system is greatly influenced by flow. For
example, under the same level of development the salinity at Imperial
Dam in 1990 with a 12 million acre-foot per year supply would be
156 mg/l greater than with a 16 million acre-foot supply. In order to
maintain salinities in 1990, at or below those found in the lower main
stem in 1972 while the basin states continue to develop their compact
apportioned waters, salinity control measures must be implemented.
Only under low rates of development and high annual flows could the
criteria be met without salinity control measures.
References


The first phase of the Colorado River Basin input-output analysis began in 1962 under sponsorship and funding from the U.S. Public Health Service and continued through mid-1968 with funding shifting to the Federal Water Pollution Control Administration. The economic model covered each of the six sub-basins of the Colorado River Basin and the original concerns were with the relationship between salinity in the water and economic activity. The bulk of the study was conducted by the Bureau of Economic Research at the University of Colorado, the Department of Economics at the University of New Mexico, and the Economic Research Service of the U.S. Department of Agriculture at Logan, Utah (Udis, 1967, 1968).

The results of this phase of the study were utilized by various federal government agencies. The Public Land Law Review Commission used the three upper sub-basin input-output tables to analyze the economic consequences of alternative public policies for the uses of federal lands. These dealt with range livestock, oil shale, big game hunting, winter sports, grazing lands, and the pulp and paper industry. This work was combined with similar analysis for the state of Washington and the results appeared in 1969 in a report entitled Study of Impact of Public Lands on Selected Regional Economies, prepared by the Consulting

*Respectively, Professor of Economics and Director, Bureau of Economic Research, University of Colorado; Professor of Economics and Chairman, Department of Economics, University of Colorado; and Environmental Consultant.

1 The component counties of each sub-basin are listed in Table 1.
Table 1. Component counties of each sub-basin of the Colorado River Basin.

<table>
<thead>
<tr>
<th>Sub-basin</th>
<th>State and County</th>
<th>Sub-basin</th>
<th>State and County</th>
</tr>
</thead>
<tbody>
<tr>
<td>I. Upper Main</td>
<td>Colorado</td>
<td>III. San Juan</td>
<td>Utah</td>
</tr>
<tr>
<td>Stem</td>
<td>1. Delta</td>
<td>(cont'd.)</td>
<td>1. Garfield</td>
</tr>
<tr>
<td></td>
<td>2. Dolores</td>
<td></td>
<td>2. Kane</td>
</tr>
<tr>
<td></td>
<td>3. Eagle</td>
<td></td>
<td>3. San Juan</td>
</tr>
<tr>
<td></td>
<td>5. Grand</td>
<td>IV. Little</td>
<td>Arizona</td>
</tr>
<tr>
<td></td>
<td>6. Gunnison</td>
<td>Colorado</td>
<td>Arizona</td>
</tr>
<tr>
<td></td>
<td>7. Hinsdale</td>
<td></td>
<td>1. Apache</td>
</tr>
<tr>
<td></td>
<td>8. Mesa</td>
<td></td>
<td>2. Navajo</td>
</tr>
<tr>
<td></td>
<td>9. Montrose</td>
<td></td>
<td>1. McKinley</td>
</tr>
<tr>
<td></td>
<td>10. Ouray</td>
<td></td>
<td>New Mexico</td>
</tr>
<tr>
<td></td>
<td>11. Pitkin</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>12. San Miguel</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>13. Summit</td>
<td>V. Gila</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1. Moffat</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>2. Rio Blanco</td>
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<td></td>
</tr>
<tr>
<td></td>
<td>3. Routt</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Utah</td>
<td>1. Carbon</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>2. Daggett</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>3. Duchesne</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>4. Emery</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>5. Uintah</td>
<td>VI. Lower</td>
<td>Arizona</td>
</tr>
<tr>
<td></td>
<td>6. Coconino</td>
<td>Main Stem</td>
<td>Nevada</td>
</tr>
<tr>
<td></td>
<td>1. Lincoln</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>2. Mohave</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>3. Yuma</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>4. Uinta</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Wyoming</td>
<td>1. Lincoln</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>2. Sublette</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>3. Sweetwater</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>4. Uinta</td>
<td></td>
<td></td>
</tr>
<tr>
<td>III. San Juan</td>
<td>1. Archuleta</td>
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<td>Utah</td>
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<td></td>
<td>2. La Plata</td>
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<td>1. Washington</td>
</tr>
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<td></td>
<td>3. Montezuma</td>
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<td></td>
<td>4. San Juan</td>
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<td></td>
</tr>
<tr>
<td>New Mexico</td>
<td>1. San Juan</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Services Corporation of Seattle, Washington. In addition, the Federal Interagency Group comprising the Pacific Southwest Interagency Committee used our work as the basis for the economic analysis appearing in the Comprehensive Framework Studies for the Upper and Lower Colorado Regions published in 1971.

Since mid-1970 we have been funded by the Economic Development Administration of the U.S. Department of Commerce to develop models of air and water pollution to link with the economic model. The models have been developed to the point where a change in the level of output in any of the economic sectors can be exhaustively traced in terms of its impact on other portions of the economy, as well as upon the level of river salinity and the emission of five major airborne residuals (total suspended particulates, sulfur dioxide, oxides of nitrogen, carbon monoxide, and total unburned hydrocarbons). This work covers only the three upper sub-basins and a report describing the models was submitted to EDA in the summer of 1973 (Udis et al., 1973). Since then the models have been applied to problems in a specific area; namely, sharply increased coal output from underground mines along the North Fork of the Gunnison River in Delta and west Gunnison Counties, Colorado. Part of the current effort has involved the reduction of the I/O tables for the Upper Main Stem sub-basin to cover the six counties of Colorado State Planning and Management Region No. 10. In addition, other forms of economic analysis are being used to trace the broad impacts of increased coal production by 1980 on the North Fork area. A component of this work concentrates upon a socio-economic analysis of the region at the sub-county level. This involved a detailed analysis of the size, characteristics and distribution of the existing population, an inventory of existing housing and other items of social overhead capital and services, and a projection of adequacy of this inventory to meet expected population growth resulting from the coal development.
In this paper we shall describe briefly the three interacting models which comprise the heart of our analytical approach. Input-output analysis provides a means of representing industrial structure and determining how changes in the output of any industry will affect other industries. The primary focus of the analysis is the inter-relationship of firms in the dual roles of purchasers of inputs and producers of outputs. This interrelationship is summarized in a transactions table which tabulates dollar sales and purchases for each industrial sector.

The Structure of the I/O Model

A simplified I/O transactions table is presented in Figure 1 (Richardson, 1972, p. 18-30). It is presented for illustrative purposes only and hence aggregated sectors and shows only major relationships. Each row in the table shows the disposition of the output of each industry and sector of the economy. Thus an industry's output is assumed to be distributed to other processing sectors of the economy, which represent intermediate demand, and to such components of final demand as households' consumption, private investment, government spending and exports. By convention, intermediate demand plus final demand sums to total gross output of each particular industry. The purchases of each industry are recorded in the vertical columns. Here again there are various categories of inputs to producing sectors. These include purchases of an industry from all other industries (intermediate

\[\text{\footnotesize 2For a simple introduction to input-output analysis, the reader is referred to Miernyk (1965). A more sophisticated treatment may be found in Chenery and Clark (1959). Detailed and advanced critiques of the method are available in Conference on Research in Income and Wealth (1955) and Morgenstern (1954). The basic reference to input-output analysis are those of its modern father, Leontief (1951) and Leontief and others (1953). A convenient collection of Leontief's articles has been published as Input-Output Economics (1966). Interesting applications of input-output are presented in Richardson (1972).}\]
<table>
<thead>
<tr>
<th>From</th>
<th>Purchasing sectors</th>
<th>Local final demand</th>
<th>Total gross output</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1 \ldots f \ldots n</td>
<td>Households \quad Private Government \quad Investment</td>
<td>Exports</td>
</tr>
<tr>
<td>1</td>
<td>\ldots</td>
<td>C_1 \quad I_1 \quad G_1</td>
<td>E_1 \quad X_1</td>
</tr>
<tr>
<td>i</td>
<td>\ldots</td>
<td>C_i \quad I_i \quad G_i</td>
<td>E_i \quad X_i</td>
</tr>
<tr>
<td>n</td>
<td>\ldots</td>
<td>C_n \quad I_n \quad G_n</td>
<td>E_n \quad X_n</td>
</tr>
<tr>
<td>Labour</td>
<td>L_1 \ldots L_f \ldots L_n</td>
<td>L_C \quad L_I \quad L_G</td>
<td>L_E \quad L</td>
</tr>
<tr>
<td>Other value added</td>
<td>V_1 \ldots V_f \ldots V_n</td>
<td>V_C \quad V_I \quad V_G</td>
<td>V_E \quad V</td>
</tr>
<tr>
<td>Imports</td>
<td>M_1 \ldots M_f \ldots M_n</td>
<td>M_C \quad M_I \quad M_G</td>
<td>- \quad M</td>
</tr>
<tr>
<td>Total gross outlay</td>
<td>X_1 \ldots X_f \ldots X_n</td>
<td>C \quad I \quad G</td>
<td>E \quad X</td>
</tr>
</tbody>
</table>

Figure 1. Simplified, input-output transactions table.
inputs) and from what may be viewed as primary inputs such as labor, capital, and imports. Thus a transactions table should fully explain the disposition of each industry's output as well as its outlays for inputs. Since profits are counted as a necessary return to capital for its services, each processing sector industry must show an equality between its total gross output and its total gross outlay. This requirement for equality does not apply to the individual components constituting final demand and final payments.

It should be noted that the economy is assumed to consist of several classes of sectors: (1) an autonomous sector which responds to forces external to the regional economy, and (2) a non-autonomous sector which is responsive to changes originating within the regional economy.

While useful as a representation of interindustry accounting, the transactions table does not yield an answer to the basic question: How will a change in the output of one industry affect all other industries in the region? For this purpose additional steps are necessary which involve mathematical manipulation of figures in the transactions table. The goal of the analysis is to unearth structural interrelationships within the non-autonomous sectors. Figure 2 is a skeletonized version of the transactions table which was presented in somewhat greater detail as Figure 1. It represents a framework of three processing sector industries, aggregate final demand and final payment (value added) sectors, gross output, and gross outlays. Final demand is indicated by "Y" and "V" signifies value added. Summing across the rows using the first row for illustration, it may be shown that

\[ X_1 = X_{11} + X_{12} + X_{13} + Y_1. \]

Assuming that industry 1's output is allocated to each purchasing industry (1, 2, and 3) as a stable function of the output of the buying industries, the first equation may be rewritten as follows:

\[ X_1 - a_{11}X_1 - a_{12}X_2 - a_{13}X_3 = Y_1 \]

where \( a_{11} = \frac{X_{11}}{X_1}; a_{12} = \frac{X_{12}}{X_2}; \]

and \( a_{13} = \frac{X_{13}}{X_3}. \)
The a’s are known as direct input coefficients and represent the direct requirements of the output of any sector i per unit of output of any other purchasing sector j, where both i and j run from 1 to n. The basic underlying assumption is that the value of goods and services delivered by industry i to other producing sectors is a linear and homogeneous function of the level of output of the purchasing sector j. The limiting assumptions are the following: no joint products appear as each commodity is viewed as being supplied by a single industry which utilizes one method of production; the linear input function implies constant returns to scale and no substitution occurs between inputs; external economies and diseconomies are not present, i.e., the total effect of production is the sum of the separate effects; the system is in equilibrium at given prices; and no capacity restraints are assumed leading one to ignore problems of capital formation (in static forms of I/O analysis).

<table>
<thead>
<tr>
<th></th>
<th>To 1</th>
<th>To 2</th>
<th>To 3</th>
<th>Final demand</th>
<th>Gross output</th>
</tr>
</thead>
<tbody>
<tr>
<td>From 1</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1 X_{11}</td>
<td>X_{12}</td>
<td>X_{13}</td>
<td>Y_1</td>
<td>X_1</td>
<td></td>
</tr>
<tr>
<td>2 X_{21}</td>
<td>X_{22}</td>
<td>X_{23}</td>
<td>Y_2</td>
<td>X_2</td>
<td></td>
</tr>
<tr>
<td>3 X_{31}</td>
<td>X_{32}</td>
<td>X_{33}</td>
<td>Y_3</td>
<td>X_3</td>
<td></td>
</tr>
</tbody>
</table>

\[ V = V_1 V_2 V_3 \]

\[ \text{Gross outlay} \quad X_1 \quad X_2 \quad X_3 \quad Y \quad X \]

Figure 2. Skeletal input-output table.

Input-output analysis links the interaction of the following elements of an economic system: final demands arising in the needs of households, investment, government and exports, the input requirements of each industry and their gross outputs. Input-output analysis
provides a means to determine the effects of specific changes in final demand upon gross output of specific industries, given the input requirements or coefficients matrix. There are various levels of effects which must be taken into account. These include not only the direct impact—that is, the first round of requirements—but also indirect effects of additional deliveries of these inputs on all industries in the economy.

Typically the I/O system is presented in matrix form where the overall matrix equation represents a set of individual equations for each sector. Thus, \( X = AX + Y \), where \( X \) and \( Y \) are column vectors of gross output and final demand and \( A \) represents an \( n \) by \( n \) matrix of direct input coefficients, \( a_{ij} \). After rearrangement the set of equations resembles the following:

\[
\begin{bmatrix}
1 - a_{11} & -a_{12} & \ldots & -a_{1n} \\
-a_{21} & 1 - a_{22} & \ldots & -a_{2n} \\
\vdots & \vdots & \ddots & \vdots \\
-a_{n1} & -a_{n2} & \ldots & 1 - a_{nn}
\end{bmatrix}
\begin{bmatrix}
X_1 \\
X_2 \\
\vdots \\
X_n
\end{bmatrix}
= 
\begin{bmatrix}
Y_1 \\
Y_2 \\
\vdots \\
Y_n
\end{bmatrix}
\]

The result is inverted, enabling the inverse matrix to express gross output as a function of final demand: \( X = (I - A)^{-1}Y \). The term \( (I - A)^{-1} \) is known as the Leontief inverse matrix, sometimes identified as \( B \). Each coefficient entry in this table represents the direct and indirect requirements of sector \( i \) per unit of final demand for the output of sector \( j \). Thus, \( X_1 = b_{i1}Y_1 + b_{i2}Y_2 + \ldots + b_{i1}Y_1 + b_{i2}Y_2 + \ldots + b_{in}Y_n \).

The indirect input requirements reflect the fact that a change, for example an increase in the output of a particular industry, will require that industry to increase its purchases of inputs from its suppliers, but the input requirements of the supplier industries will
frequently reverberate back on the originating industry plus others as well. This also has employment income consequences. A table of direct and indirect coefficients of input requirements takes all these levels of impact into account. When the inverse matrix B is multiplied by a particular size and composition of final demand one can determine the gross output level for each industry; thus providing an extremely useful analytical tool which provides the means to measure the total impact on the economy of an originating change in final demand. The input-output approach is particularly useful in analyses in which industry specific information is required, i.e., in which the composition of output is important. Different groupings of industries yielding the same total value of output may have widely differing requirements for labor, land, water, power and municipal services and also differ in terms of their polluting characteristics.

It was determined early in the study that input-output tables then available at the national level were not appropriate for use because of the sharp dissimilarities between the economy of the highly industrialized United States and of the Colorado River Basin. The CRB was lightly populated with an economy oriented to agriculture, mining, and tourism. In the survey of the Colorado River Basin something over 2,000 interviews were conducted in the field by graduate student interviewers who had been trained in the procedures both of conventional business accounting and input-output accounting. The resulting data were utilized to determine average coefficients of direct input requirements for each industrial sector. Independent estimates of final demand were derived which together with the direct and indirect coefficients yielded the gross output and gross outlays figures. In addition new coefficients of direct input requirements were projected for the year 1980 based principally upon the "best practices" technique developed by the U.S. Bureau of Labor Statistics. This method assumes that the productivity or technical production function reflecting inputs and outputs of more advanced
firms will become diffused over a period of time and eventually become the typical or average pattern of input coefficients for each industry.

In order to accommodate the space limitations we shall turn directly to the applications of the I/O model to water and air quality considerations.

**Detailed description of the Colorado River Basin hydro-salinity model**

The C. U. hydro-salinity model is a digital computer adaptation and extension of an analog computer model developed by M. Leon Hyatt and others at Utah State University. The model consists of mathematical and logical representations of the various hydrological and routing functions which occur in all river basins. The model thus is not limited to any particular geographic area. The specific characteristics of each basin are incorporated into the model during the calibration process.

The model includes an economic-to-hydrologic interfacing routine which takes total gross output (TGO) data generated by a regional economic model and converts these into demands for water and other consequent impacts. The model also allows for the presence of both within-basin and end-of-basin reservoir storage. Since most within-basin storage is used for irrigation, a feedback mechanism which translates shortages of irrigation water into increased reservoir releases has also been included.

The hydro-salinity model can be viewed as consisting of three components: an economic I/O interfacing package; a hydrologic model; a salt flow model overlying the hydrologic model. Depletion of water from a sub-basin occurs only through evapotranspiration.

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municipal and industrial consumption, and exports. The remainder of the hydrologic model concerns itself with changes in various stocks, flows, and routings.

The total inflow of any basin is made up of contributions from: outflows from higher basins on the mainstem river; runoff of tributaries within the basin; precipitation and snowmelt in the valley bottoms; and trans-basin imports into the basin. The model allows for inflow from up to three prior basins.

The runoff of tributaries within the basin consists of both surface flow and groundwater flow. It is usually impossible to get data for every tributary within the basin, so the model is designed to use one or two "key" streams as representative of runoff patterns within the basin. These flow patterns serve as a basis for deriving a correlated value for both surface inflow and additions to valley bottom groundwater stocks.

Determining whether a particular amount of precipitation will take the form of rain or snow is handled through the use of air temperature of 32°F as the transition point. Both valley bottom precipitation and runoff from valley bottom snowmelt contribute to mainstem flow within the basin and are included in the function explaining surface inflow.

Salt loads added by each of these inflows follow the same pattern. The loadings accounted for by previous basin outflows and imports are input data or the result of a prior calculation. The salt loadings of the "key" streams are again taken as representative and total within-basin contributions are calculated.

The model treats all "high in the basin" reservoir storage as if it were a single reservoir. The release criterion used for this storage is based on average monthly historical releases summed over all such reservoirs. The percentage of total basin inflow which is regulated through this storage is determined during the calibration process as that necessary to replicate the historical
pattern of active storage levels. The model allows for a single upward modification of a particular month's release during months when attempted irrigation diversions exceed available mainstem flow.

Water consumed for the purpose of industrial and agricultural activity is estimated using the total gross outputs generated by the input-output model. In the case of agriculture, only withdrawals are thus calculated, while consumptive use and return flows are calculated in the body of the model. For municipal and industrial activities, consumptive use, withdrawals, and salt loadings in return flows are derived from coefficients input to the model.

The model allows for two types of transmission loss from water diverted for irrigation: direct runoff into drains and deep percolation directly from the main irrigation canals.

The concept of multiple diversions is important in basins with a large amount of agricultural activity. A parameter is used to indicate the number of times water can be withdrawn during one month. This allows for rediversion of the same water further down stream. This parameter can be estimated a priori or through the calibration process.

Deep percolation is the movement of water out of the plant root zone into the underlying aquifers. It is assumed in the model that deep percolation occurs only when the root zone soil moisture stock exceeds the capacity of the root zone to hold water, the saturation point.

The salts which are carried or picked up by irrigation water enter the root zone and move downward through the solid profile. Since deep percolation usually does not occur every month and since water is lost through evapotranspiration, a gradual increase in the TDS concentration in the root zone usually occurs during much of the year. When deep percolation occurs, the model assumes that salt is removed from the root zone in proportion to the percentage of soil moisture which is moved into the groundwater stock.
Several methods of estimating the potential evapotranspiration rate are available. The method adopted for this model is a modification of the Blaney and Criddle method. A weighting coefficient for each crop is derived month by month during the growing season from growth stage curves found in Hyatt et al., and the Soil Conservation Service Irrigation Water Requirements. An aggregate coefficient is then calculated from the above and from crop outputs obtained from the I/O analysis, which in turn is used as a scaling factor for the potential evapotranspiration rate equation.

Groundwater refers to water present in the aquifers underlying any particular basin. Much of the water which moves down into the valley bottom as groundwater reappears as surface water base flow in the mainstem channel. Groundwater flow can originate in previous basin groundwater outflow, from recharge from high mountain tributaries, and from deep percolation of precipitation, snowmelt, and irrigation water. While an insignificant amount of water is pumped from groundwater in the study area used to develop this model, provisions for agricultural, municipal, and industrial pumping have been included.

The final segment of the model is the handling of reservoirs at the end of the basin. The total amount of water available for outflow or as end-of-basin reservoir inflow consists of the total basin inflow, plus valley bottom precipitation, less net system losses due to evapotranspiration or municipal and industrial consumptive use. If no reservoir is present at the end of the basin, it is likely that a portion of the total outflow of the basin will leave as groundwater flow. The model allows for lagging this flow. If a reservoir is present, it is assumed that there will be no groundwater outflow from the basin.

The operating rules of many end-of-basin reservoirs within the Colorado River Basin are complicated by the use of these

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reservoirs for power generation. The actual release pattern of any one power generation reservoir is a function of many things. After analyzing the release records of several power generation facilities, it was determined that a close approximation to the historical record of releases could be generated as a function of the previous months' reservoir levels alone. This is the method adopted for this model.

In most of the basins within the study area used to develop this model, transbasin exports are diverted from flows high in the tributary headwaters. In some basins, however, significant exports of water occur just prior to the basin outflow point. The model permits deducting the required export water from the total water available for output.

This model was developed as a generalized hydro-salinity model designed to be applicable to any geographic area. The wide variety of sub-basins studied during the development phase of the model has helped to ensure the generalized character and flexibility of the model.

**Air Pollutant Generation and Dispersion Model (APGDM)**

The impacts on air quality of increased coal production are both primary and secondary in nature. Primary impacts include, for example, emissions from drag lines, coal trucks, trains, and refineries. Secondary impacts can result from increased population, for example, which results directly and indirectly from increased mining. The input/output framework is an orderly method by which primary, secondary, and higher order impacts on air quality may be quantified without inadvertently overlooking any level of interaction among the activities in the impact area.

Air quality impacts are characterized in two ways in the APGDM. The first is the generation rate (Tons/yr) or residuals
coefficient (Tons/$\text{TGO}$) for five major airborne species for each I/O sector. The five species are Total Suspended Particulates--TSP (d ≤ 20μ), Sulfur Dioxide--$\text{SO}_2$, Oxides of Nitrogen--$\text{NO}_x$, Carbon Monoxide--$\text{CO}$, and Total Unburned Hydrocarbons--$\text{THC}$ (non-methane).

The second method for quantifying air quality impacts is by the surface level concentrations (micrograms per cubic meter - $\mu\text{g/m}^3$) of the above contaminants.

The APGDM is linked to the economic model by its output--"Total Gross Outputs" (TGO's)--and to the social model by its outputs of population levels, number of dwelling units, employment, etc. These two linkages are used to drive the APGDM in its predictive mode wherein the impact of increased economic activity and population upon air quality are calculated. Any scenario of increased or decreased activity can be simulated by this set of integrated models.

The Residuals Generation Model

In order to transform industrial process rates and fuel consumption rates for point and area sources from the source files into residuals generation rates an emission factor is applied to each. Process emission factors are unique to each process while fuel emission factors vary depending not only upon the fuel type, but on the geographic location of the fuel source and type of combustion equipment as well. Emission factors have units such as grams of particulates per ton of coal, grams of CO per thousand cubic foot of natural gas, etc. This approximate emission factor approach is required since emissions for specific sources are not known.

Emission factors used in the models are based upon those in the Compilation of Air Pollutant Emission Factors (EPA, 1973). For processes and sources not included therein reference is made to other data sources (Perry, 1963; Kreichelt, 1966; and Colorado Department of Health, 1969). Certain modifications to the EPA
emission factors are required because of the high average altitude of some regions of the West. Emission factors used in the model also include the effects of any abatement devices used at the source. For example, the lower pollution generation level resulting from emission controls on autos on highways or from fabric dust collectors at cement plants must be included in the emission factors used in the model.

One major purpose of the residuals model is to enable the user to predict the generation of pollutants for years other than the base year in which the emission inventory was conducted. Data from I/O models are used for this purpose. I/O models predict gross outputs ($) for each of the sectors in a region. For purpose of APGDM implementation, the growth or recession experienced by each SIC category, i.e., its TGO change, is assumed to apply uniformly to all industries which comprise that category. That is, the subset contained in the emission inventory of the set of all firms in each SIC sector is assumed to be a microcosm of the sector. It is then a simple matter to scale residuals generation rates.

The Pollutant Dispersion Model

The Gaussian model

Experimental data describing the distribution of concentration of pollutants in plumes from stacks show that these plumes exhibit a statistically strong tendency toward a Gaussian or normal distribution of concentration in any downwind cross section. The Gaussian model has also been shown to be valid over downwind travel distances of several hundred kilometers (Koch, 1971), and is used to describe concentration fields of airborne pollutants which issue from stacks and undergo no gravitational settling. The APGDM is a steady/state model so the concentration field of each pollutant is defined for a time period during which average transport and dispersion characteristics of the earth's atmosphere are assumed to be unchanged.
The ground level concentration of a given species along the centerline of the plume (assumed straight in the plan view) is given by (Turner, 1970):

\[
\chi = \frac{Q}{2\pi u \sigma_y \sigma_z} \left\{ \exp \left[ -\frac{1}{2} \left( \frac{z-h}{\sigma_z} \right)^2 \right] + \exp \left[ -\frac{1}{2} \left( \frac{z+h}{\sigma_z} \right)^2 \right] \right\} \times 10^6
\]

(Equation 1)

where

- \( \chi \) = concentration (micrograms/cu. meter)
- \( Q \) = residual emission rate (grams/second)
- \( u \) = magnitude of the mean wind velocity (meters/second) (assumed uniform)
- \( \sigma_y, \sigma_z \) = crosswind and vertical dispersion parameters (meters) depending on atmospheric stability, insolation, and downwind distance \( x \).
- \( h \) = plume center line height (meters) which also depends on \( x \).
- \( z \) = vertical coordinate measured from the source base elevation, e.g., power plant stack base.

The coordinate axes \((x, y, z)\) are Cartesian coordinates with origin below the point source, \( x \) axis along the mean wind direction, \( z \) axis vertical, and \( y \) axis located to give a right-handed system. Modifications of this equation to account for the effects of a stable layer aloft and various pollutant decomposition processes in the atmosphere are described later.

There are five parameters in the Gaussian model, each of which is discussed below.

**Diffusion Parameters** \( \sigma_y, \sigma_z \)

It is assumed that the plume spread parameters \( \sigma_y \) and \( \sigma_z \) depend only on the stability class and the downwind distance \( x \). Many empirical functions and tabulations have been proposed to represent \( \sigma_y \) and \( \sigma_z \). The parameters of Pasquill which have been presented
graphically by Turner are used in the present model (Pasquill, 1961; Turner, 1970). A curve fit of the form

\[
\log_{10} \sigma = (A + B \log_{10} 10x + C(\log_{10} 10x)^2)
\]  

(Equation 2)

has been made of \( \sigma_y \) and \( \sigma_z \) in each of the six Pasquill stability classes. The variables \( \sigma_y \) and \( \sigma_z \) have units of meters and \( x \) has units of kilometers. Pasquill's six stability classes A through F are described in Table 2.

**Table 2. Pasquill stability classes.**

<table>
<thead>
<tr>
<th>Surface Wind Speed (at 10 m), m sec(^{-1})</th>
<th>Day</th>
<th>Night</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Strong</td>
<td>Moderate</td>
</tr>
<tr>
<td>&lt; 2</td>
<td>A</td>
<td>A-B</td>
</tr>
<tr>
<td>2-3</td>
<td>A-B</td>
<td>B</td>
</tr>
<tr>
<td>3-5</td>
<td>B</td>
<td>B-C</td>
</tr>
<tr>
<td>5-6</td>
<td>C</td>
<td>C-D</td>
</tr>
<tr>
<td>&gt; 6</td>
<td>C</td>
<td>D</td>
</tr>
</tbody>
</table>

The neutral class, D, is assumed for overcast conditions during day or night.

**Effective Plume Height (h)**

Emissions leaving large industrial stacks are generally fast moving and hot. As a result, they exit from the stack with upward momentum and considerable buoyancy. As the plume interacts with the atmosphere its momentum and buoyancy are reduced until the plume usually levels off some distance downwind. The vertical distance from the top of the stack to the center line of the plume is termed the plume rise \( \Delta h \). Therefore the effective height \( h \) for dispersion calculations is
\[
    h = h_s + \Delta h
\]
where \( h_s \) is the physical stack height.

Many empirical formulas exist for determining plume rise. G. Briggs has examined these formulas and compared their results with empirical observations. His recommended correlations are used in the present model (Briggs, 1969).

**Average Wind Speed and Direction**

The model assumes a quasi-steady state and hence the existence of appropriate time averages of wind speed and direction. Historical wind data in the form of wind roses are available from the National Climatic Center (NCC) for many reporting stations in the West. The reporting stations are generally airports located outside the urban areas for which the data are to be used. This location discrepancy plus the terrain differences between cities and airports introduce unavoidable errors into the model.

A wind rose is a graphical or tabular representation of how frequently wind of a given magnitude blows from a given compass direction near the surface. No vertical variation of wind speed or direction is considered in the model. The NCC data for each reporting station use a different format with different wind speed classes. A data preprocessing program is therefore required to convert the multi-farious formats into one format usable by the dispersion model. The following wind speed classes and directions are used in the model (see Table 2).

Eight wind speed directions have been selected instead of 16 since 16-point roses are not available for all stations and the computer cost of dispersion modeling is reduced by a factor of 2 for eight wind directions. The wind rose data are also aggregated to quarterly time periods: January-March, April-June, July-September, October-December. There are then four seasonal wind roses for each location.
for each stability class. Most are based upon 10 years of historical data.

<table>
<thead>
<tr>
<th>Wind Speed Class</th>
<th>Wind Speed Range (Miles/hour)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1-3</td>
</tr>
<tr>
<td>2</td>
<td>4-7</td>
</tr>
<tr>
<td>3</td>
<td>8-12</td>
</tr>
<tr>
<td>4</td>
<td>13-17</td>
</tr>
<tr>
<td>5</td>
<td>18-24</td>
</tr>
<tr>
<td>6</td>
<td>25-40</td>
</tr>
</tbody>
</table>

Wind Directions (8): N, NE, E, SE, S, SW, W, NW

It is in the wind rose data that all effects of terrain upon wind flow patterns are assumed to be contained. The terrain is assumed to shape the wind rose in mountainous areas. This is recognized as an approximation since terrain changes may cause local eddy formation which may enhance dispersion or have other effects. The Gaussian model does not reflect subtleties of this nature. No known model is able to quantify mountain valley flows satisfactorily at this time. The model predicts the concentration levels at the surface taking into account plume impact on terrain protuberances. That is, if the terrain downwind from the source rises toward the plume trajectory then the surface level concentration is greater than that which would be predicted for a plane passing through the source base level.

Mixing Layers and Atmospheric Pollutant Removal

In order to make the Gaussian model somewhat more representative of actual pollution dispersal phenomena two modifications may be introduced. A stable layer aloft through which pollution does not pass...
in appreciable quantities is a very common phenomena in mountainous areas of the West. The existence of an inversion markedly reduces vertical mixing in the atmospheric surface layer. Such mixing ceilings may vary from 100 meters at night to 1500 meters during daylight hours and also vary widely seasonally. This short-term variation is on a time scale much smaller than the model time scale so that use of an average inversion height \( L \) would be somewhat meaningless as would the use of one average Pasquill stability class for an entire quarter. If the user selects the mixing ceiling computational option, the results are meaningful only in that they represent a dispersion situation which would exist if average wind and temperature co-existed with a fixed inversion height for a substantial period of time.

Mixing depth data in the Upper Main Stem are quite sparse. If the user makes computer runs with the mixing layer depth as an option, a considerable amount of judgment needs to be exercised in using data from one site for another nearby. A study in Utah showed that dispersion phenomena in one mountain valley differed significantly from those in the neighboring mountain valley (Reynolds, 1970).

A second optional refinement to the Gaussian equation is included in the model. If an exponential decomposition rate is assumed for \( \text{SO}_2 \) or \( \text{NO}_x \) dispersing in the atmosphere, Equation 1 may be modified by multiplying by the factor \( \exp\left(-\frac{0.693x}{\frac{\tau}{2}u}\right) \) where \( \frac{\tau}{2} \) is the pollutant half life (seconds), \( x/u \) is the travel time from stack exit (seconds), and 0.693 is the natural logarithm of 2. Half-lives vary from hours to days depending upon relative humidity, insolation, etc. Thus \( \frac{\tau}{2} \) must be specified by the user based upon the particular combination of the factors he wishes to model.

**Shorter Time Scale Simulations**

The model as described heretofore is a long time scale model. If the user wishes to simulate a shorter time scale dispersion situation
(e.g., 8 hours) it can be done with the long time scale model if certain changes are made and if it is done with caution. In summary, the user must ascertain that the following model inputs are all compatible and are specific to the short time scale:

1) Source emission rates - diurnal variation, if any.
2) Wind rose - must have only one non-zero entry corresponding to short term wind magnitude and direction (table entry value is 1.0).
3) Mixing depth and stability class - compatible with wind rose and insolation of season.
4) Plume height - must be below mixing depth ceiling.
5) Ambient dry bulb temperature.

Some Examples of APGDM Output

The outputs from the APGDM are of two forms. The first is a tabulation of concentration for five pollutants at selected distances from the source. An example of this output is shown in Figure 3.

The second, more useful form of output is computer-generated isopleth maps which show the distribution of pollutants, at ground level, around a source. Figure 4 shows the expected concentrations in μg/m of total suspended particulates (TSP) and SO₂ around a power plant proposed for location above the city of Delta, Colorado. Regions lying below the plant along the Gunnison River are not impacted. However, more elevated regions to the northeast and southwest are impacted as shown.

Figure 5 shows computer predictions of long term average pollutant concentration of TSP and SO₂ around the Four Corners Power Plants for 1970. The diurnal nature of the winds in the San Juan Valley is indicated by the two concentration peaks upstream and downstream of the plant.

Output from the APGDM also includes residuals generation levels to TGO on a sector by sector basis. Table 3 is an example of residuals coefficients for a small region within the Colorado River Basin.
Figure 3. Typical page of computer output.
Figure 4. \( \text{SO}_2 \) increment isopleth map for first quarter 1980.
Figure 5. Suspended particulate isopleth map--first quarter 1970.
Table 3. Direct residuals coefficients—North Fork Sub-Basin.

<table>
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<th></th>
<th></th>
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<td>4. Food/Field</td>
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<td>.00231</td>
<td>.00312</td>
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<td>.00140</td>
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<td>NOₓ</td>
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<td>.0121</td>
<td>.0163</td>
<td>.0148</td>
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<td>6. Fruit</td>
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<td>0.0</td>
</tr>
<tr>
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<td>0.0</td>
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<td>.00258</td>
<td>.00549</td>
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<tr>
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<td>28. Electric Energy</td>
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Bibliography


Salinity in the Colorado River is of major national concern for not only has it resulted in losses to regional economy, but, in addition, high salinity levels have aggravated relations with the Republic of Mexico. Even in its virgin state, the salt load of the Colorado River in its lower reaches was about 600 to 700 ppm. However, man's development of water resources has affected both the quantity and quality of water supplies. Salinity levels in the lower reaches of the river now average 850 ppm with a predicted concentration of 1,300 ppm by the year 2000.

The sources and causes of dissolved solids within the Colorado River are of importance, for if they can be identified, strategies may be developed for effective management and control. In addition, this information would allow estimates to be made of downstream costs associated with upstream salt production, thus facilitating the development of economic trade-offs on a basin-wide level.

Recent estimates suggest the largest single man-caused source of salinity is irrigation return flow amounting to about a third of the total salt load. Natural sources as salt wells and springs plus concentration by evaporation account for another third. The remaining salt load is attributed to diffuse sources originating on immense areas of wildland watersheds.

Methods are presently available to quantify salt input from point sources. However, the same is not true for diffuse sources. The summation of salt inflows from widespread natural diffuse sources

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can result in significant mineral concentrations at tributary outlets.
In view of the present, as well as future concern for water quality, it is imperative that reliable methods to predict salt loading from diffuse sources be developed. Such information will be available in the design of effective control and management procedures.

In the spring of 1974, a study of land processes involved in diffuse salinity production was started in the Price River Basin of Utah (Figure 1). The overall objectives of the three-year study are:

1. To determine the role of overland flow on salt movement for selected land and vegetative types.
2. To determine the relative magnitude of surface erosion from overland flow for selected land and vegetative types.
3. To determine the vegetative influence on salt movement in the hydrologic cycle.

Figure 1. The Price River Basin in East-Central Utah.
4. To develop a working mathematical model which accurately predicts salt production from diffuse sources as a function of overland flow and time.

5. To determine the relative worth of selected treatments on control of salt movement in the hydrologic cycle.

This paper discusses progress to 30 June 1975. The data were obtained during the initial field season with emphasis placed on objectives 1, 2, and 3.

The Study Area

The Price River Basin was selected for the study area. It is one of the major sources of salinity to the Colorado River, and in addition, has both climate and vegetation typical of the Upper Colorado River Basin (5).

The Price River Basin encompasses nearly 1900 square miles (mi^2) and is located principally in Carbon and Emery Counties of east-central Utah. The altitudinal range varies from about 10,500 feet (ft) in the headwaters to nearly 4,200 ft at the confluence of the Price and Green Rivers.

Precipitation varies widely within the basin. Altitude, topography, and geographic location relative to the predominant west-to-east storm track are factors that effect the amount of precipitation (4). In general, annual precipitation in the headwaters area ranges between 20 and 25 inches (in), while the lower portion of the basin receives about 8 in. Nearly 65 percent of the total precipitation occurs as snow during the period of late October to early May. Of the total annual precipitation, about 50 percent falls on the upper 30 percent of the basin, while 70 percent falls on areas having altitudes greater than 7000 ft (4). Consequently, approximately 70 percent of the basin may be classified as semi-arid.
The Price River has a length of nearly 100 miles and flows in a south-easterly direction. The majority of flow originates in the upper third of the watershed. The streamflow is greatly affected by irrigation use in the central portion of the basin (4).

Land use is primarily the raising of cattle and sheep, while about 2 percent of the area is irrigated and produces sugar beets, hay, and grain. The major industry of the area is underground coal mining.

Geology of the Basin

Geographically, the Price River Basin contains portions of the Uinta Basin, the High Plateaus, and Canyon Land section of the Colorado Plateau province (2). Although the geology of the area is complex, it has been well documented (7, 10). Figure 2 illustrates the major stratigraphic units present in the basin.

The Cedar Mountain formation is located in the south-eastern portion of the basin and may be thought of as a pivot point with the other geologic formations forming a semi-circular pattern around it. The strata are of sedimentary origin, dipping 10 degrees away from the Cedar Mountain formation, with Tertiary period deposits comprising the upper layers and Cretaceous period deposits the base. Quarternary gravel capped pediment surfaces, which give rise to prominent benches, along with alluvial deposits are apparent throughout much of the basin.

The upper portions of the watershed are comprised of a series of cliff forming limestone and sandstone strata (Green River formation through Star Point Sandstone, Figure 2). Surface waters that drain through these strata are considered high quality with the predominant water type being calcium-bicarbonate (4).

The central and lower portions of the basin are comprised predominantly of marine shale deposits intermixed with sandstone lenses or fingers and non-marine beds (Mancos Shale through the Cedar Mountain formation, Figure 2). The Mancos Shale is a marine deposit covering nearly 25 percent of the area and accounting for 61 percent of the
altitudinal range (3800 ft) in the basin. It is a drab, slightly bluish-gray shale intermixed with thin lenses of calcareous sandstone, limestone, and a few concretionary beds (7). Traditionally, the Mancos Shale has been considered to be the prime source of salt in the basin.

The Mancos Shale is divided into three distinct members, Masuk, Blue Gate, and Tununk, which are separated by identifiable sandstone fingers. As a result of the 10 degree dip of the strata, each member of the Mancos is exposed in the basin. The Masuk member is the youngest and is separated from the Blue Gate member below by the Emery and Garley Canyon sandstones. The Masuk forms a relatively large band above the Blue Gate and accounts for 6 percent of the basin area.

![Diagram of geologic units](image)

**Figure 2.** Major geologic stratigraphic units present in the Price River Basin.
The Blue Gate member is the most extensive, extending 2150 ft vertically and covering nearly 17 percent of the basin. It contains a high concentration of evaporites (10), as well as gypsum (CaSO₄·2H₂O).

Below the Blue Gate member, separated by the Ferron sandstone, is the Tununk, which is the oldest member of the Mancos formation in the basin. It is a very narrow band, generally less than one mile in width and accounts for only 2 percent of the total basin area.

The remainder of the basin is composed of miscellaneous geologic types, mostly of non-marine origin, and consequently contribute relatively few salts into the drainage water.

**Research Design**

**Surface runoff and soil studies**

The infiltrometer technique was selected to study the process of overland flow and its relation to salt transport. The basic design and use of the infiltrometer is discussed in detail by Dortignac (1).

Ideally, site selection should have been on the various defined soil series present in the basin. However, a review of the literature revealed that the soil survey carried out by the Soil Conservation Service (8) was limited to only a narrow band of agricultural land running north-south through the central basin. As a result, it was decided to identify the various geologic types in the basin, which might serve as parent material for the overlying soil.

Criterion for site selection was as follows:

1. Sample "predominate" geologic types or soils derived from them.
2. Be accessible by road and located on land managed by the U.S. government.
3. Have a slope of approximately 10 percent.

The identification and extent of basin coverage by each geologic type was determined using a standard USGS geologic map (10). The
predominant geologic types sampled, which total nearly 83 percent of the basin area, are listed in Table 1, along with their respective site numbers and a brief description of each. The remaining geologic types present in the basin are of non-marine origin and probably do not contribute substantially to the salt load of the Price River. Specific sites were selected to be as representative as possible.

Since the Mancos Shale has traditionally been considered to be a prime source of salt in the basin, it was intensively sampled as illustrated in Table 1. The Mancos shale sites were selected in such a manner as to assess (a) its potential as a prime source of salt, (b) if members within the Mancos varied in their degree of salt release, and (c) if the Blue Gate member differed within itself as a source of salinity. It should be noted that the USGS (10) only mapped the Mancos shale with respect to its various members south of the Price River. Much of the Mancos north of the Price River is mapped as Mancos Undivided. Sites 13, 14, and 15 were selected to examine the variation of salt production within the Mancos Undivided.

At each field site the following activities were carried out:

1. Six plots (1 ft x 2.5 ft each) were selected and subjected to a simulated rainfall of similar intensity produced by a Rocky Mountain Infiltrometer for a period of 28 minutes. Distilled water was used in all cases.

2. The amount of rainfall and runoff was measured at the 0-3 minute interval and at 5-minute intervals thereafter through 28 minutes.

3. Electrical conductivity readings were taken of runoff samples collected over each interval.

4. A composite sample (1 liter) was then created by mixing all the interval samples.

5. A vegetation survey was taken of each plot. Each survey point was recorded either as bare ground, litter, grass, shrub or forb. If applicable, genus and species were also recorded.
Table 1. Site numbers and a brief description of each geologic type sampled.

<table>
<thead>
<tr>
<th>Geologic Type (Identification Code)</th>
<th>Site Number</th>
<th>Characteristics</th>
</tr>
</thead>
<tbody>
<tr>
<td>A. Mancos Shale Members</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1. Masuk (M)</td>
<td>1, 2, and 3</td>
<td>Gray, non-resistant marine shale</td>
</tr>
<tr>
<td>2. Blue Gate (BG)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>a. Upper BG (UBG)</td>
<td>4 and 5</td>
<td>Light gray, calcareous marine shale</td>
</tr>
<tr>
<td>b. Middle BG (MBG)</td>
<td>6 and 7</td>
<td></td>
</tr>
<tr>
<td>c. Lower BG (LBG)</td>
<td>8 and 9</td>
<td></td>
</tr>
<tr>
<td>3. Tununk (T)</td>
<td>10, 11, and 12</td>
<td>Gray marine siltstone and claystone</td>
</tr>
<tr>
<td>4. Mancos Undivided (MUD)</td>
<td>13, 14 and 15</td>
<td>Light-gray, non-resistant, marine shale</td>
</tr>
<tr>
<td>B. Cedar Mountain (CM)</td>
<td>16 and 17</td>
<td>Nodular shale with fluvial sandstone beds</td>
</tr>
<tr>
<td>C. Alluvial Deposits (AD)</td>
<td>18 and 19</td>
<td>Young alluvial deposits along active streams</td>
</tr>
<tr>
<td>D. Gravel Caps (GC)</td>
<td>20 and 21</td>
<td>Mainly terraces and pediments undergoing erosion; may not be associated with active streams</td>
</tr>
<tr>
<td>E. Black Hawk Fm. (BH)</td>
<td>22</td>
<td>Sandstone, mudstone, shale and coal</td>
</tr>
<tr>
<td>F. Price River (PR)</td>
<td>23</td>
<td>Interbedded sandstone and mudstone</td>
</tr>
<tr>
<td>G. North Horn Fm. (NH)</td>
<td>24</td>
<td>Fluvial sandstone and mudstone</td>
</tr>
<tr>
<td>H. Colton Fm. (C)</td>
<td>25</td>
<td>Fluvial red beds with channel sandstone</td>
</tr>
<tr>
<td>I. Green River Fm. (GR)</td>
<td>26</td>
<td>Lacustrine shale and siltstone</td>
</tr>
</tbody>
</table>
6. A soil sample was collected in the 0-1 in, 1-6 in, and 6-12 in depths at each site.

The composite runoff sample was analyzed for primary cations ($\text{Na}^+, \text{K}^+, \text{Ca}^{+2}, \text{Mg}^{+2}$), anions ($\text{Cl}^-, \text{CO}_3^{-2}, \text{HCO}_3^-, \text{SO}_4^{-2}$), total solids, pH, and EC of the clear solution (after setting 24 hours). Laboratory analysis followed the procedures outlined in Standard Methods (6).

The total solids analysis includes suspending particles as well as dissolved minerals.

The soil samples were taken to the laboratory and 1:1 soil-water extracts and saturation extracts prepared. Chemical analysis was performed on the respective extracts.

**Vegetation washing studies**

**Field studies**

1. Salt release with time for various plants was examined in the field. One gram of plant material was clipped and placed in a beaker containing a known quantity of water. Salt release was recorded at 1 minute intervals using an EC meter. Criterion for termination of the run was a constant EC value for an extended period of time or 30 minutes, whichever came first.

**Laboratory studies**

1. Maximum ionic concentrations were determined by grinding 50 gm samples (oven dry) of plant litter and then mixing them with a known volume of distilled water. The mixture was allowed to set 24 hours, at which time it was filtered. The filtrate was then analyzed for the major chemical parameters.

2. The amount of nutrients washed off by a high intensity rain (3 in/hr) was determined using a rainfall simulator developed by Meeuwig (3). The experiment was run using 50 gm (oven dried) of litter for a duration of 1 hour.
Results

The results of the hydrologic data collected from the field sites is presented in Table 2. These values represent a composite average of the total precipitation and runoff for six plots at each site for the 28 minute event. No data are shown for site 3 because of its unsuitability for setting up the infiltrometer.

Table 3 summarizes the water quality data obtained from the analysis of samples representing a mixture of the total runoff occurring during the 28 minute period. The values have been corrected to the control and represent a composite average of the six plots at each site. Along with the major cations and anions, electrical conductivity taken in the field (ECF) and laboratory (ECL), hydrogen ion activity (pH), total dissolved solids (TDS) and total solids (TS) are given. No data were collected from site 3.

Table 4 presents some of the chemical analysis of the 1:1 soil extracts, while Table 5 summarizes the analysis of the saturated soil

Table 2. Total precipitation (P) and runoff (Q) data for each site.

<table>
<thead>
<tr>
<th>Site (Geologic code)</th>
<th>P (in)</th>
<th>Q (in)</th>
<th>Site (Geologic code)</th>
<th>P (in)</th>
<th>Q (in)</th>
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<td>0.96</td>
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<td>--</td>
<td>16 (CM)</td>
<td>1.20</td>
<td>0.47</td>
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<td>4 (UBG)</td>
<td>1.33</td>
<td>0.35</td>
<td>17 (CM)</td>
<td>1.30</td>
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<td>5 (UBG)</td>
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<td>0.84</td>
<td>18 (AD)</td>
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</tr>
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<td>6 (MBG)</td>
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<td>0.62</td>
<td>19 (AD)</td>
<td>1.17</td>
<td>0.84</td>
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<td>7 (MBG)</td>
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<td>0.50</td>
<td>20 (GC)</td>
<td>1.35</td>
<td>0.40</td>
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<td>0.40</td>
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<td>25 (C)</td>
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<td>13 (MUD)</td>
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<td>0.73</td>
<td>26 (GR)</td>
<td>1.13</td>
<td>0.58</td>
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Table 3. Summary of water quality data, corrected to the control. All values represented a composite average of plots one through six at each site.

<table>
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<th>Site (Geologic Code)</th>
<th>Na⁺ meq/l</th>
<th>K⁺ meq/l</th>
<th>Ca²⁺ meq/l</th>
<th>Mg²⁺ meq/l</th>
<th>CO₃⁻ meq/l</th>
<th>HCO₃⁻ meq/l</th>
<th>SO₄⁻² meq/l</th>
<th>Cl⁻ meq/l</th>
<th>ECF µhos/cm</th>
<th>TDS mg/l</th>
<th>TS g/l</th>
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<td>0.1</td>
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<td>4 (UBG)</td>
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<td>0.1</td>
<td>0.6</td>
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<td>0.0</td>
<td>0.9</td>
<td>0.0</td>
<td>0.0</td>
<td>50</td>
<td>52</td>
<td>2.52</td>
</tr>
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<td>0.3</td>
<td>0.2</td>
<td>0.1</td>
<td>0.2</td>
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<td>0.0</td>
<td>26</td>
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<td>0.5</td>
<td>0.3</td>
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<td>0.0</td>
<td>0.2</td>
<td>67</td>
<td>39</td>
<td>1.52</td>
<td></td>
</tr>
<tr>
<td>12 (T)</td>
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<td>0.1</td>
<td>0.6</td>
<td>0.3</td>
<td>0.7</td>
<td>6.8</td>
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<td>743</td>
<td>484</td>
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<td>0.1</td>
<td>1.1</td>
<td>0.1</td>
<td>1.0</td>
<td>0.2</td>
<td>0.1</td>
<td>60</td>
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<tr>
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<td>56</td>
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<tr>
<td>17 (CM)</td>
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<td>0.1</td>
<td>0.2</td>
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<td>0.0</td>
<td>0.1</td>
<td>36</td>
<td>30</td>
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<tr>
<td>18 (AD)</td>
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Table 4. Chemical analysis of some of the 1:1 soil extracts for the 0-1 in. depth from surface flow study sites in the Price River Basin.

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<th>Site</th>
<th>Geologic Code</th>
<th>Na$^+$ meq/l</th>
<th>K$^+$ meq/l</th>
<th>Ca$^{2+}$ meq/l</th>
<th>Mg$^{2+}$ meq/l</th>
<th>HCO$_3^-$ meq/l</th>
<th>SO$_4^{2-}$ meq/l</th>
<th>Cl$^-$ meq/l</th>
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<th>Ca(^{2+}) meq/1</th>
<th>Mg(^{2+}) meq/1</th>
<th>HCO(_3) meq/1</th>
<th>SO(_4^{2-}) meq/1</th>
<th>Cl(^-) meq/1</th>
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</table>
extracts. Although the 1-6 in. and 6-12 in. depths were analyzed, only the results of the 0-1 in. depth are presented. This is the layer most active in salt release to overland flow during the type of events being considered in this paper.

Table 6 lists the results of preliminary washing studies carried out in the field. The data represent the quantity of salt (mg) released in a known volume of distilled water as a function of time per gram (dry weight) of plant material. Table 7 represents maximum ionic concentrations measured in leachate from litter from various plant species. Table 8 compares the amount of nutrient washed off by simulated high intensity rain (3 in/hr), 1 hour duration, from 50 gm litter and the maximum possible salt determined. Table 9 shows the salt removal from 50 gms of dry litter under 3 in/hr rainfall in terms of the EC of the leachate.

Discussion

Surface hydrology

Average precipitation intensities applied to each site ranged from 2.2 in/hr to 4.17 in/hr with an arithmetic mean of 2.8 in/hr and a standard deviation of 0.5 in/hr. Ideally, when a mechanical device such as the infiltrometer is employed, variation in precipitation intensity would be expected to be much lower. However, although many of the variables responsible for such variation have been eliminated by using the infiltrometer, some inherently persist. The primary factor responsible for the variation in these data was wind. Even though a wind screen surrounding the plots on three sides was used, gusts occurred that visibly affected the rainfall distribution.

To examine the relation between geologic type and hydrology, SCS curve numbers (CN), also referred to as hydrologic soil-complex numbers, and the ratio of runoff to precipitation (Q/P) were computed (Table 10). Although a definite relation between CN and geologic type was expected, no distinct separation between geologic types was observed.
Table 6. Quantity of salt (mg) released in a known volume of distilled water as a function of time per one gram (dry weight) of plant material.

<table>
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<th>Time (min.)</th>
<th>Artemisia tridentata</th>
<th>Sarcobatus vermiculatus</th>
<th>Atriplex corrigata</th>
<th>Atriplex gardneri</th>
<th>Atriplex canescens</th>
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Table 6. Continued.

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<th>Juniperus osteosperma</th>
<th>Pinus edulis</th>
<th>Ephedra spp.</th>
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<td>Rabbit brush</td>
<td>Juniper</td>
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Table 7. Maximum ionic concentrations in litter leachate per 50 grams dry weight of litter from various plant species.\(^a\)

<table>
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<th>Species</th>
<th>(\mu)hos/cm</th>
<th>mg/l</th>
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<td></td>
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<td>Na(^+)</td>
</tr>
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<td>1735</td>
<td>310</td>
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<td>747</td>
<td>7</td>
</tr>
<tr>
<td>Rabbitbrush</td>
<td>868</td>
<td>12</td>
</tr>
<tr>
<td>Hadscale</td>
<td>2208</td>
<td>334</td>
</tr>
<tr>
<td>Russian thistle</td>
<td>762</td>
<td>7</td>
</tr>
<tr>
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<td>2</td>
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<tr>
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<td>2</td>
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<tr>
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<td>3404</td>
<td>94</td>
</tr>
<tr>
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<td>6809</td>
<td>1389</td>
</tr>
<tr>
<td>Harder saltbush</td>
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<td>904</td>
</tr>
<tr>
<td>Juniper</td>
<td>593</td>
<td>-</td>
</tr>
<tr>
<td>Halogeton</td>
<td>5222</td>
<td>1325</td>
</tr>
<tr>
<td>Redian ricegrass</td>
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</table>

\(^a\)Determined by grinding 50 gm samples (oven dry) of plant litter and then mixing with known volume of distilled water (1230 ml), allowing the mixture to set for 24 hours, filtering, and then analyzing.

\(^b\)See Table 6 for genus and species of some of the plants listed here.

\(^c\)Russian thistle = *Salsola kali*, Salt cedar = *Tamarix gallica*, Halogeton = *Halogeton lomeratus*, Indian ricegrass = *Oryzopsis hymenoides*.

\(^d\)ppm = 640 \(\times\) (EC \(\times\) 10\(^3\)) which is about the same as total concentration of ions for each species.
Table 8. Comparison of amount of nutrient washed off by simulated high intensity rain (3 in/hr), one hour duration, from 50 gm litter and the maximum possible salt determined. *

<table>
<thead>
<tr>
<th>Plant Species</th>
<th>Nutrient Substances Concentration (mg/l)</th>
<th>Na</th>
<th>K</th>
<th>Ca</th>
<th>Mg</th>
<th>SO₄</th>
<th>HCO₃</th>
<th>CO₃</th>
<th>Cl</th>
</tr>
</thead>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Greasewood</td>
<td>Maximum possible</td>
<td>310</td>
<td>55</td>
<td>22</td>
<td>22</td>
<td>29</td>
<td>750</td>
<td>0</td>
<td>43</td>
</tr>
<tr>
<td></td>
<td>Washed off by heavy rain</td>
<td>264</td>
<td>43</td>
<td>4</td>
<td>6</td>
<td>49</td>
<td>305</td>
<td>0</td>
<td>49</td>
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<tr>
<td>Big sagebrush</td>
<td>Maximum possible</td>
<td>7</td>
<td>55</td>
<td>76</td>
<td>18</td>
<td>5</td>
<td>274</td>
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</tr>
<tr>
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<td>27</td>
<td>14</td>
<td>6</td>
<td>Trace</td>
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<td>0</td>
<td>14</td>
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<td>22</td>
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<td>0</td>
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<td>35</td>
<td>10</td>
<td>4</td>
<td>Trace</td>
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<td>0</td>
<td>14</td>
</tr>
<tr>
<td>Shadscale</td>
<td>Maximum possible</td>
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<td>98</td>
<td>3</td>
<td>29</td>
<td>69</td>
<td>573</td>
<td>0</td>
<td>227</td>
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<td>39</td>
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<td>3</td>
<td>126</td>
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<td>NA</td>
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*Note: The values in this table (nutrient loss) can be expressed as a percent of the dry weight of litter, multiplying the amount of loss (mg/l) by the total volume of distilled water (ml) dividing by the dry weight of the litter (50 grams) considering the units.
Table 9. Periodic variation of the salt removal from the litter of different species under the high intensity rain (3 in/hr).

<table>
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<th>Time (Minutes)</th>
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<th>Big Sagebrush</th>
<th>Garden Rabbit Brush</th>
<th>Saltbush Shadscale</th>
<th>Saltbush Cedar</th>
<th>Malt Rice Saltbush</th>
<th>Halogeton Grass</th>
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*Each of the readings in this table is the mean of Ec readings from 4 different samples.*
Table 10. Hydrologic soil-complex numbers and runoff to precipitation (Q/P) ratios.

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<th>Site</th>
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<td>(GC)</td>
<td>86</td>
<td>.291</td>
</tr>
<tr>
<td>8</td>
<td>(LBG)</td>
<td>92</td>
<td>.465</td>
<td>21</td>
<td>(GC)</td>
<td>83</td>
<td>.258</td>
</tr>
<tr>
<td>9</td>
<td>(LBG)</td>
<td>94</td>
<td>.554</td>
<td>22</td>
<td>(BH)</td>
<td>82</td>
<td>.330</td>
</tr>
<tr>
<td>10</td>
<td>(T)</td>
<td>93</td>
<td>.525</td>
<td>23</td>
<td>(FR)</td>
<td>96</td>
<td>.690</td>
</tr>
<tr>
<td>11</td>
<td>(T)</td>
<td>90</td>
<td>.375</td>
<td>24</td>
<td>(NH)</td>
<td>96</td>
<td>.706</td>
</tr>
<tr>
<td>12</td>
<td>(T)</td>
<td>88</td>
<td>.281</td>
<td>25</td>
<td>(C)</td>
<td>96</td>
<td>.663</td>
</tr>
<tr>
<td>13</td>
<td>(MUD)</td>
<td>96</td>
<td>.666</td>
<td>26</td>
<td>(GR)</td>
<td>93</td>
<td>.511</td>
</tr>
</tbody>
</table>

Figure 3 illustrates the relation of runoff to precipitation (Q/P). The dashed lines delineate regions of Q/P where clusters of points seem to exist. It would be expected that the Mancos shale members, which are predominantly clay texture would have the greatest Q/P ratio. Observation of Figure 3 shows this is not the case. Points representing the Mancos shale are found in all clusters, with predominance in the Q/P range of 0.46 to 0.58; with only one non-Mancos site existing in the cluster, the Green River Formation. Two Mancos sites are present in the highest range, 0.65 to 0.72. The alluvium site 19, was expected to be the highest since it was located only a few feet from the bank of the Price River where the soil was near saturation. The reason the other non-Mancos sites are present in the highest Q/P region is considered due to compaction by grazing. These geologic types are located in the upper reaches of the watershed where the majority of precipitation occurs and consequently, high grazing activity.

Most of the Mancos sites cluster below the highest region. This is probably a function of precipitation, lack of grazing, and seasonal
temperature patterns. Most of the Mancos areas receive less than 8 in of annual precipitation, much of it occurring during the winter months. As a result, vegetation is sparse and grazing is severely limited. The freezing and thawing process which occurs throughout the winter months, coupled with the presence of salts acts to keep the soil flocculated. Consequently, the surface layer is loosely packed thus allowing a greater infiltration capacity than expected for a clay soil.

The Cedar Mountain formation (sites 16 and 17) has a consistent Q/P relation, falling in the 0.37 to 0.38 range. The Gravel Cap (sites 20 and 21), Alluvium (site 18), and Black Hawk (22) sites were expected to be in the lowest Q/P range, 0.16 to 0.32, due to their sandy texture; the presence of three Mancos type sites (1, 4, and 12) was unexpected. All three of these Mancos sites were located near sandstone fingers which probably had a definite affect on their surface textures.
Infiltration curves of the Horton form \( f = f_c + (f_o - f_c) \exp(-kt) \) were fit by a least squares procedure to all applicable infiltrometer data. The parameters of the Horton equation are defined as follows:

- \( f \) is the infiltration rate at time \( t \),
- \( f_o \) is the initial infiltration rate,
- \( f_c \) is the infiltration rate which is related to the conductivity of the soil, and
- \( k \) is the decay constant for the infiltration curve.

Average values of the infiltration parameters for each site are given in Table 11. The values of \( f_c \) ranged from 0.54 to 2.60 in/hr, and from \( f_o \) from 1.69 to 12.0 in/hr; while values of \( k \) ranged from 0.134 to 1.189 min\(^{-1}\), with conspicuous clusters at about 0.23, 0.33, and 1.18 min\(^{-1}\). The latter cluster of \( k \) values is composed primarily of non-Mancos sites.

Table 11. Results of the least square analysis of the Horton equation.

<table>
<thead>
<tr>
<th>Site (Geologic type)</th>
<th>( f_c ) (in/hr)</th>
<th>( f_o ) (in/hr)</th>
<th>( k ) (1/min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 (M)</td>
<td>5.08</td>
<td>2.60</td>
<td>0.134</td>
</tr>
<tr>
<td>2 (M)</td>
<td>7.20</td>
<td>1.26</td>
<td>0.705</td>
</tr>
<tr>
<td>3 (M)</td>
<td>--</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>4 (UBG)</td>
<td>3.95</td>
<td>2.03</td>
<td>0.376</td>
</tr>
<tr>
<td>5 (UBG)</td>
<td>11.26</td>
<td>1.13</td>
<td>0.664</td>
</tr>
<tr>
<td>6 (MBC)</td>
<td>6.07</td>
<td>1.01</td>
<td>0.246</td>
</tr>
<tr>
<td>7 (MBC)</td>
<td>3.93</td>
<td>0.65</td>
<td>0.240</td>
</tr>
<tr>
<td>8 (LBC)</td>
<td>4.34</td>
<td>0.72</td>
<td>0.196</td>
</tr>
<tr>
<td>9 (LBC)</td>
<td>3.87</td>
<td>0.58</td>
<td>0.226</td>
</tr>
<tr>
<td>10 (T)</td>
<td>6.19</td>
<td>0.73</td>
<td>0.310</td>
</tr>
<tr>
<td>11 (T)</td>
<td>5.63</td>
<td>1.15</td>
<td>0.349</td>
</tr>
<tr>
<td>12 (T)</td>
<td>1.69</td>
<td>1.02</td>
<td>0.237</td>
</tr>
<tr>
<td>13 (MUD)</td>
<td>10.04</td>
<td>0.54</td>
<td>1.005</td>
</tr>
<tr>
<td>14 (MUD)</td>
<td>5.61</td>
<td>0.69</td>
<td>0.397</td>
</tr>
<tr>
<td>15 (MUD)</td>
<td>12.02</td>
<td>0.67</td>
<td>1.145</td>
</tr>
<tr>
<td>16 (CM)</td>
<td>7.14</td>
<td>0.67</td>
<td>0.323</td>
</tr>
<tr>
<td>17 (CM)</td>
<td>8.39</td>
<td>0.94</td>
<td>0.227</td>
</tr>
<tr>
<td>18 (AD)</td>
<td>6.81</td>
<td>2.15</td>
<td>0.191</td>
</tr>
<tr>
<td>19 (AD)</td>
<td>1.97</td>
<td>0.69</td>
<td>1.044</td>
</tr>
<tr>
<td>20 (CC)</td>
<td>7.98</td>
<td>1.25</td>
<td>0.213</td>
</tr>
<tr>
<td>21 (CC)</td>
<td>9.09</td>
<td>1.98</td>
<td>0.397</td>
</tr>
<tr>
<td>22 (BH)</td>
<td>10.86</td>
<td>2.57</td>
<td>1.189</td>
</tr>
<tr>
<td>23 (PR)</td>
<td>11.51</td>
<td>0.61</td>
<td>1.179</td>
</tr>
<tr>
<td>24 (NH)</td>
<td>12.76</td>
<td>0.60</td>
<td>1.162</td>
</tr>
<tr>
<td>25 (C)</td>
<td>8.73</td>
<td>0.67</td>
<td>1.179</td>
</tr>
<tr>
<td>26 (GR)</td>
<td>11.22</td>
<td>0.97</td>
<td>1.149</td>
</tr>
</tbody>
</table>
while the other two clusters are predominantly Mancos sites. However, no distinct separation between the \(k\) clusters of the Mancos members could be made.

**Surface water chemistry**

Analysis of the surface water chemistry data allow some conclusions to be made concerning land types and their relation to surface flow salinity. Table 3 shows that some geologic types have runoff with higher TDS values than others. Price River alluvium (site 19) which was located about 3 meters from the stream and included visible salt deposits, had the highest TDS values recorded. The river channel was in the Blue Gate member of the Mancos shale. The Miller Creek alluvium (site 18) had a much lower runoff salinity. Here the site was located in an area of no visible salt crust and about 30 feet from the channel in sandy materials. It was also located in the Blue Gate member of the Mancos shale.

The surface water chemistry data show that the youngest member of Mancos shale, Masuk (sites 1 and 2) yields relatively good quality runoff water. This is related primarily to the soil developed over the Masuk. The parent material of this soil is considered to have resulted from the weathering of the overlying Star Point sandstone and Black Hawk limestone formations. The surface soil is a calcareous loamy sand and the runoff water is predominantly a calcium-bicarbonate type.

Table 3 shows that considerable variation occurs between the quality of runoff water within each division of the Blue Gate. The Upper Blue Gate (sites 4 and 5) yielded the best quality water of the three divisions. As with the Masuk member, the water is a calcium bicarbonate type. Again the runoff water reflects the nature of the overlying soil which, in this case, was derived from Emery and Garley Canyon sandstones. The soil was a calcareous silty loam. The data suggest that the Middle and Lower members of the Blue Gate formation are prime sources of salinity in the basin. Sites 6 through 9 yielded runoff
water which contained predominantly $\text{Na}^+$, $\text{Ca}^{+2}$, and $\text{SO}_4^{-2}$ ions. The soils developed on these members are considered to belong to the Chipeta-Badland Association with the dominant soils being the Chipeta silty clay loam (a typic-torriorthent). The vegetation associated with these sites included a high percentage of Mat Saltbush (*Atriplex corrugata*).

Sites 10, 11, and 12 are the analyses of the runoff from the Tununk member. Whereas sites 10 and 11 yielded reasonable quality runoff, site 12 proved to be considerably more saline. The soil analyses (Tables 4 and 5) show that the soil of site 12 was more chemically similar to that of sites 8 and 9 than the soils from sites 10 and 11, showing the overriding importance of the soil mantle that covers the underlying geologic formation. The parent material for the soil overlying the majority of the Tununk is considered to be the Ferron sandstone which results in a calcareous soil with relatively low salinity in the surface soil.

Sites 14 and 15, listed as Mancos Undivided, yielded the highest salinity runoff of any Mancos shale site. The overlying soil of these sites was essentially weathered Mancos shale. From field observations it is felt that both these sites are probably on soils similar to the lower Blue Gate member. However, the chemical analyses show them to be much higher in $\text{SO}_4^{-2}$. Site 13, also listed as Mancos Undivided, yielded relatively good quality runoff and was probably located on soils similar to the Masuk member.

Figure 4 illustrates the electrical conductivity change with time for selected geologic members. The plot for the Black Hawk type represents, in general, the trend for all the non-Mancos types. The data in Figure 4 show (a) highest concentrations of salt in surface runoff is obtained in the first 13 minutes of flow and (b) the Blue Gate and Tununk members of Mancos shale are the prime sources of salt in the basin. The data from site 12, however, gives a strong bias to the Tununk data. A reasonable estimate of the ECF for Tununk should be in the order of
60 μmhos/cm. These data of Figure 4, plus the data from Table 3, confirm the suspicion that Mancos shale is a major source of diffuse salinity in the basin.

Linear regression analysis was carried out on several pertinent parameters for the Mancos shale (Table 12). The Mancos shale has a high concentration of evaporites, along with CaSO$_4$·2H$_2$O. Consequently, the good relation between ECF and Ca$^{+2}$ and SO$_4^{-2}$ were to be expected. Also noted was the expected correlation between ECF and ECL. Similar operations relating the other chemical parameters to ECF yielded poor correlations. It was expected that the relation between runoff to precipitation (Q/P) and total runoff (Q) to total solids would have a much stronger relation.
Table 12. Linear regression analysis for selected parameters of the Mancos shale sites.

<table>
<thead>
<tr>
<th>X</th>
<th>Y</th>
<th>a</th>
<th>b</th>
<th>$r^2$</th>
<th>$s_{x\cdot y}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>ECF</td>
<td>ECL</td>
<td>99.2240</td>
<td>0.8314</td>
<td>0.98</td>
<td>45.45</td>
</tr>
<tr>
<td>ECF</td>
<td>Ca$^{2+}$</td>
<td>-0.1394</td>
<td>0.0096</td>
<td>0.98</td>
<td>0.44</td>
</tr>
<tr>
<td>ECF</td>
<td>SO$_4^{2-}$</td>
<td>-0.5350</td>
<td>0.1101</td>
<td>0.99</td>
<td>0.38</td>
</tr>
<tr>
<td>Q/P</td>
<td>TS</td>
<td>0.5966</td>
<td>9.7985</td>
<td>0.25</td>
<td>2.39</td>
</tr>
<tr>
<td>Q</td>
<td>TS</td>
<td>3.0917</td>
<td>1.3665</td>
<td>0.06</td>
<td>2.69</td>
</tr>
</tbody>
</table>

Soil chemistry studies

The soil chemistry analysis was restricted to determine which extraction method was best correlated to salt yield of the surface runoff (ECF) and the relation between soil salinity and that of the runoff. In this regard, the analysis of the 0-1 in. depth was considered to be of most value.

The EC data, in both Tables 4 and 5 for the 0-1 in. depth compared with the ECF values in Table 3 show that only a small fraction of the salt in the surface layer is removed by the overland flow resulting from the 28 minute simulated rainfall. This suggests that sediment may play an important role in salt production from the basin.

The data from Tables 4 and 5 were analyzed statistically to find if certain chemical parameters could be correlated to the ECF values. Only the analysis of the 0-1 in. depth extracted was considered in the following statistical treatment. To date, statistics have been applied only to the cation analyses and their relation to the ECF.

The data in Table 13 show the results when an attempt was made to correlate a single chemical analysis with the ECF value by linear regression. Poor correlation was found regardless of the extraction method. It was of interest to note that the electrical conductivity (EC) of the soil extract was also poorly correlated with ECF.
Table 13. Linear correlation between single chemical variables of the soil extract and the ECF value.

<table>
<thead>
<tr>
<th>Variable</th>
<th>Correlation coefficients (r)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Sat. Extracts</td>
</tr>
<tr>
<td>Mg$^{2+}$</td>
<td>.4648</td>
</tr>
<tr>
<td>Ca$^{2+}$</td>
<td>.6943</td>
</tr>
<tr>
<td>Na$^+$</td>
<td>.5435</td>
</tr>
<tr>
<td>K$^+$</td>
<td>.4355</td>
</tr>
<tr>
<td>Cl$^-$</td>
<td>----</td>
</tr>
<tr>
<td>Total cations</td>
<td>.6411</td>
</tr>
<tr>
<td>EC</td>
<td>.2670</td>
</tr>
</tbody>
</table>

Table 14 shows the application of the linear correlation tests to ion interaction for given specific conditions. Case 1 is where Na$^+$ ion constitutes 8 percent or more of the total cations in the extract. Under these conditions Na$^+$ is highly correlated with ECF as is (Ca$^{2+} \times$ Na$^+$). In addition, the (Ca$^{2+} +$ Mg$^{2+}$) (Na$^+$) interaction is also highly related to ECF. Locations which had the condition of ≥ 8 percent Na$^+$ were site numbers 8, 10, 13, 15, 16, and 17. Case 2 is where the Na$^+$ is less than 8 percent of the total cations in the soil extract. Under this constraint, all correlation coefficients decreased. The highest correlation (0.9715) existed for the (Ca$^{2+} \times$ Na$^+$) interaction with the 1:1 extract.

Several conclusions can be drawn from the statistical analyses. The results show that the 1:1 soil to water ratio extract data correlated as well or better, with the ECF data, than did the saturation extract data. This is fortunate since obtaining saturation extracts is time consuming and requires specialized equipment.

The (Ca$^{2+} \times$ Na$^+$) interaction was highly correlated to the ECF values for all the sites studied. No single chemical analysis proved to
Table 14. Linear correlation for ion interaction of the soil extract and the ECF value.

<table>
<thead>
<tr>
<th>Variable</th>
<th>Correlation coefficients (r)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Sat. Extracts</td>
</tr>
<tr>
<td><strong>CASE 1 (Na⁺ &gt; 8%)</strong></td>
<td></td>
</tr>
<tr>
<td>Na⁺</td>
<td>.9886</td>
</tr>
<tr>
<td>Mg²⁺ + Ca²⁺</td>
<td>.8773</td>
</tr>
<tr>
<td>Ca²⁺ x Na⁺</td>
<td>.9928</td>
</tr>
<tr>
<td>(Ca²⁺ + Mg²⁺) Na⁺</td>
<td>.9895</td>
</tr>
<tr>
<td><strong>CASE 2 (Na⁺ &lt; 8%)</strong></td>
<td></td>
</tr>
<tr>
<td>Na⁺</td>
<td>.6507</td>
</tr>
<tr>
<td>Mg²⁺ + Ca²⁺</td>
<td>.7194</td>
</tr>
<tr>
<td>Ca²⁺ x Na⁺</td>
<td>.9178</td>
</tr>
<tr>
<td>(Ca²⁺ + Mg²⁺) Na⁺</td>
<td>.9182</td>
</tr>
</tbody>
</table>

be as highly correlated with ECF. However, the (Ca⁺² x Na⁺) factor did not correlate to the same degree in systems with high and low sodium contents, i.e., greater or less than 8 percent Na⁺. This fact is shown in the regression equations derived from the 1:1 data.

For Na⁺ ≥ 8 percent of total cations:

ECF = 55.717 + 0.428 (Na⁺ x Ca⁺²), r² = 0.987.

For Na⁺ < 8 percent of total cations:

ECF = 30.03 + 33.11 (Na⁺ x Ca⁺²), r² = 0.944.

To this point, the data suggest that no simple relation exists between the soil salinity and the salinity that is found in the overland flow that occurs over the soil. Only cation concentration data of the soil extracts have been statistically analyzed for linear correlation with ECF. These data were chosen because of the ease with which cations can be analyzed relative to the anions.
Suspended sediment

Table 15 illustrates the suspended sediment (S.S.) concentrations for the various sites. With respect to the Mancos shale, suspended sediment concentrations are highest for the middle and lower Blue Gate divisions (site 14 and 15, although classified as Mancos Undivided, can be considered to be on the Blue Gate member of the Mancos). This is a significant point since these types are the potentially highest salt contribution in the basin, excluding alluvial deposits with visible salt crusts next to stream channels.

It was noted earlier that EC values of the soil extracts are much higher than those of surface runoff water. If one observes the EC values in the Price River water reported by Mundorff (4), it is apparent that surface water flowing through the Mancos members also has EC values greater than those obtained from the surface runoff studies. This is true even when the incoming salt mass and concentration by evaporation are taken into account.

Consequently, it is presently felt that sediment obtained by overland flow contributes to the salt load in the river with time. As Table

<table>
<thead>
<tr>
<th>Site (Geologic Code)</th>
<th>S.S. (g/l)</th>
<th>Site (Geologic Code)</th>
<th>S.S. (g/l)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 (M)</td>
<td>2.32</td>
<td>14 (MUD)</td>
<td>8.38</td>
</tr>
<tr>
<td>2 (M)</td>
<td>2.01</td>
<td>15 (MUD)</td>
<td>9.30</td>
</tr>
<tr>
<td>3 (M)</td>
<td></td>
<td>16 (CM)</td>
<td>2.72</td>
</tr>
<tr>
<td>4 (UBG)</td>
<td>2.47</td>
<td>17 (CM)</td>
<td>0.78</td>
</tr>
<tr>
<td>5 (UBG)</td>
<td>5.29</td>
<td>18 (AD)</td>
<td>1.82</td>
</tr>
<tr>
<td>6 (MUG)</td>
<td>6.13</td>
<td>19 (AD)</td>
<td>3.43</td>
</tr>
<tr>
<td>7 (MUG)</td>
<td>7.77</td>
<td>20 (GC)</td>
<td>2.21</td>
</tr>
<tr>
<td>8 (LBG)</td>
<td>6.98</td>
<td>21 (GC)</td>
<td>1.33</td>
</tr>
<tr>
<td>9 (LBG)</td>
<td>7.06</td>
<td>22 (BH)</td>
<td>2.54</td>
</tr>
<tr>
<td>10 (T)</td>
<td>2.14</td>
<td>23 (PR)</td>
<td>5.95</td>
</tr>
<tr>
<td>11 (T)</td>
<td>1.48</td>
<td>24 (NH)</td>
<td>5.90</td>
</tr>
<tr>
<td>12 (T)</td>
<td>4.00</td>
<td>25 (C)</td>
<td>5.50</td>
</tr>
<tr>
<td>13 (MUD)</td>
<td>3.71</td>
<td>26 (GR)</td>
<td>7.36</td>
</tr>
</tbody>
</table>
15 illustrates, substantial sediment concentrations are being released from the Mancos Members. What is currently lacking is knowledge of the rate of salt release from these suspended particles. Studies to examine the kinetics involved are in progress.

It is important that we understand the processes involved with suspended sediment in the surface runoff water. Table 3 and Figure 4 would lead one to believe it is not a major contributor to the salinity problem in the basin. However, it may in fact be a problem when the potential of salt release by the sediment it yields is considered.

Vegetative study

Preliminary studies have indicated that the pattern and magnitude of salt removal is different for each plant species. Figure 5 is a graphic expression of the data in Table 6. In general, the plant material continues to contribute salt over several minutes, then the salt contributing rate diminishes. Salt from pinyon and juniper is minimal while species like mat saltbush contribute greater quantities of salt. Data in Table 7 provide further evidence for identifying potentials for salt contributions, with important species being mat saltbush, halogeton, gardner saltbush, salt cedar, shadscale, greasewood, and four wing saltbush.

Infiltrometer trials in the field have not shown a consistent relationship between total plant cover and salt loading of overland flow (Figures 6 and 7). In these figures each vertical line represents one infiltrometer plot, with the square representing the initial conductivity measure of runoff after 8 to 13 minutes and the triangle representing the final conductivity measure of all runoff after 28 minutes of simulated rainfall. Figure 6 illustrates the EC vs percent plant cover for the Tununk member. Site 10 is represented by the dashed lines, site 11 by the solid lines, and site 12 by the dot-dash lines. Even though site 12 has a much higher EC value at any given percent plant cover, it is obvious that within sites as well as between sites 10 and 11 there is no relation between salt released and percent vegetation cover. The Upper Blue Gate member is
Figure 5. Quantity of salt released in a known volume of distilled water as a function of time per one gram of plant material.

illustrated in Figure 7. The dashed lines represent site 4 while site 5 is represented by the solid lines. It can be noted that a slight relation exists between initial conductivity values and percent cover at the extreme cover values. However, due to the variability present in the intermediate values (10 to 55 percent cover) no definite conclusions can be drawn relating EC to percent cover. In fact, the infiltrometer runs illustrated in Figures 6 and 7 were designed for other purposes and may or may not reflect the actual impact of salt contributions from certain plant species.

Examination of Table 8 illustrates that both the chemical composition of the leachate and the degree of leachability varied with plant litter when subjected to a high intensity rain for one hour. Greasewood, gardener saltbush, shadscale, four wing saltbush, big sagebrush
Figure 6. Electrical conductivity in relation to percent plant cover for the Tununk member of the Mancos Shale.

Figure 7. Electrical conductivity in relation to percent plant cover for the Upper Blue Gate member of the Mancos Shale.
released about 75 percent, 36 percent, 34 percent, 29 percent, and 29 percent of their salt content under the 3 in/hr rain, respectively.

The largest amount of salt in the leachates was observed for gardener saltbush, greasewood, mat sagebrush, halogeton, shadscale, four wing saltbush, salt cedar, rabbitbrush, big sagebrush, juniper, and Indian ricegrass, respectively, in declining order. About 30 to 75 percent of the salt could be leached from the litter in less than 30 minutes under the 3 in/hr intensity rainfall.

Table 9 shows the salt removal from 50 gms of dry litter under 3 in/hr rainfall in terms of the EC of the leachate. The difference in salt production between species is marked.

Work is continuing at both the field and laboratory level to fully ascertain the contribution of natural vegetation to diffuse salt production.

*Overland flow-salt transport model*

A computer model is presently being developed which will predict salt release (TDS) from small watersheds on the Mancos shale as a function of surface runoff. This model is composed of three components; a surface hydrology function, a surface chemistry function, and a soil-water chemistry function. Sediment yield and associated mineral releases will also be accounted for.

Micro-watershed plots, about 40 ft², have been established on the Lower Blue Gate member of the Mancos shale. Simulated rainfall will be applied to these plots by a modified infiltrometer apparatus during the summer of 1975. The resulting data from these studies will be used to calibrate the model. Micro-watersheds have also been established on the Upper Blue Gate member. Resulting data from these watersheds will be used to verify the model. This model should be operative by October 1975.
Conclusions

The results of the first year's research allows several conclusions to be drawn:

1. Alluvial deposits near the channel of a perennial stream tend to yield runoff water of high salinity.

2. The Blue Gate and Mancos Undivided members are the prime salt producers in the basin. Variation within the Blue Gate member does exist with the middle and lower divisions yielding runoff water of higher salinity than the upper member.

3. No simple relation exists between the soil salinity and the salinity of the overland flow over the respective soil.

4. Certain plant species have been shown in preliminary studies to have a potential for contributing to the total salt load in overland flow. Further studies are needed to further quantify this impact.

5. Sediment obtained by overland flow may play an important role in salt production from the basin. Studies are in progress to examine this problem.

6. The relationship between geologic type and surface hydrology was limited. A slight correlation between geologic type and the ratio of Q to P was found; while hydrologic soil-complex numbers showed no distinct separation between types.

7. Three distinct clusters of \( k \) values for the Horton infiltration equation were found to exist. Most of the non-Mancos sites fell around 1.18 min\(^{-1}\), while the Mancos members clustered around 0.23 and 0.33 min\(^{-1}\) values.

8. Strong linear relations exist between EC and Ca\(^{+2}\) and SO\(^{4-2}\), while very poor correlations exist between Q/P and Q to TS for runoff from the Mancos members.

9. The 1:1 soil to water ratio extract data correlated with the ECF data better than did the saturation extract data.
References


This study is concerned with modeling of one of the possibilities for ameliorating the salinity problem of downstream Colorado River waters. The study deals with the physical nature and the cost effectiveness of an irrigation management approach to reducing salinity in the river.

Irrigation return flow constitutes a large portion of the water in streams and rivers of the western United States. In some river basins, such as the Colorado River Basin, some water may by "used" for irrigation several times before entering the ocean. Since this "use" involves the evapotranspiration process which accounts for the major loss of water by crops, there is an inevitable buildup of salt concentration in irrigation return flows. This is seen in the salinity of the Colorado River which ranges from less than 50 mg/l (total dissolved solids) in the upper basin mountains to about 850 mg/l at the Imperial Dam in lower California. While irrigation return flow is involved in only part of this salinity concentration, it has been suggested to be one of the major areas capable of management. Little research work has been done on management of irrigation water to influence downstream salinity and, therefore, relatively little is known about the manifold effects of such management. This study is an attempt to evaluate some of these effects. Specifically, the study
involves (1) the development of a physical model to predict the response of soil, water, and crop factors to irrigation and (2) the development of an economic model which, using the physical model for basic data, assesses the cost effectiveness of irrigation management as related to return flow salinity.

Economic background of the study

The salinity problem in river basins, especially in large ones like the Colorado River Basin, is an interesting and difficult challenge to policy makers. The well-being of some users of the river conflicts with the well-being of others in river use programs that have been or may be undertaken. An ideal competitive economy would yield an allocation of resources such that no alternative pattern of resource use would make anyone better off without making someone worse off. This ideal situation does not exist in the matter of allocation of water and the quality aspects of water for at least two reasons. First, prices do not correctly reflect the social value of resources and commodities. Misallocations of resources occur. The individual decision-maker has no incentive (except for his conscience or good will) for taking all costs or benefits into account in making a resource allocation decision. Second, producers of "public goods" are unable to collect revenues from beneficiaries, since users cannot be excluded for nonpayment of the price. Each user may expect to reap the benefits whether or not he pays the cost. The private market is, therefore, unable to supply optimal amounts of goods with collective consumption characteristics. The salinity problem in the Colorado River exhibits both of these aspects of market failure. More than half of the salinity concentration in the river is due to natural causes, but if there were no man-made effects, the concentrations would probably not be sufficient to trouble downstream users.
General procedure

The study was done in two phases. The first phase involved the development of the physical model to be used to supply basic data. The second phase involved the development of an economic model to analyze cost effectiveness. While these two phases were carried out somewhat independently at the beginning of the study, it soon became apparent that much interchange was necessary. The physical model originally produced much information not needed for the economic model and did not supply some basic data needed. Thus, considerable modification of the physical model was necessary. Similarly, the economic model originally devised assumed availability of basic physical data that could not be obtained. Thus, the economic model had to be adjusted to use the basic data that was obtainable.

The details of the two models are discussed separately in the following pages for purposes of organization. This will allow the reader to consider only one of the models according to his interest. However, we have found much to be gained by interchange of ideas and methods necessary to develop answers to a particular problem and would advise considering both models together.

The Physical Model

Recent field work has shown that many situations are much more complicated than can be handled by present models of plant response to salinity. The field situation discussed in this paper, for example, was studied by Gupta (1972) and King and Hanks (1973). They found the models used previously gave good prediction for the water portion but poor prediction for the salt portion. Where water of different salt concentrations had been added as irrigation water, there was a very small effect on the salt concentration of the soil solution. It appeared that the soil acted like a large buffer that was influenced only slowly by relatively small salt additions or removals through
irrigation and drainage. It became evident that the inclusion of com-
plicated reactions used by Dutt et al. (1972) were of little practical
use because they were not completely accurate and they required con-
siderable computer time. Consequently, it was decided to devise a
simplified salt flow model to simulate the long time effects of salt
buildup by varying the initial conditions.

The model is based on the work of Nimah and Hanks (1972a, b)
which is concerned with the soil water flow in response to varying
irrigation management inputs. The general equation for water flow
is given as Equation (1):

\[ \frac{\partial \theta}{\partial t} = \frac{\partial}{\partial z} \left( K \frac{\partial H}{\partial z} \right) + a(z) \]

where \( \theta \) is the water content, \( H \) is the matric potential, \( K \) is the hy-
draulic conductivity, \( t \) is time and \( z \) is depth, and \( a(z) \) is the root
extraction term.

The root extraction term is somewhat more complicated because
it has plant and soil characteristics in it as the following equation
shows:

\[ a(z) = \frac{H_{\text{root}} + 1.05z - h(z, t) - s(z, t)}{\Delta z \cdot \Delta x} \cdot \text{RDF}(z) \cdot K \]

where \( H_{\text{root}} \) is the water potential at the surface of the root which is
modified to have a different water potential due to gravity and a small
friction resistance term of 0.05, \( h(z, t) \) is the soil solution matric
potential, \( s(z, t) \) is the osmotic potential, and \( \text{RDF}(z) \) is a root density
function. \( \Delta x \) is the distance between the plant root and the point in
the soil where \( K \) is realized (we assume equal to 1.0). Depending on
the climate and the plant and soil conditions, water may be extracted
from the soil without any limitations so that the transpiration would
be equal to that potential transpiration. However, if the osmotic con-
centration or the matric potential is sufficiently low, keeping in mind
the negative sign, the soil water system will not be able to supply
sufficient water to the plant to maintain the transpiration at potential transpiration and then the transpiration falls off. These equations have been discussed in considerably more detail in Nimah and Hanks (1973a) and Childs and Hanks (1975).

The salt flow portion of the model is given as follows:

$$\frac{\partial(C_0)}{\partial t} = \frac{\partial}{\partial z} (D \frac{\partial C}{\partial z}) - \frac{d(Cq)}{dz} \quad \ldots \quad (3)$$

where $C$ is the salt concentration, $D$ is the combined diffusion and dispersion coefficients, and $q$ is the mass flux of water.

Salt is assumed to move within the soil profile according to mass flow of water and subject to the diffusion restrictions. No consideration is taken for source or sink term where precipitation or solution of salts could come out of the solid phase of the soil. A numerical approximation of both the water flow and moisture flow parts of the model have been written as described by Nimah and Hanks (1973) and Childs and Hanks (1975), as well as Hanks et al. (1974). To determine the influence of the salinity on crop yield, another component of the model has to be added. This is done by using the assumptions that have been described by Hanks (1974) and Childs and Hanks (1975) where the relative yield of a crop is related to the relative transpiration.

The validity of this assumption for saline conditions is substantiated by the data of Lunin and Gallatin (1965), Bingham and Garber (1970), and Shelhavet and Bernstein (1968). A linear relationship between relative transpiration and relative yield is indicated. Relative yield is here restricted to the dry matter yield and does not include the yield of grain which might be considerably more complicated.

The estimation of a relative yield is necessary to interface with the economic model discussed later. The variations that are sensed by the model are the result of various initial conditions or boundary conditions that change with time at the top and bottom of the soil. The soil conditions also influence the results as well as the crop conditions because soil properties influence water uptake and water infiltration.
in the soil. The plant grown also influences root uptake as well as the boundary conditions of the surface.

As described in detail by Childs and Hanks (1975), it is necessary to determine what the potential evapotranspiration or the potential infiltration rate for the soil would be for any kind of a management system that is imposed. This is done by either measurement of the potential evapotranspiration such as described by Nimah and Hanks (1973b) or by using some method such as described by Jensen (1973) to compute potential evapotranspiration. This model does not require an estimation of the crop coefficient but requires that the potential evapotranspiration be broken into potential evaporation and potential transpiration as described by Childs and Hanks (1975).

The basic input data required for the model are given in detail by Nimah and Hanks (1973) and Childs and Hanks (1975), but are summarized as follows: (1) Hydraulic conductivity, water content, and matric potential water content data covering the range of water content to be encountered during the period of interest (soil property), (2) air dry soil water contents (soil property), (3) root water potential below which the root will not go where presumably the plant wilts and the actual transpiration will be less than the potential transpiration (plant property), (4) root distribution function for the period of study (plant property), (5) water content and soil solution concentration data at the beginning as a function of depth (initial condition), (6) potential transpiration, potential evapotranspiration rate and potential irrigation or rainfall as a function of time for the whole period of the run (boundary condition) (potential evapotranspiration assumed equal to that from a free water surface could be calculated by the use of the Penman or some other equation as described by Jensen (1966)), (7) osmotic potential of irrigation water (boundary condition), (8) presence or absence of a water table at the bottom of the soil profile (boundary condition). The root density function may be changed as a function of time and depth as the root system grows as described by Childs and Hanks (1975).
The output data can be selected from among many variables that are computed within the model from a list of the following: (1) Cumulative evapotranspiration, transpiration, and evaporation as a function of time, (2) volumetric soil water content and soil pressure head as a function of time and depth, (3) cumulative water flow upward or downward through any boundary within the profile or at the surface, (4) the value of $H_{\text{root}}$ as a function of time, or many other factors. The main item of interest in this computation is the relative transpiration which is the transpiration computed from the particular management system compared to what the potential transpiration would have been at the same condition if soil water were not limiting.

The Economic Model

The economic model is designed to suggest ways to minimize the income losses imposed by restraints on salt outflow due to irrigation on the farm. It is based on the physical model and a set of cost and return data for the farm. The beginning point is to assume that any amount of salt can be allowed to leave the farm. The model is set to maximize income under this assumption which has been the policy in the past. The model is then successively constrained to allow smaller and smaller amounts of salt to leave the farm. Of primary concern is the income reduction which accompanies this constraint on resource use. Also of concern are the crops grown, irrigation management practices, and the quantity of water applied. As the salt outflow and incomes incrementally change, the model develops as a byproduct the marginal relationship between salt outflow and income. This value can then be compared with alternative ways of reducing salinity in the river or the damages that accrue to downstream users. The implementation of the economic model is in the form of the linear programming model of economic behavior.
The linear programming model of salt outflow

The linear programming model used in this study is a profit maximizing model which has the algebraic form of:

\[
\text{maximize } \quad Z = CX \\
\text{subject to } \quad AX \leq B \\
X \geq 0
\]

where \( Z = \text{net revenue (or profit)} \)
\[C = \text{the row vector of net revenue per unit of activity}\]
\[X = \text{the set of activities or production processes}\]
\[A = \text{matrix of technical coefficients (or production relationships)}\]
\[B = \text{the column vector of constraints of resource availability}\]

Linear programming and the economic concepts utilized are discussed by Leftwich (1970). The application to the present study is as follows: (1) Select the combination of crops produced and management practices subject to the constraints in certain fixed inputs such as land. The selection is based on the operating costs and the relative prices of the crops. (2) Many of the inputs are not fixed, thus the optimal combination of these variable inputs is selected for the production of the crops produced based on their productivity and the cost of inputs. (3) The level of output per acre is selected based on producing up to but no more than the level where the value of the incremental unit of production equals the cost of the incremental inputs unit of input.

In this study, the various components of the model are defined and constructed as follows.

Processes and activities

Production activities (the \( x_i \)) have been developed which are most relevant. These are activities like growing corn silage, or oats or alfalfa hay. Each of these can also be treated in alternative ways such as with different quantities of irrigation applied by sprinkling or flooding. All combinations of these alternative actions were used in
this study except that flood irrigation was not used in the lowest three levels of water application. It would be impossible to distribute the small amounts of water uniformly over the season by flooding.

Resource constraints

Limits on resource availability (the $b_i$) used in this study include the quantities of each of three land classes based on the beginning salinity levels of the soil profile. It was assumed that the farmer had 10 acres with each of three soil salinity characteristics. Unlimited salt outflow was allowed in the drainage water (which level was subsequently reduced to determine the profitability to the farm operator of letting salt flow into the drains and streams). There were also constraints to force growing of crops in rotation such as to provide for nurse crops for new seedings of alfalfa, limits on corn production for disease control, and diversity of crops according to farmer preference.

Yields and prices

Net profitability for each unit of production was based on approximate current prices for products and the costs of various farming supplies and operations. Yields were estimated using the 1971 data for the farm as a base with the relative yields predicted in the physical model to give specific values for the rates of water applied as influenced by the initial salt concentrations shown in Tables 10 and 11. The profit function is based on the price of alfalfa at $45/ton, corn silage at $13/ton, and oats at $1.60/bushel. These prices represent approximately the current prices but are adjusted somewhat to a normal long run relationship to each other.

Situations Studied

There were several situations studied in terms of water management. The data for Vernal, Utah, 1971, as described by Nimah and
Hanks (1973b) were taken for the initial conditions and water was applied in different amounts but at the same frequency as given in the 1971 data. The irrigation water quality which was used throughout was 6.35 meq/liter, which was equivalent to the present conditions at the Vernal, Utah, farm.

To simulate the effects of soil salinity storage within the root profile, three different levels of soil salinity were studied—20 meq/l uniform throughout, which is approximately the condition on most of the farm at present, 50 meq/l uniform throughout and 200 meq/l uniform throughout.

Because data were collected from various sources for the three crops that were studied on the farm, the root distribution functions were arbitrarily chosen as shown in Table 1 for the three crops studied. The corn and oats were modeled as annual crops with different values of crop cover as a function of time during the year. This had an influence on the potential evapotranspiration distribution as described by Childs and Hanks (1975).

Table 1. Relative proportion of roots at different depth increments at maturation assumed for the calculations.

<table>
<thead>
<tr>
<th>Depth</th>
<th>Corn</th>
<th>Alfalfa</th>
<th>Oats</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.5 to 10.5 cm</td>
<td>.09</td>
<td>.14</td>
<td>.18</td>
</tr>
<tr>
<td>10.5 to 25.5</td>
<td>.20</td>
<td>.30</td>
<td>.40</td>
</tr>
<tr>
<td>25.5 to 52.5</td>
<td>.34</td>
<td>.33</td>
<td>.42</td>
</tr>
<tr>
<td>52.5 to 91.5</td>
<td>.25</td>
<td>.23</td>
<td>0</td>
</tr>
<tr>
<td>92.5 to 140.0</td>
<td>.12</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>140.0 to 235.0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

Two different irrigation systems were studied. The first was a solid set sprinkler system with a coefficient of uniformity of .88 which is approximately the same as now in place on the experimental
farm being studied. This was compared to a very poor gravity system which was on the farm before the sprinkler system was applied. The coefficient of uniformity of the gravity system was 0.42 which is a very poor system but is useful for comparison of the effect of a range of application uniformities.

Results

The physical relationships

The results of modeling a variation in the water added and initial salt concentration on various soil and water properties for corn are shown in Table 2. The data on $T/T_p$ are of primary interest because they are assumed to correspond to relative yield. The data of Table 2 show that $T/T_p$ increases as the irrigation applied is increased up to about 50 cm after which the ratio was 1.0 for all initial salt concentrations. The ratio $T/T_p$ was smaller, however, where irrigation was limited for the higher initial salt concentrations. There was relatively little difference among the values for $T/T_p$ when the initial salt concentrations were 20 or 50 meq/l, indicating that yield differences were due to water influences only. Note that where the irrigation and rain was less than about 20 cm, there was an upward flow. The amount of flow was limited by soil water transmission and plant root extraction. In cases where the initial salt concentration was 200 meq/l, upward flow was about 2.5 cm less than for the lower initial salt concentrations. However, drainage (downward flow) was influenced very little by initial salt concentrations.

One feature of the data shown in Table 2 that may be somewhat unique is the large influence of water movement up from the water table (at a depth of 235 cm). The soil properties at the Vernal farm seem to be especially conducive to high water flow in both directions. Other situations with other soils would probably not result in as much upward flow.
Table 2. Comparison of irrigation water applied and initial salt concentration on relative transpiration of corn $T/T_p$, total water used, drainage, salt flow to the groundwater, and average final salt concentration.

<table>
<thead>
<tr>
<th>Rain</th>
<th>ET</th>
<th>Transpiration</th>
<th>Salt Flow</th>
<th>Initial Salt</th>
<th>Final Salt</th>
</tr>
</thead>
<tbody>
<tr>
<td>cm</td>
<td>cm</td>
<td></td>
<td>$T/T_p$ cm</td>
<td>to Drainage meq</td>
<td>Concentration /l</td>
</tr>
<tr>
<td>5.6</td>
<td>40.3</td>
<td>35.3</td>
<td>.81</td>
<td>-14.2</td>
<td>-284</td>
</tr>
<tr>
<td>5.6</td>
<td>38.6</td>
<td>33.5</td>
<td>.77</td>
<td>-14.2</td>
<td>-710</td>
</tr>
<tr>
<td>5.6</td>
<td>26.2</td>
<td>20.6</td>
<td>.48</td>
<td>-11.6</td>
<td>-2320</td>
</tr>
<tr>
<td>10.3</td>
<td>43.9</td>
<td>36.6</td>
<td>.89</td>
<td>-14.1</td>
<td>-282</td>
</tr>
<tr>
<td>10.3</td>
<td>42.1</td>
<td>35.1</td>
<td>.86</td>
<td>-14.0</td>
<td>-700</td>
</tr>
<tr>
<td>10.3</td>
<td>30.1</td>
<td>22.3</td>
<td>.55</td>
<td>-11.4</td>
<td>-2280</td>
</tr>
<tr>
<td>15.0</td>
<td>47.7</td>
<td>38.6</td>
<td>.97</td>
<td>-14.0</td>
<td>-280</td>
</tr>
<tr>
<td>15.0</td>
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<td>37.2</td>
<td>.93</td>
<td>-13.9</td>
<td>-695</td>
</tr>
<tr>
<td>15.0</td>
<td>34.6</td>
<td>25.1</td>
<td>.64</td>
<td>-11.4</td>
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</tr>
<tr>
<td>22.0</td>
<td>49.0</td>
<td>38.5</td>
<td>.98</td>
<td>-13.6</td>
<td>-272</td>
</tr>
<tr>
<td>22.0</td>
<td>49.2</td>
<td>38.7</td>
<td>.98</td>
<td>-13.5</td>
<td>-675</td>
</tr>
<tr>
<td>22.0</td>
<td>41.2</td>
<td>30.9</td>
<td>.78</td>
<td>-11.3</td>
<td>-2260</td>
</tr>
<tr>
<td>40.8</td>
<td>50.4</td>
<td>37.6</td>
<td>.99</td>
<td>-8.7</td>
<td>-174</td>
</tr>
<tr>
<td>40.8</td>
<td>48.3</td>
<td>35.9</td>
<td>.98</td>
<td>-7.1</td>
<td>-355</td>
</tr>
<tr>
<td>40.8</td>
<td>48.1</td>
<td>35.8</td>
<td>.97</td>
<td>-6.2</td>
<td>-1240</td>
</tr>
<tr>
<td>56.4</td>
<td>51.9</td>
<td>37.3</td>
<td>1.00</td>
<td>+0.91</td>
<td>19</td>
</tr>
<tr>
<td>56.4</td>
<td>52.2</td>
<td>37.3</td>
<td>1.00</td>
<td>+1.0</td>
<td>49</td>
</tr>
<tr>
<td>56.4</td>
<td>56.7</td>
<td>37.3</td>
<td>1.00</td>
<td>+1.1</td>
<td>214</td>
</tr>
<tr>
<td>66.7</td>
<td>51.7</td>
<td>37.3</td>
<td>1.00</td>
<td>+10.5</td>
<td>210</td>
</tr>
<tr>
<td>66.7</td>
<td>51.6</td>
<td>37.3</td>
<td>1.00</td>
<td>+10.6</td>
<td>532</td>
</tr>
<tr>
<td>66.7</td>
<td>51.6</td>
<td>37.3</td>
<td>1.00</td>
<td>+10.8</td>
<td>2160</td>
</tr>
</tbody>
</table>

Note: Each line represents a computation with the same irrigation frequency but different amounts of water applied for climatic conditions of 1971 at Vernal, Utah. A negative sign indicates upward flow of salt and water. Rain was 5.6 cm.
The data shown in Table 2 are only a small part of the data collected in attaining these summary values. Each line represents one season where data have been computed at several depth increments and at no greater than 2- to 3-hour increments. Thus, data within the season are also available. Figure 1 shows a comparison of cumulative evapotranspiration as influenced by initial salt concentration for two different irrigation levels.

Table 3 shows the computations of \( T/T_p \) made for alfalfa. The data show more decrease of \( T/T_p \) for low irrigation rates than was shown for corn. This was due to a longer season for active water use by alfalfa and for a much greater proportion of transpiration to evapotranspiration for alfalfa than for corn--especially during early season when corn was just planted. Upward water flow was less for alfalfa than corn, probably due to alfalfa's assumed shallow root distribution. This result is probably not representative of other situations where alfalfa normally roots deeper than corn. The alfalfa root distribution was measured at the site where there is upward water movement, but the corn root depths were measured at another location. Like corn, the alfalfa data show little difference between the 20 and 50 meq/l initial salt concentrations but fairly large differences with 200 meq/l initial salt concentrations. Thus, the \( T/T_p \) depression at 20 meq/l initial salt concentration is due to inadequate irrigation. The differences in \( T/T_p \) at any one irrigation level, for initial salt concentrations between 20 and 200 meq/l, were due strictly to a salt effect--where 15 cm of irrigation and rain was applied, \( T/T_p \) was 0.68 because water was insufficient to maintain transpiration. A further reduction of \( T/T_p \) from 0.68 to 0.49 resulted from the high initial salt concentration.

Table 4 shows the computed data for oats when irrigation water was managed in a manner similar to corn and alfalfa. The values of \( T/T_p \) were smaller for oats for a given irrigation regime than for corn or alfalfa. This was due mainly to a more shallow root depth, but was also partly due to a difference in the relation of \( T_p \) to \( ET_p \). Because
of the shallow root zone, upward flow was less than 4 cm. This caused the ratio, $T/T_p$, to be less than 0.9 (for 20 meq/l initial salt concentration) when irrigation and rain was less than about 52 cm. As was the case for alfalfa and corn, the $T/T_p$ results with 50 meq/l initial salt concentration were only slightly different than for 20 meq/l, whereas the changes in $T/T_p$ from 50 meq/l to 200 meq/l were considerably larger.

There is a feature of the computation that is especially noticeable in Table 2 for corn. The model allows for the possibility that, if evap-

![Figure 1. Cumulative evapotranspiration as a function of time for two levels of irrigation, I, and rain, R, at two different initial salt concentrations.](image)
Table 3. Comparison of irrigation water applied and initial salt concentration on relative transpiration of alfalfa, \( T/T_p \), evapotranspiration ET, drainage, salt flow to the groundwater, and average final salt concentration.

<table>
<thead>
<tr>
<th>Irrig. and Rain</th>
<th>ET cm</th>
<th>T cm</th>
<th>T/T_p cm</th>
<th>Salt Flow to Drainage cm</th>
<th>Initial Salt Concentration meq</th>
<th>Final Salt Concentration Average meq/l</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.6</td>
<td>29.5</td>
<td>25.8</td>
<td>.52</td>
<td>-9.7</td>
<td>-195</td>
<td>20</td>
</tr>
<tr>
<td>5.6</td>
<td>28.2</td>
<td>26.6</td>
<td>.50</td>
<td>-9.4</td>
<td>-472</td>
<td>50</td>
</tr>
<tr>
<td>5.6</td>
<td>19.8</td>
<td>16.0</td>
<td>.33</td>
<td>-7.8</td>
<td>-1561</td>
<td>200</td>
</tr>
<tr>
<td>10.3</td>
<td>33.2</td>
<td>29.2</td>
<td>.61</td>
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<td>10.3</td>
<td>32.1</td>
<td>28.1</td>
<td>.58</td>
<td>-9.3</td>
<td>-466</td>
<td>50</td>
</tr>
<tr>
<td>10.3</td>
<td>24.2</td>
<td>20.0</td>
<td>.42</td>
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<td>1.00</td>
<td>0.0</td>
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<td>1.00</td>
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<td>1.00</td>
<td>9.4</td>
<td>1882</td>
<td>200</td>
</tr>
</tbody>
</table>

Note: Each line represents a computation with the same irrigation frequency but different amounts of water applied for climatic condition of 1971 at Vernal, Utah. A negative sign indicates upward flow of salt and water. Rain was 5.6 cm.
Table 4. Comparison of irrigation water applied and initial salt concentration on relative transpiration, for oats, \( T/T_p \), evapotranspiration, ET, drainage, salt flow to the groundwater, and average final salt concentration.

| Irrig. and Rain cm | ET cm | T | T/T_p | Upward Flow cm | Salt Flow to Groundwater meq | Initial Salt Concentration meq/l | Final Salt Concentration Average meq/l |
|-------------------|-------|---|#######|---------------|-----------------------------|-------------------------------|-----------------------------------|
| 5.6               | 18.3  | 13.3 | .29   | -3.8          | -74                         | 20                            | 33                                |
| 5.6               | 18.0  | 12.9 | .28   | -3.8          | -191                        | 50                            | 78                                |
| 5.6               | 14.3  | 8.2  | .18   | -3.6          | -718                        | 200                           | 248                               |
| 10.3              | 22.7  | 16.4 | .37   | -3.8          | -76                         | 20                            | 33                                |
| 10.3              | 22.2  | 16.1 | .36   | -3.8          | -190                        | 50                            | 76                                |
| 10.3              | 18.4  | 10.2 | .24   | -3.5          | -700                        | 200                           | 242                               |
| 15.0              | 27.1  | 20.2 | .46   | -3.8          | -76                         | 20                            | 33                                |
| 15.0              | 26.7  | 19.4 | .44   | -3.8          | -189                        | 50                            | 76                                |
| 15.0              | 22.9  | 13.3 | .32   | -3.5          | -700                        | 200                           | 242                               |
| 22.0              | 33.8  | 25.6 | .59   | -3.8          | -76                         | 20                            | 33                                |
| 22.0              | 33.4  | 25.3 | .58   | -3.8          | -190                        | 50                            | 76                                |
| 22.0              | 29.5  | 19.3 | .46   | -3.3          | -660                        | 200                           | 240                               |
| 40.8              | 46.0  | 35.2 | .89   | -2.5          | -50                         | 20                            | 26                                |
| 40.8              | 45.7  | 35.1 | .88   | -2.4          | -120                        | 50                            | 58                                |
| 40.8              | 42.3  | 31.5 | .80   | -1.2          | -240                        | 200                           | 208                               |
| 56.4              | 53.6  | 38.5 | .97   | +1.3          | +26                         | 20                            | 90                                |
| 56.4              | 53.4  | 38.8 | .98   | +1.3          | +66                         | 50                            | 92                                |
| 56.4              | 51.4  | 37.0 | .93   | +2.5          | +490                        | 200                           | 350                               |
| 66.7              | 52.5  | 38.6 | .99   | +10.0         | +198                        | 20                            | 20                                |
| 66.7              | 52.5  | 38.6 | .99   | +10.0         | +495                        | 50                            | 43                                |
| 66.7              | 52.5  | 38.6 | .99   | +9.9          | +1975                       | 200                           | 157                               |

Note: Each line represents a computation with the same irrigation frequency but different amounts of water applied for the climatic conditions of 1971 at Vernal, Utah. A negative sign indicates upward flow of salt and water. Rain was 5.6 cm.
Oration is less than potential evaporation, the difference, $E_p - E$, can be used in transpiration. Thus, potential transpiration is not a constant in Table 2 but increases as the irrigation and rain applied decreases. For a rain of 5.6 cm, $T_p$ for corn was 40.3 and for irrigation and rain of 56.4 cm, $T_p$ was 37.3 cm. Hanks et al. (1971) have demonstrated that this energy "trading" occurs, but it may be that the model computation overcorrects for it.

Figure 2 shows the salt concentration profiles for corn at the end of the season compared to the beginning for three differing levels of water application. Where irrigation was insufficient to cause drainage, there was a higher concentration of salt throughout the profile at the end of the season. There was a very pronounced peak in salt concentration just below the root zone, especially for the low water levels.

Figure 2 also shows the salt concentration profiles at the end of the year for oats. These concentrations are higher in the profile than those for corn because a more shallow root distribution for oats was assumed. There was relatively little water available for transpiration and the salt peak was lower with 5.6 cm of rain than when 22 cm of irrigation and rain provided for more transpiration and thus more concentration of salt. Where sufficient water for some leaching was available, the salt concentration in the profile was essentially constant.

Figure 3 shows a 10-year computation during which irrigation and rain were about one-half ET. The data indicate no decrease in the $T/T_p$ ratio until the 7th year after which it fell rapidly, leveling off at the 10th year. Figure 3 shows the average salt concentration building up to about 260 meq/l at the 10th year. When $T/T_p$ decreases, the transpiration also decreases. After the 10th year of cropping, ET had decreased by 15 cm which was only 9 cm above the water added. The difference between the water added and ET came from soil water storage and flow upward from the water table. Note that the particular results computed for a simulated run of years, shown in Figure 3, are highly dependent on the particular situation. If a crop with more shallow roots had been used, an entirely different situation would have resulted.
Figure 2. Salt concentration as a function of depth, irrigation and rain at the end of one season. Corn was assumed to have deep root distribution and oats was assumed to have shallow root distribution.
One of the purposes of the computation shown in Figure 3 was to see how these results compared with the data of Table 2 where different initial salt concentrations were used to simulate salt buildup. For the same irrigation schedule, the data of Table 2 indicate a $T/T_p$ ratio of 0.90 for an initial salt concentration of 200 meq/l and ending up with an average concentration of 296 meq/l. The data of Figure 4 indicate essentially the same ratio of $T/T_p$, although the salt concentration at the end of the year is not as high as that shown in Table 2. Thus, using a uniform salt concentration profile as the initial condition gives the same result as using the profile existing at the end of the previous crop years. In fact, the uniform profile is probably more accurate since the upward and downward diffusion and mass flow due to evaporation and drainage tends to equalize the salt in the profile over the winter.

**Figure 3.** Relative transpiration and average salt concentration for corn with deep irrigated at a rate of 24.4 cm/year as influenced by year.
The single point values, relating water added to the $T/T_p$, are somewhat unrealistic in a real field situation because water is not distributed uniformly. Even in the best system there are parts of the field that receive more water than others. To account for this, engineers have defined a uniformity coefficient $Cu$ as follows:

$$ Cu = 1 - \frac{D}{M} $$

where $M$ is the average irrigation rate and $D$ is the average deviation (sign ignored) about the average rate. It should be noted that if $Cu = 1.0$, water application would be completely uniform. To add this factor to the computations, it was necessary to assume a distribution pattern and the extent of coverage that might apply for some mean water application rate. From the distribution pattern, a new value of $T/T_p$ results from integrating $T/T_p$ over the water distribution pattern. This also provides salt outflow information. These data were calculated assuming a uniformity of 1.0 for all of the data presented up to this point. Considering nonuniform coverage, the relationship of $T/T_p$ to average water added by irrigation can then be constructed. These data are shown in Tables 5, 6, and 7 for the three crops for three different uniformities. The amount of salt outflow is also shown. These tables show essentially the same ratio of $T/T_p$ for all uniformities provided the water application is insufficient to allow any drainage (and thus salt outflow). However, once the irrigation rate is high enough to result in some drainage, the ratio of $T/T_p$ decreases as the uniformity decreases. Thus, for alfalfa $T/T_p$ is 1.0, 0.98, and 0.90 for a $Cu$ of 1.0, 0.88, and 0.42, respectively (20 meq/l initial salt concentration). This ratio variation results from poor uniformity due to irrigation greater or less than ET. The same result is also shown in Table 8 where the average water application is greater than ET. For this situation, some part of the field received water at less than ET resulting in $T/T_p$ less than 1.0.

These results point out another situation of great practical importance involving some present concepts of low leaching ratios. If water distribution is nonuniform and the average leaching ratio is low,
then there will be part of the field which is not leached at all and salts will accumulate. This could be a serious problem when the same uniformity distribution pattern prevails year after year. A ten-year simulation of this effect shows a salt buildup in a portion of the wetted area getting less water than ET and a consequent decrease in $T/T_p$ (Table 8). Where irrigation is greater than ET, essentially steady state conditions prevailed.

Table 5. Relative yield of corn, equal to $T/T_p$ pot, as influenced by three different values of Cu, water applied, and initial salt concentration.

<table>
<thead>
<tr>
<th>Irrig. &amp; Rain</th>
<th>Initial Salt</th>
<th>T/T pot</th>
<th>Salt outflow</th>
<th>T/T pot</th>
<th>Salt outflow</th>
<th>T/T pot</th>
<th>Salt outflow</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>meq/l</td>
<td>meq/cm²</td>
<td></td>
<td>meq/l</td>
<td>meq/cm²</td>
<td>meq/l</td>
<td>meq/cm²</td>
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<td>0.89</td>
<td>0</td>
<td>0.89</td>
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<tr>
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<td>0</td>
</tr>
<tr>
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Table 6. Relative yield of alfalfa, equal to \( T/T \) pot, as influenced by three different values of \( Cu \), water applied, and initial salt concentration.

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<th>Initial Salt</th>
<th>( T/T ) pot Cu = 1</th>
<th>Salt outflow ( Cu = 1 )</th>
<th>( T/T ) pot Cu = 0.88</th>
<th>Salt outflow ( Cu = 0.88 )</th>
<th>( T/T ) pot Cu = 0.42</th>
<th>Salt outflow ( Cu = 0.42 )</th>
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<td>meq/cm²</td>
<td>meq/cm²</td>
<td>meq/cm²</td>
<td>meq/cm²</td>
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</tr>
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<td>0.58 0.58</td>
<td>0.58 0.58</td>
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<td>0.66 0.66</td>
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<td>0.66 0.66</td>
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<tr>
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<td>0.98 0.98</td>
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</tr>
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<tr>
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<td>1.0 1882.0</td>
<td>1.0 1882.0</td>
<td>1.0 1882.0</td>
</tr>
</tbody>
</table>

The economic comparisons

The physical relationships discussed above are the basic data for the economic analysis. From the physical data, the relevant information on growing corn silage, oats, or alfalfa hay was accumulated. Decision options which included water application by sprinkler or by flooding at rates (from irrigation and rain) of 10.3, 15.0, 22.0, 40.8, 56.4, and 66.7 centimeters for each of the crops were utilized.

Limits on resource availability (the \( B_i \) or right-hand-side values used in the linear programming study include the quantities of each of
Table 7. Relative yield of oats, equal to T/T pot, as influenced by three different values of Cu, water applied, and initial salt concentration.

<table>
<thead>
<tr>
<th>Irrig. &amp; Rain</th>
<th>Initial Salt</th>
<th>T/T pot Cu = 1</th>
<th>Salt outflow</th>
<th>T/T pot Cu = 0.88</th>
<th>Salt outflow</th>
<th>T/T pot Cu = 0.42</th>
<th>Salt outflow</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>cm</td>
<td>meq/l</td>
<td>meq/cm²</td>
<td></td>
<td>meq/cm²</td>
<td></td>
<td>meq/cm²</td>
</tr>
<tr>
<td>5.6</td>
<td>20</td>
<td>0.29</td>
<td>0</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>5.6</td>
<td>50</td>
<td>0.28</td>
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<td>--</td>
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<td>--</td>
</tr>
<tr>
<td>5.6</td>
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<td>--</td>
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<tr>
<td>10.3</td>
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<td>0.37</td>
<td>0</td>
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</tr>
<tr>
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<td>0</td>
<td>0.36</td>
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<td>0.36</td>
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<tr>
<td>10.3</td>
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<td>0.24</td>
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<td>0.24</td>
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<tr>
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<td>0.46</td>
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<td>0</td>
<td>0.45</td>
<td>0</td>
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<tr>
<td>15.0</td>
<td>50</td>
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<td>0</td>
<td>0.43</td>
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<td>0.43</td>
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<td>0.32</td>
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<tr>
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<td>0</td>
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<td>22.0</td>
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<td>0.59</td>
<td>0</td>
<td>0.59</td>
<td>0</td>
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<tr>
<td>22.0</td>
<td>200</td>
<td>0.46</td>
<td>0</td>
<td>0.47</td>
<td>0</td>
<td>0.48</td>
<td>0</td>
</tr>
<tr>
<td>40.8</td>
<td>20</td>
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<td>0</td>
<td>0.79</td>
<td>84</td>
</tr>
<tr>
<td>40.8</td>
<td>50</td>
<td>0.88</td>
<td>0</td>
<td>0.87</td>
<td>0</td>
<td>0.78</td>
<td>209,</td>
</tr>
<tr>
<td>40.8</td>
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<td>0.80</td>
<td>0</td>
<td>0.78</td>
<td>17</td>
<td>0.71</td>
<td>818,</td>
</tr>
<tr>
<td>56.4</td>
<td>20</td>
<td>0.97</td>
<td>26</td>
<td>0.97</td>
<td>63</td>
<td>0.84</td>
<td>365,</td>
</tr>
<tr>
<td>56.4</td>
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<td>0.98</td>
<td>66</td>
<td>0.97</td>
<td>157</td>
<td>0.84</td>
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<tr>
<td>56.4</td>
<td>200</td>
<td>0.93</td>
<td>490</td>
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<tr>
<td>66.7</td>
<td>20</td>
<td>0.99</td>
<td>198</td>
<td>0.99</td>
<td>225</td>
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<td>780,</td>
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<td>66.7</td>
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<td>0.99</td>
<td>563.</td>
<td>0.87</td>
<td>1967,</td>
</tr>
<tr>
<td>66.7</td>
<td>200</td>
<td>0.99</td>
<td>1975</td>
<td>0.98</td>
<td>2178.</td>
<td>0.82</td>
<td>6492,</td>
</tr>
</tbody>
</table>

three land classes based on the beginning salinity levels in the soil profile. It was assumed that the farm under study had 10 acres with each of the three soil characteristics (20, 50, and 200 meq/l) described earlier. Also, an unlimited quantity of salt outflow was allowed in the drainage water (which level was sequentially reduced to determine the loss in profitability to the farm from restricting salt flow into the
Table 8. Relation of time and irrigation rate, for Cu = 0.42 (square) to relative transpiration, T/Tp, and average salt content Sf at different positions with the uniformity pattern with beginning soil salinity at 20 meq/l.

<table>
<thead>
<tr>
<th>Relative area</th>
<th>0.20</th>
<th>0.20</th>
<th>0.20</th>
<th>0.20</th>
<th>0.20</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relative I rate</td>
<td>.10</td>
<td>.30</td>
<td>.50</td>
<td>.70</td>
<td>.90</td>
</tr>
<tr>
<td>Year</td>
<td>T/Tp Sf</td>
<td>T/Tp Sf</td>
<td>T/Tp Sf</td>
<td>T/Tp Sf</td>
<td>T/Tp Sf</td>
</tr>
<tr>
<td>1</td>
<td>.45 33</td>
<td>.75 30</td>
<td>.96 24</td>
<td>.96 24</td>
<td>1.0 20</td>
</tr>
<tr>
<td>2</td>
<td>.44 53</td>
<td>.72 37</td>
<td>.96 28</td>
<td>.96 26</td>
<td>1.0 21</td>
</tr>
<tr>
<td>3</td>
<td>.43 81</td>
<td>.70 43</td>
<td>.96 32</td>
<td>.96 28</td>
<td>1.0 21</td>
</tr>
<tr>
<td>4</td>
<td>.42 117</td>
<td>.69 47</td>
<td>.96 35</td>
<td>.96 29</td>
<td>1.0 21</td>
</tr>
<tr>
<td>5</td>
<td>.39 162</td>
<td>.68 50</td>
<td>.96 39</td>
<td>.96 29</td>
<td>1.0 21</td>
</tr>
<tr>
<td>6</td>
<td>.35 208</td>
<td>.67 53</td>
<td>.96 42</td>
<td>.96 29</td>
<td>1.0 21</td>
</tr>
<tr>
<td>7</td>
<td>.30 249</td>
<td>.67 56</td>
<td>.96 45</td>
<td>.96 29</td>
<td>1.0 21</td>
</tr>
<tr>
<td>8</td>
<td>.26 280</td>
<td>.66 58</td>
<td>.96 49</td>
<td>.96 29</td>
<td>1.0 21</td>
</tr>
<tr>
<td>9</td>
<td>.22 298</td>
<td>.66 60</td>
<td>.96 52</td>
<td>.96 29</td>
<td>1.0 21</td>
</tr>
<tr>
<td>10</td>
<td>.20 306</td>
<td>.65 61</td>
<td>.96 55</td>
<td>.96 29</td>
<td>1.0 21</td>
</tr>
</tbody>
</table>

Drains and streams). There were also constraints to force growing of crops in rotation such as to provide for nurse crops for new seedings of alfalfa and limits on corn production for disease control.

The net profit values for each unit of production were based on approximate current prices for products and the costs of various operations. Yields were estimated using the 1971 data for the farm as a base and the relative yields predicted in the physical model to give specific values for the rates of water applied as influenced by the initial salt concentration in the soil as shown in Tables 9 and 10.

The profit function is based upon a price for alfalfa of $45/ton; for corn silage, $13/ton; and for oats, $1.60/bushel. These represent approximately the current prices, but are adjusted somewhat to a normal long-run relationship to each other. The cost of raising crops was computed as shown in Table 11.
Table 9. Predicted yield of crops under sprinkler irrigation by initial salt content of soil, by water application rates.\textsuperscript{a}

<table>
<thead>
<tr>
<th>Initial Salt Content of Soil</th>
<th>Water Level (Irrigation plus rain)</th>
<th>Crop Yield</th>
<th>Crop Yield</th>
<th>Crop Yield</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Alfalfa (medium roots)</td>
<td>Oats (shallow roots)</td>
<td>Corn Silage (deep roots)</td>
</tr>
<tr>
<td></td>
<td>centimeters</td>
<td>tons</td>
<td>bushels</td>
<td>tons</td>
</tr>
<tr>
<td>20 Meq/L</td>
<td>10.3</td>
<td>3.3</td>
<td>34.0</td>
<td>20.5</td>
</tr>
<tr>
<td></td>
<td>40.8</td>
<td>5.3</td>
<td>80.1</td>
<td>22.8</td>
</tr>
<tr>
<td>50 Meq/L</td>
<td>10.3</td>
<td>3.2</td>
<td>32.8</td>
<td>19.7</td>
</tr>
<tr>
<td></td>
<td>40.8</td>
<td>5.3</td>
<td>79.8</td>
<td>22.8</td>
</tr>
<tr>
<td>200 Meq/L</td>
<td>10.3</td>
<td>2.2</td>
<td>22.2</td>
<td>12.9</td>
</tr>
<tr>
<td></td>
<td>40.8</td>
<td>4.9</td>
<td>71.9</td>
<td>22.8</td>
</tr>
</tbody>
</table>

\textsuperscript{a}Based on Tables 5, 6, and 7, above, and assuming a coefficient of uniformity of application (CU) = 0.88.

Single year analysis

Two main sets of results were desired in order to draw conclusions. These were the set of production activities that would maximize farm profit at each level of salt outflow and the loss in income from
Table 10. Predicted yield of crops under flood irrigation by initial salt content of soil, by water application rates.\(^a\)

<table>
<thead>
<tr>
<th>Initial Salt Content of Soil</th>
<th>Water Level (Irrigation plus rain)</th>
<th>Crop Yield</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Alfalfa (medium roots)</td>
<td>Oats (shallow roots)</td>
</tr>
<tr>
<td></td>
<td>centimeters</td>
<td>tons</td>
</tr>
<tr>
<td>20 Meq/L</td>
<td>40.8</td>
<td>4.9</td>
</tr>
<tr>
<td></td>
<td>56.4</td>
<td>5.0</td>
</tr>
<tr>
<td></td>
<td>66.7</td>
<td>5.1</td>
</tr>
<tr>
<td>50 Meq/L</td>
<td>40.8</td>
<td>4.9</td>
</tr>
<tr>
<td></td>
<td>56.4</td>
<td>5.0</td>
</tr>
<tr>
<td></td>
<td>66.7</td>
<td>5.1</td>
</tr>
<tr>
<td>200 Meq/L</td>
<td>40.8</td>
<td>4.5</td>
</tr>
<tr>
<td></td>
<td>56.4</td>
<td>4.7</td>
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<tr>
<td></td>
<td>66.7</td>
<td>4.9</td>
</tr>
</tbody>
</table>

\(^a\)Based on Tables 5, 6, and 7, above, and assuming a coefficient of uniformity of application (CU) = 0.42.

not allowing an incremental ton of salt to flow out. The latter may also be characterized in its mirror image, the value to the farm of allowing an additional ton of salt outflow.

A number of different situations were modeled to determine the effects of irrigation method and rate of application, and restrictions on the crop combinations.

Situation 1 - Unrestricted corn in the rotation, corn roots deep, alfalfa roots shallow, sprinkler or flood irrigation. Without any constraint on corn in the rotation, the production activities in the optimal production pattern included nothing other than corn. In Figure 5, the most profitable production activities are summarized. Note that the tons of salt outflow for the entire 30 acres is on the scale at the bottom of the figure. The set of crops which is optimal is plotted for the 10
Table 11. Cost components of crop production by crop and by method of water application.

<table>
<thead>
<tr>
<th>Crop</th>
<th>Fixed Cost</th>
<th>Growing Cost</th>
<th>Irrigation Costs</th>
<th>Water Level (Irrig. plus rain)</th>
<th>Sprinkler Construction Cost</th>
<th>Energy Cost</th>
<th>Flood Cost</th>
<th>Harvest Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$ per acre</td>
<td>$ per acre</td>
<td>cm</td>
<td>$ per acre</td>
<td>$ per acre</td>
<td>$ per acre</td>
<td>$ per acre</td>
<td>$ per acre</td>
</tr>
<tr>
<td>Alfalfa Hay</td>
<td>13.65</td>
<td>27.09</td>
<td>10.3</td>
<td>1.22</td>
<td>15.0</td>
<td>1.65</td>
<td>22.0</td>
<td>3.30</td>
</tr>
<tr>
<td>Oats</td>
<td>13.65</td>
<td>58.11</td>
<td>10.3</td>
<td>1.22</td>
<td>15.0</td>
<td>1.65</td>
<td>22.0</td>
<td>3.30</td>
</tr>
<tr>
<td>Corn Silage</td>
<td>13.65</td>
<td>70.39</td>
<td>10.3</td>
<td>1.22</td>
<td>15.0</td>
<td>1.65</td>
<td>22.0</td>
<td>3.30</td>
</tr>
</tbody>
</table>

acre units by soil type (where initial soil salt is at the high, medium, or low level) for each level of salt outflow. For instance, at a level of 60 tons of salt outflow, the model indicates that for the low soil salt condition, the entire 10 acres should be in corn irrigated at the 5th level (next to highest) by flooding. For the medium salt condition, there should be about 4 acres of corn at the 4th level of water application.
tion. On the saltiest land, there should be 10 acres of corn irrigated at the 5th level by sprinkling.

In meeting the requirement for low salt outflow, sprinkler systems and low rates of water application were required in the mode. As the allowable salt outflow was increased, the irrigation rates were increased and the method of irrigation changed to flooding. Net profit increased by about $900 (or $30 per acre) as the salt outflow constraint was relaxed. Almost all of this profit increase occurred in the first 20-ton increment. Only about $100 of additional profit (Figure 6) for

Figure 5. Optimal cropping and irrigating pattern for high, medium, and low initial soil salt conditions where corn roots are deep and alfalfa shallow and either flooding or sprinkling is allowed as an irrigation method.
the 30 acre block of land could be attained beyond this first 20-ton increment. In a practical management situation, all 30 acres would be irrigated by sprinkling or by flooding, rather than a combination of systems.

There are two main reasons for obtaining these results. First, it was assumed that corn was a deep-rooting plant so that this crop was profitable at low levels of irrigation, since in the physical model the corn obtained considerable water from deep soil moisture or underground supply. In a static one-year analysis with a light application of water, there would be no outflow of salts, but there would be an accumulation in the soil profile. Second, corn was the most profitable crop assuming that yields can be maintained.

In Figure 7, the value to the farm of an additional ton of salt outflow as a function of salt outflow is shown. Note that the cost to the farm of reducing the outflow of salt (or value for letting an additional ton flow out) is very low compared to any possible costs of removal by desalination or other methods.

**Situation 2 - Corn restricted to one-half of the acreage, corn roots shallow, alfalfa roots deep, sprinkle or flood irrigation.** This situation was tested for several reasons. Corn could probably not be grown exclusively for several years due to varied needs for livestock feed, disease and fertility problems on the land, and grower preference for multiple crops. Also, the depth of corn roots may be somewhat shallower than the perennial alfalfa crop. The data which indicated corn was deep-rooted and alfalfa somewhat shallower were from separate experimental plots that may not be appropriate for the area of this study.

Under these assumptions, the cropping patterns over the range of salt outflow are as shown in Figure 8. Alfalfa would be profitable and the required nurse crop would accompany low salt outflows since alfalfa roots are assumed deep where more soil moisture or groundwater can be obtained, and heavy water application is not required for reasonably good yields. Low levels of irrigation are again optimal.
Figure 6. Net revenue by amount of salt outflow for the 30 acres as shown in Figure 4a.

Figure 7. Shadow price or value of an additional ton of salt outflow for the 30 acres as shown in Figure 4.
Figure 8. Optimal cropping and irrigating pattern for high, medium, and low initial soil salt conditions where corn roots are shallow, and alfalfa deep and either flooding or sprinkling is allowed as an irrigation method.
at low levels of salt outflow. Higher levels of water application are most profitable for high salt outflow. Note that compared to the previous situation in which corn was unrestricted and the corn roots were deeper than alfalfa a higher total salt outflow is more profitable than if there are no restrictions on these factors. This higher level of salt outflow is caused by the requirement for a mix of crops and by shallow corn roots which do not tap the underground water supply. As before, the most restrictive constraints in salt outflow are the most costly to the farm plan. Very high levels of additional salt outflow add little to the profit (Figures 9 and 10).

Situation 3 - Corn restricted to one-half of the acreage, corn roots shallow, alfalfa deep, flood irrigation only. Under this assumption (flooding only), a relatively small amount of corn would be produced except at high levels of water application and for high levels of salt outflow (Figure 11). This result is due to alfalfa being able to obtain water from underground sources so that fairly good yields can be obtained without high levels of salt output resulting from the leaching due to heavy water application.

Note that for a given total level of salt outflow the water application levels on alfalfa are largest on the low salt soil and then lower successively to the high salt soil and the land remains idle at low levels of permissible salt outflow because it is unprofitable to operate without applying water. The system cannot meet the tight constraint on salt if all land is used, since flood irrigation is possible only at the three highest levels of water application.

The highest level of salt output is much higher, nearly 100 tons, than with the previous situations in which sprinkling is one of the options. The highest penalties for restricting salt output, as usual, are where the salt constraint is most restrictive as shown in Figures 12 and 13. But, once the constraint is relaxed to more than three tons per acre, the value is less than $1 per ton.
Figure 9. Net revenue by amount of salt outflow for the 30 acres as shown in Figure 8.

Figure 10. Shadow price or value of an additional ton of salt outflow for the 30 acres as shown in Figure 8.
Figure 11. Optimal cropping and irrigating pattern for high, medium, and low initial soil salt conditions where corn roots are shallow and alfalfa deep where flood irrigation only is allowed.
Figure 12. Net revenue by amount of salt outflow for the 30 acres as shown in Figure 10.

Figure 13. Shadow price or value of an additional ton of salt outflow for the 30 acres as shown in Figure 10.
Situation 4 - Corn restricted to one-half of the acreage, corn roots shallow, alfalfa deep, sprinkler irrigation only. Under sprinkler irrigation, the most noticeable difference is that corn is produced to the maximum allowed in the rotation at all levels of salt outflow (Figure 14). As usual, the irrigation rate increases as the allowable salt outflow is increased.

In Figures 15 and 16, it can be seen that as salt outflow reaches one ton per acre, there can be very little additional profit by applying higher levels of water with the resultant higher salt outflow.

Comparison and evaluation of situations. In comparing the different situations studied, it is clear that the crop which has the assumed deep roots is generally more profitable. As mentioned, this results from extraction of water from underground sources alleviating the demand for the heavy applications of water and the salt leaching that accompanies heavy watering. This net upward flow leads to salt accumulation with time so these one year results do not apply for a period of years where net leaching does not occur. In other situations in which groundwater would not be available, such a result would not be expected. Without constraints on salt outflow, it appears that flood irrigation is most profitable to the farm. The advantages of better yields and the lower water use cost were not sufficient to make sprinkling generally profitable. It was found that net profit at the maximum was about $8 per acre less ($250 for the 30 acres) if the irrigation system was constrained to sprinkling. If the farm was constrained to one ton salt output per acre, sprinkling would be more profitable by a few hundred dollars. At 2 tons per acre, sprinkling would be more profitable than flooding by about $300 ($10 per acre). This difference depends on leaving some land idle under flooding to meet the restriction in addition to the yield advantages and lower water costs due to sprinkling.

In evaluating the shadow prices of salt output (value to the farm of an additional ton of salt output), it is clear that the first ton or two of salt per acre under any assumptions are most critical. It is not
Figure 14. Optimal cropping and irrigating pattern for high, medium and low initial soil salt conditions where corn roots are shallow and alfalfa deep where sprinkler irrigation only is allowed.
Figure 15. Net revenue by amount of salt outflow for the 30 acres as shown in Figure 13.

Figure 16. Shadow price or value of an additional ton of salt outflow for the 30 acres as shown in Figure 13.
known just how much salt is presently coming from cultivation of lands of this type, but the amount is likely somewhat higher than one or two tons. Therefore, it may well be possible under any set of management objectives to reduce salt outflow considerably with minimum cost (usually less than $1 per ton). This value surely is much less than other cost estimates of salt reduction in the Colorado River. The Bureau of Reclamation currently estimates other control measures at $9 to $30 per ton of salt (U.S. Bureau of Reclamation, 1974). But, these conclusions are limited to a single year in which soil salinity buildup is not accounted for.

**Multi-year analysis**

A multi-year analysis of management practices was developed by using the final conditions of the previous year for the initial soil salinity conditions of the current year subject to the assumptions of the physical model. Four levels of water application were used in the modeling. The initial soil salinity figures of 20, 50, and 200 meq/l were prime data for this analysis. The final soil salinity for each year, salt outflow, and yields depended heavily on the beginning soil salinity as well as on water application and other factors.

**Initial soil salinity.** In the following discussion, we present the results of initial soil salinity and water application level combinations. Results will be presented as final soil salinity, salt outflow, and net revenue per acre. A brief commentary on cropping patterns will also be included.

**Initial soil salinity at 20 meq/l.** A number of somewhat expected results occurred in the multi-year simulation of soil salinity (Figure 17). First, the lowest level of water application (20 cm) resulted in a salt buildup in the soil profile. Second, this buildup tended to taper off in the last few years of the six year period. This was caused by the profit optimizing model letting a few acres remain idle and heavier water application being available for the remainder. This heavier
Figure 17. Multi-year final soil salinity comparisons for four average rates of water application at initial soil salinity of 20 meq/l.
application reduced the salt in the profile on part of the acreage and also for the average. We would expect to find farmers doing exactly this if water was restricted for salt control purposes. Third, the heaviest water application rates resulted in no particular change in soil salinity over time. Note that the water application rates were an average for the several acres of soil with this initial condition. Some, depending on the crop, may have received more and some less or even none as noted above if some land were left idle. This resulted in the slightly erratic patterns shown especially for the intermediate water application levels.

The simulation of salt outflow over time is shown in Figure 18. As might be expected, the heavier water applications flush the salt through the soil. Lighter applications of water lead to salt buildup to a severe degree.

Alfalfa with the necessary nurse crop of oats dominates the cropping pattern where minimum water application is allowed. Application is by sprinkler. The reason is the assumed deep rooting of alfalfa which enables it to obtain additional water from the groundwater. Corn with flood irrigation dominates the high level water application situation.

The net revenue (gross income less variable costs) comparisons for the multi-year period are shown in Figure 19. At heavier rates of water application the net revenue is maintained, but at lower rates of water application the revenue declines sharply over time because of higher soil salinity and falling yields.

Initial soil salinity at 50 meq/l. Again, several comparisons have been made with initial soil salinity at this higher level. The ending soil salinity over a period of years is in much the same pattern as shown earlier. The heaviest average water application rate results in a slight decline in soil salinity. See Figure 20. Salt outflow as shown in Figure 21 is fairly stable at the lowest rate of water application, is higher and increases at intermediate rates of application and is
Figure 18. Multi-year salt outflow comparisons for four average rates of water application at initial soil salinity of 20 meq/l.
Figure 19. Net revenue comparisons for four rates of water application at initial soil salinity of 20 meq/l.
Figure 20. Multi-year final soil salinity comparisons for four average rates of water application at initial soil salinity of 50 meq/l.
Figure 21. Multi-year salt outflow comparisons for four average rates of water application at initial soil salinity of 50 meq/l.
quite high but declines as leaching occurs at the highest rate of water application. Net revenue follows much the same pattern as with 20 meq/l soil salinity for high rates of water application, but is more depressed at low water application (Figure 22). Cropping pattern is nearly identical to the situation with soil salinity at 20 meq/l.

**Initial soil salinity at 200 meq/l.** Changes in soil salinity over time are shown in Figure 23. The two heaviest water application rates result in declines in soil salinity over time. Low water application results in an ever greater buildup.

Salt outflow ranges up to high amounts of 15 to 16 tons per year for heavy water application, but is fairly minimal for light applications of water since little or no water goes through the profile. Net revenue is depressed by one-third or more because of the saline conditions, but improves slightly in cases where leaching is accomplished.

**Policy Implications of the Study**

This study although done for a specific site in Eastern Utah indicates a number of management possibilities for irrigation water may be quite useful in reducing Colorado River water salinity. Assume that the range of current average estimates of salinity outputs from irrigated agriculture are 1.5 to 3.0 tons per acre. Then, it appears that costs of reducing this level to one ton per acre or a little less may be fairly minimal. This is based on the single year analysis, however, and may lead to further increases in soil salinity and either greater salt outflow in the future or even greater losses in income from attempting to reduce the salinity. It is readily apparent that a zero discharge standard is at best immensely costly or totally impossible. Moderate rates of improvement may be possible with limited cost to producers. The multi-year study showed that low rates of water application cause excessive salt buildup in the soil profile and reduce net revenue very significantly. High rates of water application, of course, alleviate this problem but cause continued large salt outflow.
Figure 22. Net revenue comparisons for four rates of water application at initial soil salinity of 50 meq/l.
Figure 23. Multi-year salt outflow comparisons for four average rates of water application at initial soil salinity of 200 meq/l.


THE RELATIONSHIPS BETWEEN HYDROLOGIC MODELING AND THE DATA BASE: SOME OBSERVATIONS AND COMMENTS

by

Richard H. Hawkins*

Introduction

The most obvious response to the title of this paper is simply "... there is one!" Data does indeed interact with models in many ways. The involvement becomes apparent early in modeling experiences, and is usually intellectually cubby-holed and taken as a curious and interesting sidelight thereafter. The joy of modeling is in the conceptualization, calibration, and application stages: few modelers relish on the tedious and frustrating work of data preparation, extension, and rectification, however necessary the task.

Thus, the threat of elaboration on the data base theme is not likely to stir much excitement. Much of what is to follow is derived from experiences with hydrologic and river models, although application can no doubt also be made to a wider scope of ecological situations.

Data - What Is It?

It is reasonably difficult to arrive at a good definition of data that will meet all our preconceived notions and usages, and still be scientifically sound. Unfortunately, the word itself is subject to grammatical intimidation: it is a plural form of datum, and editors, etc., delight in enforcing compliance with its Latin roots. The Webster's definition is wide of our experiences, so the following is offered for our purposes: Data; any measured or estimated numerical value which we can use to

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draw inferences. We might think of data as what we see through windows or peepholes in the system. Thus, in hydrology work data includes precipitation, temperature, streamflow, water chemistry, soil moisture, and similarly measured items. Whether such concepts as field capacity, infiltration rates, interception and surface storage, and groundwater recession rates are data will be a point of further discussion.

Hydrologic modeling data can be described or discussed in terms of a series of attributes which circumscribe its utility. These are 1) topic (what's being measured); 2) accuracy (how representative or biased); 3) precision (how reproducible or noiseless); 4) Areal and temporal resolution (how fine or coarse a representation is it?); 5) continuity; and 6) synchronization (is it temporally in phase with concurrent measurements?). Naturally these vary: streamflow measurements may be imprecise but accurate, concurrent rainfall measurements may be (and usually are) unsynchronized, of sparse resolution, and biased (about 10 percent under with respect to ground rainfall). Soil moisture measurements are usually quite precise, but of questionable accuracy, and certainly a doubtful representative of areal conditions. Water quality data may be temporally transient and inaccurate as well.

Like almost any other commodity (such as water), data also has value in terms of its quantity, quality, and timeliness. The usual gut concern is for a sufficient quantity of data. The quality item is easily ignored or shrugged off if acknowledged at all. The timeliness consideration speaks for itself, as data must be available when it is needed, and for the period under study. The mystery of data acceptability resides in its quality dimension, i.e., accuracy, precision, resolution, and synchronization. Additional quality factors arise when filling in missing values, extending point samples to a watershed basis, and attempting to satisfy quantity requirements.

Modeling - A Data's Eye View

As we know the topic, hydrologic models are applications of systems analysis taken in a hydrologic vein: inputs are the hydrologic
driving variables of precipitation, and radiation (as represented by temperature), and the outputs are streamflow attributes and evapotranspiration. In related hydrologic situations the inputs may include streamflow from an upstream unit. Quality considerations may include such as salt masses, heat contents, and dissolved gases. As an example, a hydrologic model in flow chart form is shown in Figure 1.

The "system" which transforms inputs to outputs with such interesting results is the usual item of attention. As in Figure 1, it is the watershed itself, and it is assigned a structure (an internal plumbing array) in accordance to our preconceived notions and model objectives. The model structure draws on a series of coefficients (value sizes and openings, storage capacities, and thresholds) which we sometimes dignify as "parameters" and equate to actual field concepts, such as field capacity, soil moisture indexes, etc. It is important that we respect and understand what we have created without deifying it: insofar as we create models based on our understanding, we are victims of our own delusions.

A useful concept in visualizing what models represent has been promoted by Crawford (1). He describes a pure representation of reality as a "white box", i.e., all processes, structures and coefficients defined with certainty; a completely true image of nature in detail. On the other extreme, a pragmatic empirical input-output transformation without regards to the causative mechanisms is described as a "black box." The "black box" may get the job done, but it gives little or no insight to the causative factors or the system itself. With the black box, manipulation of the model and/or input beyond the original conditions is risky, and carries all the dangers of extrapolating regressions. Hydrologic models are something in between the two extremes, and may be thought of as "gray boxes." The degree of grayness varies with model detail, and its dependence on the quantity and quality of data required to calibrate and operate the model. The whiter the model, the more it demands in terms of data quantity and quality. A relatively white model has a gluttonous appetite for input information corroborating field measurements.
Figure 1. A flow chart of a simple deterministic lumped rainstorm-runoff model. Note that it uses a series of coefficients (parameters?) which must be determined by calibration with field data from the watershed under study.
Modeling proceeds by a series of steps, either formal or casual. Even if all are not consciously followed, some are either tacitly assumed or purposely ignored. A chart showing the basic skeleton of the process is given in Figure 2. It will be used as a basis for discussion the role of data. It is instructive to begin at the end (output) and proceed towards the beginning. The end result (step 4) is an application of a trustworthy working model to a problem to give otherwise unobtainable solutions. For example, a hydrologic model may be used to estimate the hydrologic effects of a planned land use or condition. To get to this point, however, we need the trustworthy model: this calls for model verification (step 3), which is composed of calibration (3a) and testing (3b). In calibration, the input and output from historical events (data!) are used, comparing model output to reality to arrive at an acceptable model coefficient set; and sometimes to amend model structure. When model output most closely matches observed output, a state of "calibration" is said to exist. There are some statistical questions which arise in this regard, but which are not covered here. However, for assurance, the model is usually run on some independent input (not used in the calibration), and the validation of the parameter set and structure observed through the degree of matching. Happily this testing is usually successful. However at this point it is possible to return to the conceptualization step and restructure the model and/or seek new data. This option (not shown in Figure 2) is exercised by the modeler when results are clearly unsatisfactory or missing processes are suspected.

The model verification step requires both a conceptual model (in computer form) and data (step 2). As shown in Figure 2, these tasks in themselves interact: the model performance is limited by the data and yet the model dictates what data are needed. The necessary compromises and requirements are negotiated considering the study objectives and certainly by budget considerations. Parenthetically, the objectives also influence the choice of the objective function used to evaluate goodness-of-fit in the calibration.
Figure 2. A flow diagram of a frequently used hydrologic modeling procedure. Note that data considerations enter into all steps either directly or indirectly.
The data preparation stage is a long, tedious, and unglamorous task, often relegated to underlings. It is, however, absolutely necessary, and usually deserved more attention than it gets. There is little rewarding or enjoyable about doing a good job in extending records, tracking down charts from discontinued stations, filling in data gaps, etc. The task has a definite service flavor; the results are an otherwise uninteresting series of tables of prepared, laundered, rectified, and believable numbers. The calibration stage hangs on faith in this data.

For the verification stage, the data used should, at the very least, be continuous (no missing items), and hopefully of an acceptable areal and temporal resolution. With no knowledge to the contrary it is assumed to be both accurate and precise, and synchronized as well. Those who install instruments, extract readings, reduce records, and work with data should appreciate the folly of the above specifications. Often periods of record are missing, and information must be either extrapolated, or interpolated to the area of interest through regressions, lapsing (with meteorological data), averaging, or similar techniques. In short duration event modeling, the important time characteristic may be less than the width of a pen trace on the records, or less than the time differences on gage clocks. Considering all the possible sources of error and confusion it is surprising that some models do perform consistently. Data errors manifest themselves in the model coefficients in the calibration. However, some models are apparently "robust" enough to deal with many data shortcomings and still perform adequately.

An Example: West Branch of Chicken Creek

In order to illustrate data-model interactions, a specific case will be used. While a rather elementary example, the general ideas presented can be used as a basis for extrapolation and grounds for extension to more baroque situations.

The West Branch of Chicken Creek is a 217 acre (87.8 ha) small watershed in Utah's Wasatch Front, east of Farmington, Utah, about
20 miles (35 km) northwest of Salt Lake City. It receives about 45 inches (1140 mm) of precipitation annually, mostly in the form of snow, which results in about 19 inches (480 mm) of runoff. Occasional summer thunderstorms produce short duration hydrographs of low volume, but sometimes intense rates of runoff. The watershed ranges in elevation from 7,550 ft. (230 m) to 8,396 ft. (2,559 m). It is a part of the U.S. Forest Service's Davis County Experimental Watershed (DCEW), historically notorious for a classical sequence of land abuse, flooding, and debris production. Instrumentation includes a recording rain gage network and a flume at the watershed mouth. A summary paper on Chicken Creek has been prepared by Johnston and Doty (4).

The watershed was used as the topic of study for class exercises in a watershed modeling course at Utah State University. Thus, although in this case the objectives per se may be more diffuse than usual, an operational objective may be stated as the prediction of hydrographs from summer rain storms, and the hydrological evaluation of design land use changes.

A model to meet these needs was written, keeping in mind any possible data limitation. The model is drawn from a direct tank analogy of watershed hydrology, as just presented by Dawdy and O'Donnell (2), and is similar to the digital storm runoff model used by Dawdy, Lichty, and Bergeman (3). Although such similarities smack of plagiarism, adherence to the conceived realities of the hydrologic cycle inevitably draws hydrology models towards a common structure. The model in flow chart form is shown in Figure 1.

There are several features of the model which should be detailed. First, there is no snowmelt routine, insofar as the model is intended to deal only with summer rainstorms. This is a major simplifying item, as snowmelt hydrology occupies a major portion of most "full feature" models. Secondly, evapotranspiration (ET) is ignored, and plays no role in the model. This is done on the following rationale: (a) it is usually small during a storm duration, and the state of the driving variables operating them (overcast sky, high relative humidity, and cooler
temperatures) militate against evapotranspiration; (b) insofar as the real ET is small, it could be trivial when compared to the error of rainfall measurement.

Experiences with this model offer good grist for discussing the data phenomenon in modeling.

First, some model specifications were established with an eye on the data. The model uses a time increment of 1/2 hour, a limitation imposed by the resolution of the rainfall data. Unfortunately this resolution approximates the reaction time of the watershed (time of concentration), and causes some coarseness in the output. This is exacerbated by synchronization difficulties discussed subsequently.

Second, a most obvious comment is the lack of any evapotranspiration function. Justification in this is found in above paragraphs, although its exclusion is absolutely unforgivable to hydrology purists. Note the data considerations: (a) the relevant role of short term ET, (b) the requirement for short term temperature data, etc., and finally (c) the pragmatic consideration: the model seems to work well without considering evapotranspiration. Thus, in future efforts, ET instrumentation might be an unnecessary encumbrance.

Third, there is a matter of synchronization of the data. Precipitation was taken from three recording rain gages and averaged with a Theessen mean procedure. As the model uses a 1/2 hour time resolution it is highly doubtful that the clocks were synchronized within, say, half of that. For a valid representation of reality, this input should be in phase with the streamflow, which was recorded on a punch tape system with 1/4 hour resolution. The problems are obvious: a time bias is highly probable (and variable) for each storm studied. The model deals with this by brute force: a channel routing procedure swamps any errors from this source, but it then in itself less valid.

Fourth, the utility of using less than exceptional events to calibrate creates some awkward situations. The model uses a constant infiltration capacity as the criteria for the occurrence of overland flow. Apparently none of the storms studied created any overland flow: all hydrographs
could be explained in terms of channel interception and interflow ("quick-flow"), without resorting to overland flow. Although this is in keeping with our knowledge of hydrology in small forested watersheds, it does not permit definition of the infiltration parameter. A lower limit to the infiltration capacity can be defined only as greater than the maximum input intensity. Any problem solution for more intense storms would hang on estimated value drawn from the modeler's judgment. Also, the saturation moisture parameter could not be quantitatively defined, as it was apparently never attained. Such real limitations of digital modeling argue the use of extreme events in calibration, when all processes and parameters are operating. Thus, for example, snowmelt coefficients could not be determined from a summer thundershower, even though a snowmelt routine was included in the model structure.

Finally, there is the subtle matter of the identity relationship between model processes and process parameter, and the actual field values. This question deals with the relative "whiteness" of the model; e.g., is the field capacity (FC) of the model the actual field capacity of the on-site Chicken Creek soils? If field tests had been conducted to determine the soil moisture relationships (data), could this data then be used in the model, and to what effect? Could recorded initial soil moisture levels be used as initial condition for model runs? More serious modeling efforts attempt to detail these matters and account for pre-knowledge. If such refinement is not necessary goal, the modeler is as well off to lapse into the more convenient, less disturbing, and less expensive rationale of modeling as an approximation, i.e., a "blacker box."

In summary, our simple dark gray model reacts with data in many disturbing ways: (1) The model resolution is limited by the data quality; (2) The conceptualization of the evapotranspiration processes is neglected considering the data demands and the relevance of the process; (3) Questionable input-output synchronization blurs the model calibration and its usefulness for further application; (4) Some model coefficients cannot be fully determined because of the nature of the events, which did
not stress all the model processes; and (5) There are severe questions as to what the model coefficients represent on the real-life prototype watershed.

It requires only a moderate imagination to see how these problems would be compounded with a whiter model. Further model detail would introduce more structure and more coefficients, which would increase the difficulties of calibration, and demand more data. Should the model be distributed (split up into sub-watersheds), or be expanded to include water quality dimensions, the data requirements could increase exponentially. A veritable Pandora's Box of problems, questions, and insecurities arise, and the modeler's judgments by necessity assume a prominent role.

**Summary and Conclusions**

There are distinct relationships between hydrologic modeling and the associated data base. Data considerations arise at almost every turn in the modeling procedure. Important data considerations are its accuracy, precision, resolution, synchronization, and quantity and continuity. Data requirements reflect upon the model's whiteness, and the more informative the model output, the more exacting are the data requirements.

Despite the provocative criticisms that data considerations promote, models are still quite useful tools in applied hydrology and research. Modeling is often the only technique available with certain problems. The modeling procedure incorporates the modelers personality, his notions, and his judgments; his resourcefulness drives him to make do with what is available. The effect of these less-than-ideal conditions confuses the model credibility to an unknown degree; although it should be acknowledged and considered in all stages. Despite its faults and the inherent data shortcomings, modeling remains the finest product of the hydrologists art.
Acknowledgments

The ideas presented in this paper have been generated from several research and instructional efforts at Utah State University. I am indebted to the Utah Water Research Laboratory for participation in several studies over the past several years. Much peripheral effort and several studies, including the Chicken Creek work described herein, have been supported by the Utah Agricultural Experiment Station. The Chicken Creek data were supplied by the Intermountain Forest and Range Experiment Station of the U.S. Forest Service, through the personal cooperation of Mr. Robert S. Johnston. The figures were prepared by Mr. Charles Pettee, and the model work was done as class exercises in WS-CEE 570, Watershed Hydrologic Modelling, at USU. Messrs J. P. Riley and R. W. Hill have cooperated extensively in these efforts.

References


ECONOMIC AND HYDROLOGIC ASPECTS OF
DATA NETWORK DESIGN

by

Marshall E. Moss*

The rational design of hydrologic-data networks increases the net output of the data-gathering community in two ways: (1) It provides information for those decisions that can profit from additional hydrologic input, and (2) it permits definition and discontinuance of those data-gathering activities with costs that surpass their returns. It would seem, therefore, that economic criteria based on the worth of the data would be a firm basis for network design. Ideally it is, and studies of the design of hydrologic structures (Dawdy et al., 1970; Moss, 1970; Davis et al., 1972) have indicated that under some circumstances it may even be realizable. However, general implementation of this approach is not now (1975) achievable; nor is it likely to be in the near future.

Several factors contribute to the relative immaturity of this field of endeavor. First, where schemes have been devised for estimating the worth of hydrologic data, they are dependent on the uses of the data being known. Many data uses cannot be foreseen in sufficient time to make the data available between the time of their anticipation and the time at which they are required; thus, the validity of the economic analysis may be tainted by uncertainty contained in the forecast of the data use. Second, satisfactory means for measuring the economic value of all the factors affected by a water-resources system may not be available. This problem is acute in the rather common occurrence in which several factions will be involved in the evaluation of a plan of development. The aspects of the plan that seem to be positive to one

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faction may seem negative to others and vice versa. Problems also occur in valuating intangible assets such as recreation (Knetsch, 1974) and particularly human life itself (Buehler, 1975).

A third factor that often thwarts economic design of hydrologic networks is the numerical complexity required to solve the problem. One reason is that the network design is a function of the parameters that it is established to measure, as was brought out by Davis and others (1972). Another reason is that the system or model that digests the data into a form that is useful in a decision context frequently is so complex itself that to analyze the effects of, or to measure the worth of, the data becomes a very arduous task to say the least.

To illustrate this last point, take as an example the planning of water-resources developments for the Commonwealth of Puerto Rico. The U.S. Geological Survey undertook a study to develop a capacity-expansion model that could be used in a realistic context as a substitute for a water-resources planner (Moody et al., 1973). One of the prime reasons for this undertaking was to gain insight into the role that hydrologic data play in the planning process, with the aim that such understanding would lead to useful design procedures for hydrologic-data networks. The planning model, in essence, defined a least cost set of water-development projects that would supply the projected municipal, industrial, and agricultural needs subject to certain constraints such as the maintenance of minimum flows in certain reaches of the rivers. The supplies could come from various sources, such as run-of-the-river diversions, surface-water reservoirs, groundwater development, and desalination plants. The only model inputs of a hydrologic nature were the definition of the various projects (sources) and their capacities to supply water (yield).

The model defined that set of projects and their times of construction that yielded the minimum present value for construction, operation, maintenance, and replacement costs and met the projected water needs.
The yield of each project, except the desalination plants, was pre-specified by some form of design procedure that took into account the available hydrologic data. Although a single value was used for the yield of each project, some uncertainty remained concerning the true yields because the hydrologic-data base was not such that the exact character of the hydrology could be defined. It is doubtful that a state of perfect definition of a hydrologic character could ever be defined; however, collection of pertinent hydrologic-data can be expected to lead to a less uncertain specification of its nature. How much value the added data contributed to the planning process was one of the questions that was to be addressed in this study.

In the real world the uncertain yield of any particular project might be described by a probability distribution of the unknown value of yield. For pragmatic reasons this distribution might be simplified to two equally likely values, a high yield and a low yield, that have the same expected financial loss as the more realistic continuous distribution. Such a simplification could be made for each project; thereby describing, after a fashion, the hydrologic uncertainty in the planning process. If the planning model is run for each possible combination of yields of the projects, a Bayesian analysis could be performed to specify that set of projects that would have the lowest expected cost that also includes the costs of water shortages (underdesign). Such a design, which would include the effects of hydrologic uncertainty, would have associated with it an expected cost that could be reduced by the collection of hydrologic data.

Suppose that a program of data collection is defined. The effects of that data program on the uncertainty of the yields can be simulated, and new estimates of high and low yield can be determined for each project. For projects where the data are pertinent to the definition of the yield, the high and low yields can be expected to converge toward some central value; where the data are not pertinent, the yields will remain the same as in the prior step. A new Bayesian analysis can be
performed to identify the new optimum design and its expected minimum cost. The reduction in expected cost after the inclusion of the effects of the anticipated data is a measure of the economic worth of this data program.

Several data programs could be examined and the one that yielded the highest net return could be specified for implementation. The major problem with this simplified approach is that on the order of $2^N$, where $N$ is the number of projects, runs of the planning model would be required for the basic solution (the solution for the currently available data base) and $2^N$ more runs would be required for each network that is explored. If the region where the plan is to be developed contains 24 possible projects (not a very liberal estimate for a developing area), and if each run takes only one second of computer time (a gross underestimate), the basic solution and the networks investigated would require in excess of 6 months of computer time each. Thus far any attempts to simplify the problem further or circumvent the "curse of dimensionality" have resulted in either the same quandary or dismal failure.

What is the alternative to such a design procedure? Obviously, anything that yields a "reasonable" design is, because the above approach will not solve the problem. The remainder of the paper will discuss available tools or strategies that can be used for network design; however, it must be stressed that each must be considered only an interim procedure for none solve the complete network-design puzzle.

The World Meteorological Organization (1970) has taken a rather pragmatic approach to network designs because of the rather early stage of development of scientific-design methodologies. For a minimum network they propose ranges in areal densities of hydrometric stations for each of several types of hydrologic regimes. These guidelines, derived from past experiences of the worldwide hydrologic community, are tempered by providing for less dense networks in areas with difficult gaging conditions.
Another pragmatic approach evolves from a desire to account for hydrologic changes that occur within specific regions. The regions of interest are delineated on maps and gaging stations are established at the significant points of inflow and outflow of each of the areas. This method has been used by the U.S. Geological Survey to locate stations for its National Stream Quality Accounting Network (Ficke and Hawkinson, 1975).

A third means for the design of networks is the use of a surrogate in lieu of the benefit-cost analysis that is so often intractible as was discussed above. A surrogate that often is used is a design criterion based on statistical information content, which is directly related to the accuracy of an estimate of a particular parameter. It seems only logical that a more accurate parameter estimate, whether it is hydrologic or otherwise, is a more valuable one; thus information content would appear to be a reasonable alternative. However, it is not a perfect approach; Moss (1970) in a study of the worth of hydrologic data in surface-water reservoir design found that, although the worth was directly related to information content, it was not proportional. Therefore, it is possible that a network-design criterion that maximizes a measure of information may not yield the optimal network in terms of the economics of developing the water resource.

Hardison (1970) has proposed an information measure called the equivalent years of record that can be used in conjunction with parameter estimates that are not derived directly from data records. This measure has much utility in the design of planning-level hydrologic-data networks, because the planning process often requires information at sites where data are not available. These information demands must be initially met by some information-transfer mechanism that uses as its information source existing records in the vicinity of the demand.

The U.S. Geological Survey adopted equivalent-years-of-record criteria for a nation-wide evaluation of its surface-water-data programs (Benson and Carter, 1973). The goals of 10 equivalent years everywhere
in the nation and 25 equivalent years for major streams were applied to the estimation of each of a rather long list of surface-water parameters. When the particular criterion was met for a set of related parameters such as those describing flood flows, changes in the operations of the networks could be identified that would aid in attaining goals for the remaining parameters. In other words, the evaluation could result in an intensification of gaging activity, or a lessening thereof, or a change in emphasis from one type of parameter to another. More recent developments (Moss and Karlinger, 1974) have provided the means by which gaging programs can be specified that will meet the goal (information criteria). Also the time frame for meeting the goals can be estimated.

Hydrologic-network design has received much worldwide attention recently with several symposia and workshops with themes that were specifically directed at this problem. Other symposia also have contributed toward network design by providing interchange of ideas that leads to better hydrologic understanding, which should be the cornerstone of new design techniques. Another mechanism for exchange of network-design information has been provided by the World Meteorological Organization. Its Casebook on Hydrological Network Design Practice (WMO, 1972) is a continuing part of the WMO program to provide international assistance in the design of water-data programs. The initial set of papers contained in the Casebook will be expanded as new experiences and methodologies are developed.

Thus, it seems that the recent interest in hydrologic-network design has created an impetus that is leading to more quantitative planning and management of hydrologic-data programs. At least the attitude that all data are "good" data seems to be passing from the scene.
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DATA BANK FOR WATER RESOURCES INFORMATION

by

Frank J. Trelease, III*

I am pleased to be representing George Christopulos, the Wyoming State Engineer, at your Conference. I wish to briefly expose to you three "data banks" which appear to me to be necessary to make investigations and studies of Colorado River water supplies in Wyoming. These data banks include:


To make water supply studies anywhere in Wyoming one must know both the legal and physical availability of water. At the outset of the Wyoming Water Planning Program in 1967 and 1968, all three of the data systems were begun in order to create a data bank for the Colorado River Basin in Wyoming and eventually for the rest of the state. In 1968, a computer storage and retrieval system was investigated to manage water right records, most of which are land descriptions indicating the number of acres irrigated in each 40-acre tract, and a system was implemented during the next few years. The storage and retrieval system of streamflow data was a first endeavor towards a water resources basic data system, and it was operational by 1969. In order to estimate available water supplies for new and supplemental uses, we found it necessary to compile and derive streamflow data for existing and discontinued stream gages in the Green River Basin.

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I wish to briefly describe the three data banks that make up the available water resource information in the Green River Basin, Wyoming.

**Water Rights Information System**

Through funding of the Water Planning Program, the State Engineer's Office implemented a computerized system for the storage and retrieval of all of the 116,300 water right records for the State of Wyoming. Perhaps 20,000 of these records are Wyoming water rights in the Colorado River Basin. These rights consist of territorial rights which were established prior to Statehood, and permits for direct flow rights, ditch enlargements, reservoirs, stockponds, wells, and certificates of adjudication from the Board of Control. To our knowledge, this is the most sophisticated water rights information retrieval system in the western states. We believe our system is unique in having a system and programming design revolving around a dictionary of elements. This allows an open end concept for adding elements to the database. The master file is a direct access file consisting of a variable number of variable length elements within fixed length records.

The data conversion has been completed, and we are now over 50 percent completed in proofreading the entire master file. Requests for water rights information which come in daily have historically been and must now be manually tabulated. These tabulations are compared with requests addressed to the computer to check the accuracy of the computer data and to assist in the proofing process. Due to the necessity of running parallel checks and the additional time delay required to verify the printouts, computer inquiries are made on a rather limited basis at this time. However, as time goes on and the database accuracy is improved, more complete use will be made of the computer information system.
The Wyoming State Engineer's Office contracted with the Wyoming Water Resources Research Institute for the implementation of a computerized system of surface water data of the State in 1967. This system, known as surface water system or SWS, was completed in 1969. It has since been updated with current data and transformed from a second generation computer, a Philco 2000, to a third generation XDS Sigma 7 computer.

Essentially, all streams in or adjacent to Wyoming with five or more years of records are included in SWS. Several kinds of data are stored within the system. The principal kind of data is mean daily flows. Maximum annual instantaneous peaks are included with the flow records. Monthly streamflow volumes in acre-feet and end-of-the-month reservoir contents in acre-feet are stored separately. The reason for storing the monthly volumes rather than computing them as needed from the daily values is because there is a large amount of published monthly data for which there are no published daily values. Most of the data is from U.S. Geological Survey publications, but diversion records from the State Engineer's Office and Bureau of Reclamation have been added.

Of course, storage of the data per se would be of little value. The benefits are derived from being able to retrieve and massage, quickly and cheaply, the data for hydrologic analyses. Programs are stored on file to do several kinds of analyses. They are: MADIS, LACOR, MOCOR, DAYFLOW, DURCUR, MONMAX, MONMIN, PERMAX, PERMIN, SORT, LOGPT3, GUMBLE, HAZEN, and RESERV.

MADIS is a routine to print a table of monthly and annual discharges in acre-feet for any desired period of record. Included in the table are total and mean monthly and annual flows for the period and percent of mean annual flow by month and years. Statistics for the
monthly and annual flows are produced plus moving average statistics of annual flows for any specified time period.

LACOR is a correlation method utilizing Langbein's method in which correlations are made in terms of deviations in log units from the geometric mean of monthly discharges. Correlations are made only for selected matching months of record using mean monthly discharge. Missing values of the dependent station are extrapolated.

MOCOR is another correlation routine which uses a simple linear regression fit between stations values or their logarithms.

DAYFLOW provides a table of mean daily flows in cubic feet per second by water year plus monthly volumes in second-foot days and acre-feet, mean monthly flows, total yearly flow in acre-feet, and the amount and date of the annual maximum instantaneous peak.

DURCUR provides a flow duration table and curves prepared for a station utilizing any desired period of record for up to 32 class sizes which are selected by the user. The table printout includes total second-foot days and number of occurrences in each class for each year, total second-foot days for the entire period, mean annual second-foot days, mean daily discharge in cfs, total number of occurrences in each class, cumulative totals, percent of total accumulated occurrence in each class, class size divided by the drainage area and class size divided by the mean daily discharge. Four curves are printed as semi-logarithmic plots with discharge on the log scale. Flow duration tables and curves can be obtained for one or several months if specified by the user.

MONMAX and MONMIN are routines to tabulate monthly maximum or minimum daily flows for any desired station and period of record.

PERMAX and PERMIN are routines to tabulate maximum and minimum mean daily discharge for selected consecutive time periods. Any time periods in days can be specified.

SORT is a routine to sort mean daily flows by magnitude for any selected period and print them showing the dates of occurrence.
LOGPT3, GUMBLE, and HAZEN are routines to do flow frequency analysis by Log Pearson Type III, Gumble, or the Hazen method. RESERV is a program to print out end-of-the-month contents and change in contents in acre-feet for reservoirs.

Recently, the Wyoming Water Planning Program completed a study of monthly streamflow volumes in the Wyoming portion of the Upper Colorado River Basin. Monthly correlations using MOCOR and LACOR were very helpful in this study.

Further information on SWS can be obtained by writing to WRRI, P.O. Box 3067 University Station, Laramie, Wyoming 82071, and asking for Report No. 43, Surface Water System-1973.

SWS is now being included in a larger Water Resource data system or WRDS that is being developed by WRRI under a contract with the Wyoming Water Planning Program. Several state agencies are cooperating in the effort, and the Bureau of Land Management is cooperating in implementing WRDS. This system will encompass data for several kinds of water resources data, including surface water quantity and quality, groundwater quality, climatological and snowcourse data. The surface water portion will be enhanced by adding crest-flow data from small watersheds and providing daily hydrograph and Log Pearson Type III frequency curve plotting capabilities.

Water quality data are being prepared for storage from U.S. Geological Survey records, the Wyoming Chemicalogical and Bacteriological Laboratory, Bureau of Reclamation, and other readily available sources. Retrieval is to be provided by identification code (municipal water supply, irrigation supply, etc.), USGS station number, county or state, city, latitude-longitude by ten-minute square, township-range, and drainage basin. Drainage basins are defined, subdivided and coded by an eight-digit code. The leftmost two digits designate major drainage basins such as the Green River Basin; the next two digits designate tributaries of the major basin; the next two digits designate tributaries of the previous tributary; and the next two digits designate the next two
smaller subdivisions of tributaries. This coding provides for automatic data retrieval for entire basins, small basins, or combinations thereof.

A problem encountered with the development of the water quality segment of the system is the many parameters that may be measured and ways of measuring them. As an example, consider carbon. It may be measured as:

- Carbon, Inorganic, Bed Material (G/KG)
- Carbon, Inorganic, Dissolved (MG/L as C)
- Carbon, Inorganic, Suspended (MG/L as C)
- Carbon, Inorganic, Total (MG/L as C)
- Carbon, Organic, Bed Material (G/KG)
- Carbon, Organic, Dissolved (MG/L as C)
- Carbon, Organic, Immiscible (MG/L as C)
- Carbon, Organic, Suspended (MG/L as C)
- Carbon, Organic, Suspended (MG/KG as C)
- Carbon, Organic, Total (MG/L as C)
- Carbon, Total (MG/L as C)

A coding number has been assigned for each parameter, and a file of corresponding label headings is stored in the system. There presently is provision for 627 parameters and they can be expanded upon quite easily.

The Wyoming Department of Environmental Quality has collected city water supply samples throughout the state over the past two years. These data have been put into the system.

Programs are being developed to provide statistical, regression and plotting analyses of the data. Plotting routines will provide for plotting of one parameter versus time or one parameter versus another parameter, sediment duration curves and sediment rating curves.

Climatic data from U.S. Weather Service substations are being stored and it is planned to include wind data and hourly precipitation. Useful analytical programs applicable to these data, such as normal summaries, will be written.
Snowcourse data from the Soil Conservation Service publications plus any other readily available data will be stored and programs to statistically analyze these data will be included.

SWS has proven to be a useful tool, not only for state and private organizations but for federal agencies as well. It is believed that the largely expanded WRDS system will prove to be an even greater tool for water resource planning and management in Wyoming.

**Observed and Estimated Streamflow of the Green River Basin, Wyoming**

The Wyoming Water Planning Program began in 1967 with investigations in the Green River Basin. The U.S. Bureau of Reclamation was also authorized to study the Green River Basin, so we endeavored to make joint use of all data possible. Initially, the state had staff and the bureau did not, so we correlated streamflow data for the gages necessary to conduct reservoir operation studies and submitted the data to the USBR. Since we are looking for firm water supplies in many instances, we found we had to include the period in the mid-1930's for analysis. There were very few stream gages in existence at that time, some with partial records, some that were discontinued later. We agreed on a 1932-1967 data base for our studies. The Wyoming Water Planning Program report on the Green River Basin was published in 1970, but the bureau's studies continued until 1972 and after. Because of change of USBR personnel, they derived and compiled their own set of streamflow data which differed from the state data.

In the years since 1970, we had investigated, through the Wyo WRRI, several watershed parameter and statistical techniques of hydrologic analyses to derive streamflow information. These studies were prompted, among other things, by the continually declining level of funding for the USGS stream gage program. We found that correlation of streamflow records for existing or discontinued stream gages provided
the most reliable streamflow information, particularly for water supply studies.

Last year the Sublette Study was authorized for the Bureau of Reclamation and a USDA Type IV River Basin Survey was begun in the basin. With two studies authorized in the basin, both using the multi-objective planning criteria, it was determined that a common data base of observed and estimated streamflow data was desirable. We agreed to accomplish this in cooperation with the USBR for the water years 1930-1973.

We have just completed compiling the data for 60 stream gages, 2 reservoirs, and 4 records of adjustments and adjusted flows to reflect the effects of new depletions upon historic streamflows. Only one of the 60 stream gage records, Henrys Fork near Burntfork, Wyoming, was complete for the 44-year period of record.

The interesting thing in the compilation, or perhaps the sad thing, is the fact that the hydrologists did not agree on the past correlations. The reasons for disagreement varied from lack of substantiating data to errors in the correlations to discontinued gages that were used as basis of correlations. In fact, one of the biggest problems is the fact that many stream gages were discontinued in the recent five-year period. One would think that such an undertaking would be unnecessary with so much previous work having been done by the USBR and the state. The USGS had even published Water Supply Paper 1875 "Correlative Estimates of Streamflow in the Upper Colorado River Basin," but the discontinuance of stream gages limited the use of the publication.

My office will soon release upon request, copies of these correlations which should be utilized for any water resources planning requiring streamflow records in the Green River Basin, Wyoming. Through the use of this publication we hope to avoid any further duplication of effort in streamflow correlations. It will be necessary for future studies to add to the data and to make estimates of streamflow at discontinued gages. My office has on file a well documented set of supporting material upon which to base the future estimates.
Conclusions

Wyoming now has, or soon will complete, the necessary data banks of water rights, streamflows, water quality, and climatological data necessary for traditional water supply hydrology studies. Hopefully, we are now cooperating with federal agencies in both the storage and retrieval and utilization of the data, and by continued cooperation, we hope the data bank can be maintained and improved.

Undoubtedly, there are types of data which should be in the data bank which are not now being included or are not available. A considerable quantity of environmental data is necessary. Geology, soil types, vegetative cover, wildlife habitat, stream biota, and fisheries and many other kinds of information are needed. In addition, as we proceed further into environmental impact analyses, etc., we find it is necessary to have data on the micro rather than on the macro basis. Therefore, it is apparent that hydrologic and climatological data are needed where it has not heretofore been collected. Through continued and increased cooperative efforts, we believe the data base can be continually improved and updated, and through a systems approach the data can be more easily and quickly utilized.
LONG-TERM STREAMFLOW RECONSTRUCTION IN THE UPPER COLORADO RIVER BASIN USING TREE RINGS*

by

Charles W. Stockton**

Introduction

Any statistical work involving hydrologic records is handicapped when the records are of relatively short duration as are most such records in the Colorado River Basin. This is because the short records are not necessarily a random sample of the infinite population of events and consequently any statistical descriptions are likely to be in error to some extent.

Recent work by Stockton (1975) and Stockton and Jacoby (1975) has shown that tree-ring data can be used to extend available runoff records backward in time, thereby providing a longer record from which to more accurately estimate the three most common statistics used in hydrology: the mean, the variance, and the first order autocorrelation. In addition, records reconstructed from tree-ring data series can provide information on (a) longest periods of sustained high or low flow and (b) the representativeness of the historical record in comparison to the long-term record reconstructed from tree-ring data. Among the useful features of tree-ring series as repositories of hydrologic information are their great number, their longevity, and the critical fact that the information they contain is annually cumulative. Thus, tree-ring data can be an important source of added information to the hydrologist provided that hydrologic inferences based on such data can be supported by acceptable statistical controls.

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In statistical analysis of hydrologic phenomena, it is usually assumed that a record of events that is of finite length represents a random sample from an infinite population, the occurrence of each event being governed by some probability distribution. Any change in the hydrologic regime with which a given record of events is associated results in a change in the probability distribution.

For practical purposes, a probability distribution is described by the mean (a measure of central tendency), the variance (a measure of the average spread of the events about the mean), and the skewness (a measure of the asymmetry of the distribution of the events about the mean). In some cases these three parameters uniquely define a probability distribution and are useful for describing hydrologic phenomena. For most annual runoff and tree-ring index series, the variables are normally distributed (skewness equals zero) and the probability distribution is completely described by the mean and variance. In almost every mathematical model of runoff time series, the first order autocorrelation (a measure of persistence in a series of events) is used along with the mean and variance. The population values of these statistics are usually unknown and therefore must be estimated from the existing record of observations. Consequently, the reliability of the estimates depends primarily upon the length of record of the observations—in other words, the total number of observations.

If there are errors in the estimates of the population parameters owing to shortness of observed records, these errors are preserved in any synthetic series that is generated from the available data. Recently, Rodríguez-Iturbe (1969) showed that if the length of an annual runoff record is 40 years or less, there may be an error of 2 percent to 20 percent in estimation of the mean, from 15 percent to 60 percent in estimation of the variance, and as much as 200 percent in estimation of the first order autocorrelation. The high error in the autocorrelation is probably related to the inadequacy of short records for estimation of the low-frequency persistence in hydrologic data.
Fiering (1967), Matalas and Jacobs (1964), and Julian and Fritts (1968) have demonstrated the use of the correlation techniques for augmenting hydrologic records. In each case a single record was used to augment another. Fiering (1963) also approached the problem using multiple linear regression, that is, using several independent variables to predict a dependent variable. He showed that a better estimate of the mean can be obtained in the multivariate case if \( R^2 \geq \frac{q_i}{(n_1 - q_i)} \), where \( R \) is the combined correlation coefficient, \( q_i \) is the number of variables included in the prediction equation, and \( n_1 \) is the length of the record to be extended. In the case of the variance, the variance of the reconstructed record is a better estimate if the relative information ratio \( I \) (the ratio of the variance to that estimated from the original record) exceeds 1. When \( I \) exceeds unity, it implies that the variance of the estimate of a moment made from the original record is larger than that of the estimate made from the combined record, and therefore a more precise estimate is computed from the combined data.

As a general rule the estimate from the longer series is more reliable if \( R \) exceeds 0.80 (Table 3 of Fiering, 1963; p. 2 of Matalas and Jacobs, 1964). However Matalas and Jacobs (1964) point out that these requirements can be reduced and that the parameters estimated from the longer series are an unbiased estimate if a noise factor is added to the estimated values.

The basis for comparing annual runoff series with tree-ring series is the hypothesis that the two series respond to a common climatic signal or signals that permit prediction of annual runoff from the annual ring-width index. A schematic diagram of the climatic variables influencing both of the series and the resultant reconstructibility is shown in Figure 1.

Precipitation (a), temperature (b), and evapotranspiration (c) influence the water balance of both runoff and tree growth. However, in

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1"Indexing" (standardization) is necessary to convert the non-stationary ring-width series to a stationary time series (Stokes and Smiley, 1968).
Figure 1. Schematic diagram of relationship between ring-width series and annual runoff series for medium and large watersheds.

the case of tree growth, these variables, and especially temperature, have physiological influences not directly related to the water balance; these influences are diagrammed in Fritts et al. (1971). The seasonal distribution of the variables (d) influences both runoff and tree growth, and in the case of tree growth the influence of the monthly distribution extends to at least a 14-month period--from the July prior to the growing season in which the ring is formed to the July concurrent with the growing season (Fritts et al., 1971). Spatial distribution of precipitation
and temperature (c) within large watersheds may influence both the
annual runoff regime and the variability in growth of trees from site to site.

The noise component in Figure 1 represents both the model's inability to adequately describe the two series and the differences in the way the two series respond to climatic inputs.

Of major concern in the reconstruction of annual runoff series from tree-ring records is the difference in persistence within each of the two series—that is, how much do events of the previous year or years influence the current year? In some cases, differences in persistence have been resolved by using lagged dependent variables on the right-hand side of the reconstruction equation, as described by Johnston (1963). Unfortunately, this causes the residuals to be dependent upon residuals of prior reconstructed values. Also, the regression coefficients tend to be biased although they have the properties of consistency and efficiency (Johnston, 1963) if the residuals are normally distributed. Another remedy is to use a matrix of the tree-ring data, lagged up to three times, and extract principal components from this supplemental matrix. The covariation in this matrix can be decomposed by extracting the eigenvectors. A new set of uncorrelated variables is obtained from the amplitudes of the eigenvectors. These amplitudes may be lagged in certain ways with the runoff data, and multiple regression may be used to weight the respective series so that the differences in persistence are accounted for. This aspect is covered in greater detail in the Dendrohydrology section.

Dendrochronology

The primary objective of our study of the Colorado River has been to reconstruct long-term runoff records from major runoff-producing areas within the Upper Basin. Therefore it was desirable to utilize tree-ring series from as many of the major runoff-producing areas as possible. For many of them, climatically sensitive tree-ring series had been collected for other projects. For other areas, it was necessary to
obtain tree-ring samples specifically for the Lake Powell Research Project. All the samples were collected using a small-diameter Swedish increment borer so as not to injure the trees. Figure 2 shows the spatial distribution and relationships to major runoff-producing areas of the 31 different tree-ring sites used in this study, 12 of which were collected specifically for the Lake Powell Research Project. Table 1 lists the individual tree-ring series and shows some of the important statistical details of them. In addition to the period of record for each of the series, the first order autocorrelation coefficient ($R_1$), the coefficient of mean sensitivity ($M.S.$), and the standard derivation (S.D.) are shown. These three statistics provide measures of a) persistence, b) high frequency variation, and c) total variation, respectively, in the tree-ring data series and are described in more detail in Stockton (1975). In general, the more climatically sensitive series possess statistics in the neighborhood of $R_1 = .20-.30$, $M.S. = .35-.45$, S.D. $= .35-.45$ (Stockton, 1973). As can be seen in scanning the statistics of the 31 data series listed in Table 1, some of the series do not possess statistics equal to those of the more climatically sensitive series. However, it is believed that the position of the site within the basin and relative to major runoff-producing zones was more important for utilizing the data series in runoff reconstruction than was maximum climatic sensitivity.

All of the tree-ring series used here are mean-value functions. That is, at least two series from each tree are averaged to provide the best estimate of the series from that tree, and a multitude of tree series comprises a site series. Normally at least 10 trees are sampled at each site. At one site, the Uinta D (Number 9, Table 1), however, only four trees (8 core series) were sampled because of the lack of additional trees suitable for sampling.

The minimum objective of 10 trees (2 radii sampled from each tree) is based on experience of the staff at the Laboratory of Tree-Ring Research. In western North America we have found that sampling a "climatically homogeneous" site in such a manner gives a mean-value function that maximizes the climatic signal representative of that site.
Figure 2. Map of Upper Colorado River Region, showing (a) major runoff-producing areas (shaded); (b) locations of tree-ring data sites (dots)—see Table 1 for names of numbered sites; (c) four major gaging sites (triangles): Green River at Green River, Utah (3150), Colorado River at Cisco, Utah (1805), San Juan River near Bluff, Utah (3795), and Lee Ferry, Ariz.
Table 1. Table of tree-ring data sites used in this study. Map number refers to Figure 1 which shows the relative location of each of the sites. I.D. number refers to the Laboratory of Tree-Ring Research identification number; period of record is the period of years included in the tree-ring series; R. is the autocorrelation coefficient, M.S. is the coefficient of mean sensitivity and S.D. is the standard deviation.

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<th>Map No.</th>
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<th>I.D. Number</th>
<th>Period of Record</th>
<th>R</th>
<th>M.S.</th>
<th>S.D.</th>
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a. tree-ring data collected as part of NSF sponsored Lake Powell Research Project

b. tree-ring data from the files of the Laboratory of Tree-Ring Research

c. tree-ring data collected as part of A.R.P.A. sponsored project entitled "Reconstruction of Past Climatic Variability"

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and minimizes the noise signal due to individual tree idiosyncrasies. Because our ultimate objective requires the use of the climatic signal inherent in the tree-ring data, we particularly desire to use techniques that will maximize that signal.

The climatic sensitivity of a particular tree-ring series is controlled by the site conditions upon which the sampled trees are growing. Ideally, a site is selected that is at or near the limit of the natural distribution of the species and located on a sloping surface where soil development is negligible. But in many instances location relative to a watershed boundary or a certain climatic station to be used for calibration is an overriding factor. In this study it was necessary to consider one additional factor, and that was the variability of an existing series in the vicinity of the needed location. With limited funding, only the most crucial areas could be economically justified for new site collections. Each new site collection involves a rather large investment, which includes not just the collecting but also the laboratory dating, measuring, and computer processing. Recent estimates indicate each new series cost as much as $3000 to collect and process.

All tree-ring data utilized in this study were processed in accordance with the procedures currently in use at the Laboratory of Tree-Ring Research; that is, the individual cores were mounted in wooden core mounts, surfaced to aid in distinguishing the individual rings, and cross-dated, and the individual rings were measured to within 0.01 mm as described in Stokes and Smiley (1968).

Because most tree-ring data series are, in fact, nonstationary time series—that is, both the mean ring-width and variance are a function of time—each series must be transformed to at least a weakly stationary series. This is accomplished by fitting a least-squares fit curve, most commonly of modified exponential form, to the annual ring-width series. An index is then formed by considering the value of the curve at the time $t$ as the expected value and by dividing the actual value by the expected value. Although this operation has some drawbacks, it is necessary to transform the original nonstationary series into a more usable stationary
series. After each measured radius is transformed into a series of indices, the indices are averaged into individual tree chronologies and subsequently the tree chronologies are averaged to obtain the mean-value function for the site.

Dendrohydrology

Total annual runoff records have been reconstructed for various subbasins within the Upper Basin Region by use of the climatic signal inherent in the tree-ring series selected from major runoff-producing areas. The basic technique of reconstruction and the logic behind the use of appropriately chosen tree-ring series have been detailed by Stockton (1975) and need not be repeated here. However, it is necessary to briefly explain the system of models used.

If the climatic input into either the biologic system (represented by the tree-ring series) or the hydrologic system (represented by the runoff series) were purely an annual phenomenon (no year-to-year carryover), the model could represent a simple one-to-one relationship. However, for neither system is such necessarily the case.

Consider first the biologic system, as represented by the tree-ring series. Fritts (1975) illustrates how the tree-ring response to a climatic input can be recorded in ring widths over a number of consecutive years. This is shown, greatly simplified, in Figure 3, where a climatic input of precipitation and temperature coupled with atmospheric elements of wind and carbon dioxide is reflected in the ring width not only of year \( t \) but also of year \( t + 1 \) (through bud development and sugar and hormone storage and carryover) and of year \( t + k \) (through leaf, root, and fruit growth processes). Superimposed upon this climatic carryover effect is a food storage and soil moisture carryover as reflected in the tendency for rather significant autocorrelation in the ring-width series. This is expressed by the \( t - k \) parameters in the model.

The hydrologic system (surface runoff series) may also contain a tendency for autocorrelation. This may be a result of groundwater
Figure 3. Schematic diagram showing how climate of year \( t \) can affect tree growth in year \( t+k \) (after Fritts, 1975).

storage reflected as baseflow, evapotranspiration, bank storage, or other factors. In certain circumstances, this tendency for persistence may be large enough to require its being taken into account in any reconstruction.

We have used a set of seven empirically chosen models (Table 2) utilizing different values of \( t \pm k \) for the tree-ring series and \( f-k \) for the runoff series. Each model has been computed for each of the 12 subbasins for which runoff records were reconstructed. In each
Table 2. Seven models for predicting annual runoff by using tree-ring data.

<table>
<thead>
<tr>
<th>Model</th>
<th>Tree-ring series</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Runoff ( f_t )</td>
<td>( t^{-1}, t^{-2}, t^{-3} ) with ( f_{t-1} + f_{t-2} + f_{t-3} )</td>
</tr>
<tr>
<td>2. Runoff ( f_t )</td>
<td>( t^{-1}, t^{-2}, t^{-3} ) with ( f_{t-1} )</td>
</tr>
<tr>
<td>3. Runoff ( f_t )</td>
<td>( t^{-1}, t^{-2}, t^{-3} ) with ( f_{t-1} )</td>
</tr>
<tr>
<td>4. Runoff ( f_t )</td>
<td>( t^{-1}, t^{-2}, t^{-3} )</td>
</tr>
<tr>
<td>5. Runoff ( f_t )</td>
<td>( t+1, t, t^{-1}, t^{-2} )</td>
</tr>
<tr>
<td>6. Runoff ( f_t )</td>
<td>( t+2, t+1, t, t^{-1} )</td>
</tr>
<tr>
<td>7. Runoff ( f_t )</td>
<td>( t+3, t+2, t+1, t )</td>
</tr>
</tbody>
</table>

individual case, we chose what we considered to the "best" model and used it in the runoff reconstruction process. We chose the best model on the basis of (a) the amount of variance duplicated in the gaged total runoff record used for calibration, (b) lack of autocorrelation in the residuals, (c) ability to reproduce independent data (i.e., data not used in the calibration process), (d) capability of the reconstructed series synchronous with the recorded series to duplicate the low frequency tendencies of the recorded series, and (e) the physical reasonableness of the model based upon our knowledge of the tree-ring data, the area from which they were sampled, and the hydrology of the subbasin under consideration. The models chosen for reconstruction and the degree to which these models duplicate the calibration record expressed by the correlation coefficient along with other pertinent data are shown in Table 3.

The individual tree-ring sites within the basin for which a reconstruction was undertaken were not necessarily of equal importance. Consequently,
Table 3. Tabulation of gaged stations (USGS) for which tree-ring reconstructions of past flow have been completed. Also shown are: a) the numbers of the tree-ring series used in the reconstruction (see Figure 1 and Table 3); b) number of years in the historical record used in the calibration analysis; c) the correlation coefficient between the gaged record and the tree-ring data; d) number of years in the reconstructed record (see Appendix A for actual records); e) long-term average flow based on the reconstructed record; f) long-term average flow based on the reconstructed record; g) the number of the model (from Table 4) used in the reconstruction process.

<table>
<thead>
<tr>
<th>Runoff Record Reconstructed</th>
<th>Tree Ring Sites used in Reconstruction</th>
<th>Number of Years in Calibration Period</th>
<th>Number of Years in Reconstructed Record</th>
<th>Average Flow</th>
<th>Standard Deviation of Flow</th>
<th>Model number</th>
</tr>
</thead>
<tbody>
<tr>
<td>Green River at Green River Utah</td>
<td>1, 2, 3, 6, 9, 10, 15</td>
<td>57</td>
<td>.80</td>
<td>392</td>
<td>4.48 x 10^6</td>
<td>1.43 x 10^6</td>
</tr>
<tr>
<td>Green River near Daniel, Wyo.</td>
<td>1, 2</td>
<td>31</td>
<td>.64</td>
<td>459</td>
<td>3.649 x 10^5</td>
<td>.571 x 10^5</td>
</tr>
<tr>
<td>New Fork River near Boulder, Wyo.</td>
<td>3, 4</td>
<td>48</td>
<td>.70</td>
<td>268</td>
<td>2.672 x 10^5</td>
<td>.511 x 10^5</td>
</tr>
<tr>
<td>Whiterocks River near Whiterocks, Utah</td>
<td>6, 7, 8</td>
<td>54</td>
<td>.76</td>
<td>239</td>
<td>8.871 x 10^4</td>
<td>2.39 x 10^4</td>
</tr>
<tr>
<td>Colorado River at Cisco, Utah</td>
<td>13, 14, 15, 16, 17</td>
<td>50</td>
<td>.92</td>
<td>321</td>
<td>4.962 x 10^6</td>
<td>1.795 x 10^6</td>
</tr>
<tr>
<td>Fraser River near Winter Park, Colo.</td>
<td>12</td>
<td>51</td>
<td>.66</td>
<td>252</td>
<td>7.172 x 10^4</td>
<td>3.892 x 10^4</td>
</tr>
<tr>
<td>Taylor River near Almont, Colo.</td>
<td>17, 18, 19</td>
<td>51</td>
<td>.68</td>
<td>402</td>
<td>2.526 x 10^5</td>
<td>.551 x 10^5</td>
</tr>
<tr>
<td>Gunnison River near Grand Junction</td>
<td>16, 17, 18, 19</td>
<td>45</td>
<td>.78</td>
<td>322</td>
<td>2.134 x 10^5</td>
<td>.564 x 10^5</td>
</tr>
<tr>
<td>Delores River at Delores, Colo.</td>
<td>23, 24</td>
<td>44</td>
<td>.86</td>
<td>161</td>
<td>2.982 x 10^5</td>
<td>1.063 x 10^5</td>
</tr>
<tr>
<td>Colorado River near Cameo, Colo.</td>
<td>13, 14</td>
<td>28</td>
<td>.79</td>
<td>500</td>
<td>2.023 x 10^5</td>
<td>.562 x 10^5</td>
</tr>
<tr>
<td>San Juan River near Bluff, Utah</td>
<td>26 (2 species)</td>
<td>54</td>
<td>.85</td>
<td>309</td>
<td>2.20 x 10^6</td>
<td>.730 x 10^6</td>
</tr>
<tr>
<td>Colorado River at Compact Point (Lee Ferry)</td>
<td>1, 2, 6, 9, 10, 11</td>
<td>50</td>
<td>.86</td>
<td>450</td>
<td>13.96 x 10^6</td>
<td>3.82 x 10^6</td>
</tr>
<tr>
<td>13, 14, 15, 16, 17, 18</td>
<td>65</td>
<td>.87</td>
<td>450</td>
<td>14.20 x 10^6</td>
<td>3.54 x 10^6</td>
<td>6</td>
</tr>
<tr>
<td>19, 20, 21, 22, 23, 24</td>
<td>50</td>
<td>.86</td>
<td>450</td>
<td>13.96 x 10^6</td>
<td>3.82 x 10^6</td>
<td>6</td>
</tr>
<tr>
<td>Colorado River at Compact Point (Lee Ferry) using Framework I study</td>
<td>1, 2, 9, 13, 14, 15</td>
<td>17, 18, 19, 20, 22</td>
<td>47</td>
<td>.91</td>
<td>450</td>
<td>13.06 x 10^6</td>
</tr>
<tr>
<td>Data for calibration</td>
<td>24, 25</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
we used a method of spatial and temporal weighting where in eigenvectors are extracted from a correlation matrix for the suite of tree-ring series to be utilized and for which each has been lagged 3 times. For example, in one of the reconstruction problems for the Colorado River at Lee Ferry, we used 17 tree-ring sites, and when each is lagged by 3, the resulting matrix of data consists of 68 variables. The resulting eigenvectors are then used to weight the original series, the result being called principal components or amplitudes. The resultant weighted value has the desirable property of being orthogonal. In addition, as long as the variance that the eigenvector accounts for is large enough, the resultant weighting usually is physically reasonable. As the covariance diminishes, the eigenvectors are still orthogonal but probably have no physical relationships, as the orthogonality constraint becomes overriding. In all cases, only eigenvectors with corresponding roots greater than 1.00 and accounting for a greater percentage of the variance than would be expected from a matrix of a comparable number of random series were used.

The reconstruction equations were established for each model shown in Table 2 by using least squares analysis, in which the individual orthogonal variables were evaluated before they were entered into the equation. If the F value did not exceed 3.00, the variable was not used in the equation.

The streamflow data used for the reconstruction of the virgin flow at the Compact Point (Lee Ferry$^2$) are from the Upper Colorado River Commission (Hely, 1969, p. 49) and the Comprehensive Framework Study (Framework I) Upper Colorado Region (Water Resources Council, 1971). These figures are the measured flow with estimated Upper Basin depletions restored to the flow and represent the virgin flow at the Colorado River Compact Point, which is 1 mile downstream from the mouth of the Paria River. There is no gage at this point. The actual

$^2$The Compact Point is termed "Lee Ferry" in the Colorado River Compact and other legal documents.
flow at the compact point is computed as the sum of the Colorado River flow at Lee Ferry and the Paria River at Lee Ferry. The latter flow is measured 1/2 mile above the mouth. Because the Compact Point is the accounting point between the Upper and Lower Basins, it is extremely important to try to determine the average undepleted or virgin flow at this location.

There has been a recording gage on the Colorado River at Lee Ferry since January 19, 1923. From June 13, 1921, to that date, reference stakes and staff gages were used to determine flow, and these measurements were referenced to the present gaging site. Prior to June of 1921 there was no gaging at Lee Ferry, and the earlier data are based on extrapolations from other records at other stations in the Colorado River Basin. For the 1914 water year on, figures are available for the three major tributary stations, and these figures have been used to estimate the actual flow at Lee Ferry and the Lee Ferry compact point. Regression analysis showed that the flow at the Lee Ferry gage can be accurately computed as a fraction of the three major tributary gages. Thus the total flow data from 1914 on is assumed to be accurate enough for the reconstruction analysis. This year was used as the starting point for two of the reconstruction analyses (Table 3 and Figure 4).

The streamflow data from 1896 to 1914 are probably less accurate. This longer record was also used in a reconstruction (Table 3 and Figure 4).

Estimates of various depletions or consumptive uses pose some serious problems. Extragbasin diversions and changes in reservoir storage can be quantified fairly accurately by at-site measurements. However, evaporation and bank-storage determinations at major reservoirs are subject to some uncertainties. Also, other consumptive uses, primarily for irrigation, are not accurately measured in many cases and must be estimated. In 1962 the extrabasin diversions were on the order of 0.5 MAF, and other consumptive uses were about 2.00 MAF. With a long-term reconstructed virgin runoff of 13.5 MAF, an error of 20 percent in estimated 1962 Upper Basin depletions would be 0.56 MAF, or only 4 percent of the reconstructed figure.
Figure 4. Reconstructed hydrographs for the Colorado River at Lee Ferry (Compact Point), based on:
(A) a 50-year calibration record (Framework I study data) and a 13-station tree-ring data grid,
(B) a 50-year calibration record (Upper Colorado River Commission data) and a 17-station data grid,
(C) a 65-year calibration record (Upper Colorado River Commission data) and a 17-station data grid. Table 3 gives corresponding tree-ring data series included in each case; Figure 1 shows relative locations.
Runoff reconstructions at Lee Ferry, Arizona
(Compact Point)

We have reconstructed the total annual runoff at Lee Ferry, Arizona, using three different models, incorporating two different tree-ring data grids and two versions of virgin flow records for calibration. The hydrographs are shown in Figures 4 and 5. The models were varied on the basis of percentage variance accounted for in the calibration record, unbiasedness in the residuals, and ability to duplicate data not used in the calibration equation. The data used for calibration consisted of that from Table 6, page D49, USGS Prof. Paper 486-D. The records of actual flow for 1896-1913 and records of virgin flow for 1896-1945 were published by the U.S. Bureau of Reclamation (1954, pp. 145-146). Records of virgin flow for 1946-66 were furnished by the Upper Colorado River Commission. The other data source was from Table II from the Comprehensive Framework Study of the Upper Colorado River Basin and covered the period 1914-1965. Both sets of data represent the estimated virgin outflow from the Upper Basin. The mean and standard deviation of the data from Prof. Paper 486-D are 14.65 and 4.45 million acre-feet for the same period. For the 65-year period 1899-1963 (data from Prof. Paper 486-D), the mean is 15.09 million acre-feet. The tree-ring data grids utilized consisted of subsets with 13 and 17 tree-ring sites. The numbers of the sites used in each case are shown in Table 3 and the relative locations are shown in Figure 2.

The 65-year calibration period includes a portion of the historical record that was estimated from a longer flow record upstream. There is some question as to whether this data should be used in a calibration equation or not. However, it does include some of the larger flow years which are desirable for inclusion in the calibration equation. To check the reliability of the 65-year calibration equation, we computed another equation using only 50 years of data (1914-1963) and compared the reconstructive qualities with the published data covering the period 1896-1914. The reconstruction equations are as follows.
Figure 5. Same as Figure 4 except that data has been filtered to accentuate the low-frequency variance.
For 65-year calibration period:

\[ f_t = 14.15 - 0.589E_1 - 0.549E_2 - 0.753E_3 - 0.634E_5 - 0.831E_{10} - 0.778E_{15} + 0.542E_{22} + 0.849E_{27} + 0.844E_{29} + \text{error} \]  

where

\[ f_t = \text{reconstructed total annual runoff for year } t \]

\[ E_i = \text{ith principal component from appropriate tree-ring data grid.} \]

This equation accounts for 75 percent of the variance in the 65-year historical record.

For 50-year calibration period:

\[ f_t = 13.94 - 0.616E_1 - 0.781E_2 - 8.89E_3 - 0.701E_5 - 6.41E_{10} - 0.992E_{15} + \text{error} \]

This equation accounts for 78 percent of the variance in the 50-year historical record.

The six variables entered into Equation (2) are the same as the first six entered into Equation (1). The relative weights of the coefficients are only slightly different in the two cases. Figure 6 illustrates how Equation (1) duplicates the historically estimated data for the period 1896-1914 as compared to that for Equation (2) and Equation (3). Equations (1) and (2) are quite similar in their duplication of the historical data with the mean for the 19 years of data reconstructed by Equation (1) being 15.6 and standard deviation 3.3 whereas that for Equation (2) is 14.6 and 3.8 million acre-feet. The historical estimated record has a mean for the period of 15.80 million acre-feet and a standard deviation of 3.87 million acre-feet. The overall reconstruction seems to be unbiased in that for Equation (1) the reconstructed values are greater than the historical values 9 times and are less 9 times. For Equation (2) the reconstructed values exceed the historical values 8 times and are less 10 times.

Using Framework I study data and a slightly modified tree-ring data grid (see Table 3), the reconstruction equation becomes:
Figure 6. Comparison of the historical estimate of flow at Lee Ferry (1996-1914) with estimates based on tree-ring data using a 65-year calibration equation (Eq. 1) a 50-year calibration equation (Eq. 2), and the Framework I data calibration equation.

\[ f_t = 13.06 - 0.596E_1 - 0.506E_2 - 1.055E_3 - 0.508E_4 + 0.468E_7 - 0.573E_11 + 0.828E_14 - 0.704E_15 \] (3)

and accounts for 82 percent of the variance in the calibration record.

The long-term mean for this record is 13.06 million acre-feet and the variance is 3.46 million acre-feet. When compared to the independent data for the period 1896-1914 (Figure 6), the tendency is for slight biasness in underestimation, as for the 18 years there are 11 underestimates and 7 overestimates. The resultant mean is 13.5 million acre-feet and the standard deviation is 3.4, both considerably less than for the historically estimated data and Equations (1) and (2).
The autocorrelation structure in all three cases appears quite similar with the first order autocorrelation being approximately .33. The structure of the autocorrelation has not been analyzed yet, but judging from the correlograms (Figure 7), the structure is more complex than that of a simple autoregressive model, probably mixed autoregressive-moving average.

We have not yet computed the sample variance spectra (i.e., the distribution of variance with respect to frequency) for any of the three reconstructions included here. For an earlier version which would be similar to those above based on the data from USGS Professional Paper 486-D, Stockton (1975) computed variance spectra for both the tree-ring reconstructed data and the historically gaged data for the period 1896-1961.

Figure 7. Comparative correlograms for the gaged record and the three reconstructed records of flow of the Colorado River at Lee Ferry.
and the long-term tree-ring data reconstructed record for the period 1564-1961. Figure 8 shows the ability of the tree-ring data to duplicate the frequency distribution in the gaged record. One would expect a similar degree of comparison if any of the three reconstructions included here were similarly analyzed. Figure 9 shows the distribution of variance with respect to frequency in the long-term reconstructed record. Again, one would anticipate a similar type of spectrum from any of the three reconstructions above.

Figures 8 and 9 illustrate two important points. First, using our techniques for reconstructing runoff from the Upper Colorado River Basin using tree-ring data, we are able to duplicate the distribution of variance with respect to frequency in the gaged record very well. Second, the long-term spectrum (Figure 9) shows considerably more evidence for long-term variation in flow than exists in the gaged record (Figure 8), suggesting an inadequate length of record for the gaged series.

The question arises as to which of the three reconstructions of past runoff at Lee Ferry is the best. Our reasoning is as follows. Comparison of the three hydrographs (Figures 4 and 5) shows very little difference among the three. The reconstruction based on the 65-year calibration record (Eq. 1, graph C, Figures 4 and 5) is based on data that include 19 years of estimated record (1896-1914) that is questionable in terms of calibration. The reconstruction based on the 50-year calibration (Eq. 2, graph B, Figures 4 and 5) does not contain the drawback of Equation (1) and seems to be comparable to that for Equation (1). When a slightly different tree-ring grid and calibration data from the Comprehensive Framework Study (the most recent evaluation of virgin runoff) are used (Eq. 3, graph A, Figures 4 and 5), a slightly different reconstruction is obtained. Therefore, we feel that the best estimate of the long-term reconstruction is an average of the results of Equations (2) and (3). Consequently, we arrive at an estimated mean annual runoff of 13.5 million acre-feet.

For purposes of comparison among subbasins, the Upper Basin was divided into the traditional tributary subdivisions of Green River.
Figure 8. Autospectra for the gaged record at Lee Ferry versus that for the same period (1896-1961) as reconstructed from tree-rings. This figure shows the fidelity with which the tree-ring data duplicate the variance of the gaged data over the entire frequency range (after Stockton, 1975).
Figure 9. Autospectrum of long-term Lee Ferry record reconstructed from tree-ring data for period 1564-1961 (after Stockton, 1975).
above Green River, Utah, the Colorado Main Stem above Cisco, Utah, and the San Juan above Bluff, Utah. This allows assessment of any preferred mode of occurrence of either high or low flows. Table 3 lists the individual reconstructed records from each subbasin, the tree-ring sites used for the reconstruction, number of years in the calibration period, predominant correlation coefficient for comparison of the tree-ring data and the runoff series, number of years in the reconstructed record, the long-term average flow as interpreted from the reconstructed record, the long-term standard deviation, and finally the model number utilized in the reconstructed record (see Table 2 for model designations). Some of the records utilized in the reconstructions were based on unadjusted historical runoff records; consequently the mean annual flow figures (especially that for the Colorado above Cisco) are probably slightly low. For most of the smaller basins these divisions will probably not substantially affect the mean because most of the stations chosen for reconstruction were chosen partly on the basis of lack of upstream diversions.

The Green River Basin

Within the Green River Basin four reconstructions were made. These include Green River at Green River, Utah, Green River near Boulder, Wyoming, and Whiterocks River near Whiterocks, Utah. These stations were chosen for reconstruction because of (1) their fairly long, homogenous historical record, which provided a reliable record for calibration, (2) their location relative to existing or potential dendrochronology sites, and (3) their location within known high runoff producing areas.

Plots of the reconstructed records and their comparison show some interesting aspects. In general, the northernmost records, that is, the Green at Daniel and the New Fork, do not show the pronounced low frequency variation that is exhibited by the Whiterocks reconstruction nor the Green River at Green River reconstruction. Of specific
note is the fact that neither the reconstruction for the Green at Daniel nor the New Fork show the pronounced dominant trend since the early 1900's nor do they show the pronounced high flow period during the early 1900's. All three--the Green at Daniel, the New Fork, and the Whiterocks--do not indicate the pronounced low flow period during 1870-1890. However, the reconstruction for the Green River at Green River (Figure 10) shows a pronounced low flow period during the period 1870-1900. But this reconstruction includes three tree-ring series from sites in the southern part of the Green River Basin that are not utilized in any of the smaller northernmost subbasin reconstructions. This seems to indicate that (1) the northernmost portion of the Green River drainage is affected by climatic trends which are different from the southerly part of the Green River Basin and probably the whole Upper Colorado River Basin, (2) the Whiterocks reconstruction seems to show some of the same low frequency components as those of the northern part of the basin, but also some characteristics of the southern part, and (3) the Green River at Green River, Utah, reconstruction shows low frequency variations quite different from those of the northern part of the basin and also of the Uinta Mountains. Specifically, the drought of the late 1800's is more pronounced, the wet period from 1907-1932 is more pronounced, and the overall downward trend since 1932 is more pronounced.

Except for the reconstructions for the New Fork River, where the long term average is 287,000 versus that for the gaged 39-year record of 284,000 and the Green near Daniel, Wyoming, where the 39-year average of gaged value is 366,000 acre-feet versus 385,000 for the reconstructed record, the long term average runoff values from the reconstructed records are less than for the gaged records. For the Green River at Green River, Utah (Figure 10) and 71-year gaged record is 4,614,000 acre-feet whereas the reconstructed 392 year value is 4,480,000 acre-feet. The Whiterocks River average for 63 years of gaged data is 90,560 and that for the reconstructed record is
Figure 10. Reconstructed hydrograph for total annual runoff for the Green River at Green River, Utah. Upper graph is for unfiltered data; lower graph is for the same data after removal of high-frequency components (those with a period of less than 10 years).
88,700 acre-feet. It seems apparent that the large scale fluctuations in the southerly portion of the basin, particularly the abnormally high runoff in the early 1900's and the no-analogy drought periods such as occurred in the late 1890's has caused the mean annual runoff estimated from the historical record to be inflated. In records from the northerly part of the basin where these anomalies do not exist, the long-term reconstructed means seem to be greater than that for the measured flow suggesting the lack of inflation in the historic mean due to the anomalous wet period in the early 1900's.

The Colorado Mainstem above Cisco, Utah

Within the subbasin drained by the Upper Colorado River mainstem above the gaging station at Cisco, Utah, we have reconstructed six station records. The reconstructed record at Cisco (Figure 11) incorporates the long-term trends for both the Upper Mainstem and the Gunnison River tributaries and shows predominant high flow years during the period 1916-1932 preceded by a prolonged period of low flow from about 1873-1912. The long-term mean annual flow is 4.26 MAF as opposed to 5.59 for the 59 year historical record. Apparently the anomalously high-flow years during the 1920's tend to inflate the mean above what the long-term data seem to indicate. Those years are the largest block of continuously high flow years in the entire 321-year reconstructed record.

The anomalously high-flow period does not appear in all the smaller basin reconstructions within the larger subbasin, however. The Colorado at Cameo reconstruction does not show these predominant high-flow years. In this case it might be a result of the period of calibration being too short to include those high-flow years as only 28 years were used in the calibration. The reconstruction for the Frazer River differs significantly from the others in that it does not show the extended period of drought during the late 1800's. The long term mean for the reconstructed record at Cameo is 2.82 MAF whereas
Figure 11. Reconstructed hydrograph for total annual runoff for the Colorado River at Cisco, Utah. Upper graph is for unfiltered data; lower graph is for the same data after removal of high-frequency components (those with a period of less than 10 years).
that for the historically gaged record is 2.78 MAF and is affected by transmountain diversions, storage reservoirs, power developments, and irrigation diversions. Those factors might also explain the lack of long-term variation in the reconstructed record because of the lack of them in the calibration record. The long-term mean annual runoff for the reconstructed record for the Frazer River is 27,700 acre-feet as opposed to about 29,000 for the gaged record.

Both of the long-term reconstructed records for the Gunnison near Grand Junction, Colorado and the Taylor near Almont, Colorado show a large block of persistently high-flow years during the period 1907-1932 and each show this period being preceded by a large block of persistent low-flow years during the period 1870-1900. Equally important however, is that there appear to be earlier periods of comparable prolonged high-flow years. The long-term mean annual flow for the Taylor River is 252,600 acre-feet as opposed to 246,300 for the gaged record. That for the Gunnison near Grand Junction, Colorado is 2.13 MAF as compared to 1.86 MAF for the 62-year gaged record.

The Delores River reconstructed record shows a long-term mean of 298,000 acre-feet and the gaged record 311,000 acre-feet.

The San Juan River

Because of lack of tree-ring data sites within the San Juan River drainage, the only reconstruction attempted for this basin was for the San Juan near Bluff, Utah (Figure 12). This record shows the high-flow period of 1907-1932 as the longest sustained period of high-flow during the past 360 years. The mean annual flow for the reconstructed record is 2.20 MAF as opposed to 1.89 MAF for the unadjusted historical flow record.

Comparison of Green, Upper Mainstem, and San Juan Reconstructions

The long-term flow characteristics of some of the smaller watersheds have been pointed out. It is important to investigate the
Figure 12. Reconstructed hydrograph for total annual runoff for the San Juan River near Bluff, Utah. Upper graph is for unfiltered data; lower graph is for the same data after removal of high-frequency components (those with a period of less than 10 years).

significance of these on the larger subbasin runoff. For this reason we have compared the sample variance spectra and squared coherency spectra for the Green at Green River, Utah, the Colorado near Cisco, Utah, and the San Juan near Bluff, Utah.

The sample variance spectra are shown in Figure 13. Note that the frequency distribution of the San Juan and Colorado Mainstem are remarkably similar, with the Colorado being consistently and uniformly greater over the entire frequency range. The Green River spectrum, however, is concentrated on the low-frequency end (period greater than about 20 years) and rapidly decreases as it approaches the high-frequency end (period of 2 years). Consequently it is obvious that the Green River reconstruction contains considerably more low-frequency variation than that for either the San Juan or Colorado above Cisco, Utah.
Figure 13. Comparison of the sample autospectral functions for the long-term reconstructed runoff records for the Green River at Green River, Utah, the Colorado Mainstem at Cisco, Utah, and the San Juan River at Bluff, Utah.

The squared coherency spectra show how the individual squares of the series are covarying in time and can be thought of as the correlation coefficient defined at each frequency. That for the San Juan and
the Colorado at Cisco (Figure 14a) shows a fairly even distribution across the entire frequency range with perhaps a slightly higher average in the range from 4.5 to 2.0 years. The highest coherency is shown in the frequency range of about 7 to 2.5 years in the comparison of the Green runoff series with that for the Colorado at Cisco (Figure 14b). The average is about 0.35 but decreases from the low-frequency range to the high. It appears that, although the San Juan and Colorado are similar and unlike the Green with respect to frequency distribution of variance (Figure 13), the Green and Colorado covary more similarly than either the Colorado and San Juan (Figure 14a) or the Green and San Juan (Figure 14c). In none of the three cases is the coherency very large over the entire frequency range; it ranges from an average of about 0.35 to about 0.20.

In the filtered series of the long-term reconstructions (Figure 15), the low-frequency variation is accentuated, and it is easier to visually compare the time series. The filtered series show some interesting similarities and dissimilarities. All three series show the predominant downward trend from 1932 to 1961. The flow of the San Juan has been below the long-term mean from about 1945 to 1968. The Upper Colorado (above Cisco) also shows this prolonged period of below-normal flow except for two short periods, during the late forties-early fifties, and in the late fifties, during which the flow was above normal. The Green River (above Green River, Utah) shows below-normal flows during the period 1954-1961. Thus all three major tributaries reflect below-normal flow, starting as early as 1945 in the San Juan and as late as 1954 in the Green. The severest low-flow is reflected in the San Juan reconstructed hydrograph.

All three show the pronounced wet period during the early part of the twentieth century, and in each case it is the longest continuous period of high-flow years in the entire reconstructed hydrograph. All three also show this extended wet period ending about 1933, but the data when the high flows began varies from 1903 for the Green to 1907
Figure 14a. Squared coherency spectra for long-term reconstructed runoff records, showing coherency between the San Juan at Bluff and the Colorado at Cisco.

Figure 14b. Squared coherency spectra for long-term reconstructed runoff records, showing coherency between the Green at Green River and the Colorado at Cisco.
Figure 14c. Squared coherency spectra for long-term reconstructed runoff records, showing coherency between the Green and San Juan. The Green and San Juan reconstructions are based on calibration with virgin flow records; that for the Colorado at Cisco is based on the gaged record.

Figure 15. Comparison of the filtered runoff series for the Green River at Green River, Utah, the Colorado River at Cisco, Utah, and the San Juan River near Bluff, Utah. In the filtered series, the long-term variation is better displayed.
for the San Juan to 1911 for the Colorado. None of the three show evidence of a severe extended low-flow period during the drought of the 1930’s.

In all three cases, the extended wet period was preceded by a period of long and severe low flow. It appears to have been severest on the Green and was interrupted by an above-average flow period from 1885-1894 on the San Juan. No analogous long duration low-flow periods have occurred since the beginning of the historical gaged records.

Figure 15 shows periods during which the runoff from all three subbasins appears to have been in synchrony and other periods when one or the other did not agree with the third. Of particular interest is the period 1685-1735, when a high sustained flow occurred on the Green and San Juan but not on the Upper Mainstem. This high-flow period, the only one in the reconstructed record that is at all comparable to the high-flow years of the early 1900’s, apparently occurred only in the San Juan and Green River basins. Not shown on Figure 15 but also of special interest is a severe extended low-flow period on the Green during the period 1578-1605. No other period of such severe drought is found in the reconstructed record. Unfortunately, the other two reconstructed records do not go back far enough to cover this time period, so it is not possible to tell whether the drought was as severe and prolonged in the other two basins.

Summary

We have completed long term tree-ring reconstructions of total annual flow for 12 different stations within the Upper Colorado River Basin. On a short term basis, our tree-ring reconstructed series show comparable trends and synchrony of high and low periods in correspondence with the gage records. Many of these same trends are also noted in the tree-ring data series. For example, the tree-ring data series and selected runoff series for the Wind River Mountains
area in the Green River Basin do not exhibit the noticeable downward trend from the 1920's to the present. This represents a considerable difference from the noticeable trend in other records within the basin.

Three long-term (450 years) reconstructions have been computed for the Colorado River at the Compact Point (Lee Ferry). It is reasoned that the best of these is probably an average of two of them and results in an estimated mean annual runoff of 13.5 million acre-feet. This is not inconsistent with results obtained by others as reportedly, some federal agencies have been using this figure, arriving at the value by other methods of analysis (Jorgenson, 1975). All three hydrographs show: a) the period of about 1907-1930 to be the longest period of conservation by high-flow years in the entire 450 years of reconstructed renewal. Only one other period in the early 1600's is even closely comparable; b) the low flow periods from 1868-1892 and 1564-1600 are of longer duration and greater magnitude than for any period during the gaged record.

Between the three subbasins drained by the Green River, the Colorado Mainstem, and the San Juan, our reconstructions show similarities such as the abnormally high runoff period during the early 1900's and the no-analogy drought periods such as occurred in the late 1800's. All three show a predominant downward trend from 1925 to present. This appears to be the most pronounced trend in the entire reconstructed period. There are also some noticeable dissimilarities. For example, the low flow period during the late 1800's was most severe on the Green River and least on the San Juan. Also, during the period 1685-1735, a period of sustained high-flow occurred on the Green and San Juan but not on the Upper Mainstem.

**Implication for surface-water supply and water level of Lake Powell**

The figure of 13.5 MAF/yr runoff from the Upper Colorado River Basin takes on great significance when placed in the context of the Law of the River, increasing consumptive use in the Upper Basin.
and operation of Glen Canyon Dam. Although the Colorado River Com-
 pact of 1922 apportioned 7.5 MAF/yr to both the Upper and Lower
 Basins (Art. III, Section a), it also contains a section preventing the
 Upper Basin from interfering with the delivery to the Lower Basin of
 75 MAF each decade (Art. III, Section d). This is an annual average
 of 7.5 MAF/yr. In times of deficiency the Upper Basin also must
 furnish half of the Mexican Treaty apportionment of 1.5 MAF/yr or
 0.75 MAF/yr. This treaty apportionment is a national obligation but
 unit the federal government provides the water it remains an obliga-
tion of the Upper and Lower Basins (Colorado River Basin Project Act
 of 1968, Sec. 202). If one subtracts these two downstream obligations
 from the figure of 13.5 MAF, the amount available for Upper Basin
 consumptive use is 5.25 MAF/yr. This amount is already oversub-
scribed in that it is covered by vested water rights (or water-right
 applications), contractually committed, officially reserved or un-
officially projected for designated potential use.

Two phenomena have been taking place in the recent past. The
consumptive use in the Upper Basin has been increasing and the esti-
mates of surface-water supply have been decreasing. These factors
are shown in Figure 16. The planned consumptive uses shown on
this figure will probably not occur as rapidly as the curves imply be-
cause certain projects have been delayed or postponed. However the
general picture of a collision between demand and supply in the not too
distant future is all too apparent. Water storage will serve to delay
the time of actual shortage beyond that when demand meets supply,
but at that point, new consumptive uses can only be undertaken by
shifting water away from then current uses or by flow augmentation.

Also, at this point in time Lake Powell will be used to reduce
flows to the Lower Basin to the legal minimum and store as much
excess as possible in wetter years. In drier years, releases from
the lake will meet only the legal requirements. Thus the major factor
in reservoir management is likely to be control of surface-water supply
and other factors such as power generation and recreation may become secondary to this control.

Figure 16. Surface water available for consumptive use in the Upper Colorado River Basin and relationship to projected demands for future energy development. Stippled zone represents the most likely level of surface-water supply; the 5.25 MAF value is based on the estimated supply of 13.5 MAF/yr (after Weatherford and Jacoby, 1975).
References Cited


APPLYING A HYDRO-SALINITY MODEL TO THREE SUBBASINS WITHIN THE COLORADO RIVER BASIN

by

A. Leon Huber and V. A. Narasimhan*

Introduction

Salinity or the total dissolved solids content of the surface water has been identified as the most critical problem in the Colorado River Basin and is a significant factor in most river basins of the western U.S. where irrigation is practiced. River basin computer modeling has been adopted as a technique to study salinity management of irrigation return flows; however, the applications reported hereafter raised some questions about the validity of some model assumptions that have been commonly accepted.

A basic assumption of the various models used for studies of the Colorado River Basin is that the salt pickup is directly proportional to the amount of percolating water. This implies that an equilibrium soil-water concentration is rapidly reached and maintained for each time increment of the model, typically one month. This hypothesis seems to fit well where irrigation is practiced year around such as in Southern California and Arizona but does not account for the concentration build-up during the non-irrigation season typical of the agricultural effluent in the Upper Basin of the Colorado River. The alternative hypothesis that salt pickup is a function of time and that solubilization takes place at a constant rate regardless of the amount of percolating water does result in an increased concentration during the non-irrigation season similar to that observed in the field data from the Grand Valley near Grand Junction, Colorado. It is not difficult to

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calibrate a computer model with the observed quality and quantity data of surface outflows based on either assumption by suitably varying the corresponding model parameters. However, predictive results under various imposed management options will be widely different depending on which assumption is used. The model must accurately predict the management effects to be useful. Otherwise, it may mislead those who must make decisions concerning multimillion dollar projects.

The assumption of salt pickup being proportional to the percolating water, the assumption used in this study, gives the results that the management of agricultural water can reduce the salt outflow. The use of an alternative hypothesis that salt generation is constant regardless of the percolating water would show little improvement in the total salt outflow. It is likely that neither of these assumptions is completely correct, and that the actual mechanisms for salt would be expected to vary widely from basin to basin. Consequently, until further research is conducted and the actual salt pickup processes operating in each area are identified, a full assessment of various management options is impossible. The U.S. Bureau of Reclamation and Colorado State University have studies underway in the Grand Valley area that hopefully will furnish data that may be used to resolve this problem.

Studies on salt pickup and precipitation in soil profiles

Figure 1 shows the analysis of the composition of irrigation water and of drainage water for the Palo Verde and Grand Valley areas. In the Palo Verde area an average of about 4 percent of CaCO\(_3\) and 15 percent of CaSO\(_4\) precipitates within the soil profile. In the Grand Valley area, however, it is observed that about 22 percent of CaCO\(_3\) precipitates while there is 44 percent of CaSO\(_4\) solubilization taking place. The Na content increases in drainage waters in Palo Verde, but decreases in the case of Grand Valley. Figure 2 shows a typical
### Proportion of ions in Meq/L

#### Palo Verde: Water year 1971

<table>
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<th></th>
<th>Ca</th>
<th>Mg</th>
<th>Na</th>
<th>K</th>
<th>HCO$_3$ +CO$_3$</th>
<th>SO$_4$</th>
<th>Cl</th>
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<td>.407</td>
<td>.010</td>
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<tr>
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</table>

$^1$Implies that about 4.2 percent CaCO$_3$ precipitates

$^2$Implies that about 15.5 percent CaSO$_4$ precipitates

#### Grand Valley: (5-5-72 to 9-6-74)

<table>
<thead>
<tr>
<th></th>
<th>Ca</th>
<th>Mg</th>
<th>Na</th>
<th>K</th>
<th>HCO$_3$ +CO$_3$</th>
<th>SO$_4$</th>
<th>Cl</th>
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<td>.299</td>
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<tr>
<td>Average comp</td>
<td>.317</td>
<td>.332</td>
<td>.346</td>
<td>.005</td>
<td>.116</td>
<td>.744</td>
<td>.140</td>
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<tr>
<td>Percent change</td>
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<td>-21.9</td>
<td>-58.3</td>
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<td>148.8$^2$</td>
<td>-61.0</td>
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</table>

$^1$Implies that about 22.6 percent CaCO$_3$ precipitates

$^2$Implies that about 44.5 percent CaSO$_4$ solubilizes

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Figure 1. Analysis of inflow and drainage water for the Palo Verde, California, and the Grand Valley, Colorado, subbasins showing precipitation and solubilization of bicarbonate and gypsum salts.
Figure 2. A typical graph of the salt outflow from a drain in the Grand Valley area.

graph of the quantity of salt outflow through drains in the Grand Valley area in tons/day versus a corresponding water outflow in cfs. While many complex phenomena may account for salt outflow through drains, it is seen from Figure 2 that it may be possible to represent the salt flow by a typical yield curve.

Results of the Two-Dimensional Hydro-Salinity Study of Agricultural Impact on Colorado River Salinity

The computer model (BASIM) was used to aid in evaluating the water quality salinity impacts of irrigation management levels. Three subbasins were selected for modeling purposes—the Palo Verde Irrigation District, California, the Grand Valley, Colorado, and Duchesne Basin of Utah. The model computes various hydrologic quantities before the corresponding salt quantities are calculated. This is accomplished by determining concentrations of the flows, including the salt.
pickup, and computing the total salt flow from the basin. The model was calibrated using the observed data from the subbasin where sufficient record is available for the component flows. The results showed a close agreement between the predicted quantities and the observed data with respect to the quantity of flows (see Figure 3 for Grand Valley); however, it did not shown a corresponding agreement with respect to the concentrations of drain water. The disagreement is attributed to the simplifying assumption in representing the salt pickup.

Procedure

The following study procedure was used for each of the areas:

1. Calibrate the model using 1970-72 water year data, as available, for each subbasin.
2. Determine a base predicted runoff of the river downstream of the area in which management alternatives are to be tested. Parameters and coefficients are set at the values determined in step (1) above.
3. Impose the selected management levels on the model by changing the appropriate model parameters and evaluate the results. The management levels applied in the BASIM model were:

   Level E_1: Present canal conveyance efficiency with a higher level of application efficiency achieved by better management of water application without any capital investment.

   Level E_2: Increased canal conveyance efficiency achieved by canal lining and the same application efficiency as currently exists.

   Level E_3: Increased canal conveyance efficiency coupled with the highest technologically feasible application efficiency that might be obtained by management as well as capital investment.
Figure 3. Grand Valley calibration results for the water year 1970-1972.
General comparison of results

A summary of the results for each subbasin is given in Table 1 for the Duchesne, the Grand Valley, and the Palo Verde areas. An assessment of the annual results shows that in each of the three subbasins there appears a general decrease in the salt loading in surface outflow for the three assumed irrigation control levels. The reduction in salt loading is largest in the Duchesne basin, lesser in Grand Valley area, and is inappreciable in the case of Palo Verde irrigation district.

Detailed description of model results

Palo Verde study area. The computed outflow represents the surface runoff, flow from the drains, operational spills, and tailwater runoff. Figure 4 depicts the results of the management runs for the total surface outflow. It indicates that the base line situation representing current irrigation practice may not be greatly improved by the management levels tested. In relation to this, the model assumes a certain minimum value of application efficiency for each time increment (one month in this case). The base line situation has different efficiencies each month varying from 10 percent to 68 percent throughout the year. The model selects the minimum of the specified control level efficiency and the historic efficiency. The variability in historical efficiency is very likely a significant factor contributing to the salt loading of the base line system. This would suggest that operational scheduling may indeed by a very important means of reducing the salt pickup in such systems. The development of a model to test this management alternative is strongly recommended. The objective of such a scheduling model would be somewhat different than a traditional irrigation scheduling model. The approach would be to accumulate salt in the soil profile during some periods and then flush it out at other times. The objective would be to minimize the total impact on the river system while still maintaining a salt balance in the
Table 1. Management effects on salt loading from irrigated agriculture for three subareas as simulated by BASIM using 1972 data.

| Cases      | Study Areas | Duchesne ECV | EAP | Canal Diversion | Surface Effluent | Surface Outflow | Grand Valley ECV | EAP | Canal Diversion | Surface Effluent | Surface Outflow | Pale Verge ECV | EAP | Canal Diversion | Surface Effluent | Surface Outflow |
|------------|-------------|--------------|-----|----------------|------------------|-----------------|------------------|------------------|-----|----------------|------------------|-----------------|----------------|-----|----------------|------------------|-----------------|
| Baseline   |             | .72          | .76 | .76            |                  |                 | .72              | .76              | .97 | .97            | .97              |                 |                 |     |                |                   |                 |
|            | Water       | 594          | 79  | 367            | 641              | 170             | 3491             | 909              | 439 | 5844           |                  |                 |                 |     |                |                   |                 |
|            | Salt        | 438          | 150 | 374            | 546              | 750             | 3256             | 977              | 1086 | 6877          |                  |                 |                 |     |                |                   |                 |
| Case I     |             | .72          | .76 | .65            | 534              | 50              | 381              | NA               |     |                |                   |                 |                 |     |                |                   |                 |
|            | Water       | 534          | 50  | 381            | 531              | 112             | 3517             |                  |     |                |                   |                 |                 |     |                |                   |                 |
|            | Salt        | 389          | 108 | 349            | 451              | 532             | 3113             |                  |     |                |                   |                 |                 |     |                |                   |                 |
| Reduction  | in salt      | 11.2         | 28  | 6.68           | 17.4             | 29              | 4.39             |                  |     |                |                   |                 |                 |     |                |                   |                 |
| Case II    |             | .95          | .95 | .65            | 407              | 42              | 387              | 626              | 203 | 5870           |                  |                 |                 |     |                |                   |                 |
|            | Water       | 407          | 42  | 387            | 407              | 44              | 5556             | 665              | 614 | 6701           |                  |                 |                 |     |                |                   |                 |
|            | Salt        | 139          | 81  | 287            | 346              | 187             | 2861             | 31.9             | 43.46 | 2.56         |                  |                 |                 |     |                |                   |                 |
| Reduction  | in salt      | 68.3         | 46  | 23.3           | 36.6             | 75              | 12.1             |                  |     |                |                   |                 |                 |     |                |                   |                 |
| Case III   |             | .95          | .95 | .80            | 380              | 26              | 395              | 373              | 28   | 3565           | 516              | 133             | 5902           |     |                |                   |                 |
|            | Water       | 380          | 26  | 395            | 373              | 28              | 3565             | 516              | 133 | 5902           |                  |                 |                 |     |                |                   |                 |
|            | Salt        | 130          | 54  | 268            | 318              | 130             | 2825             | 548              | 441 | 6639           |                  |                 |                 |     |                |                   |                 |
| Reduction  | in salt      | 70.3         | 64  | 28.3           | 41.75            | 82.7            | 13.23            | 43.91            | 59.39 | 3.46         |                  |                 |                 |     |                |                   |                 |

* 1972 Historical application efficiencies.

a EAP is the application efficiency defined as the ratio between the crop evapotranspiration and the irrigation water delivered to the land for that purpose. For the various management cases, the water diverted was limited to that amount used historically, i.e., if the baseline practice was already at the selected efficiency level, that efficiency would prevail that month.

b Canal diversion includes the seepage return.

c Percent reduction in salt outflow for each case is computed with reference to baseline salt quantity.
Figure 4. Predicted water and salt outflows resulting from irrigation management alternatives applied to the Palo Verde sub-basin for water year 1972.
agricultural domain. The model would have to consider the buffering effect of reservoirs (if any) and upstream and downstream diversions as well.

Grand Valley study area. The results of the model for the Grand Valley are shown in Figure 5. The results for the various irrigation management levels showed that all three options could improve the annual loading of the river. However, the mechanisms of the sources of salt pickup in this area are still being investigated as to the relative importance of canal seepage, irrigation leaching water, and weathering by groundwater. Even under assumed conditions of maximum efficiency, the quantity of seepage water is greater than the expected quantity of deep percolation. Studies are under way by the Bureau of Reclamation and Colorado State University that may help to resolve some of the differences in hypotheses. An effort to evaluate the adequacy of the 1972 data was made by operating the model with three years of data, 1970, 1971, and 1972. The results are tabulated in Table 2, and appear to be consistent.

Table 2. Grand Valley management tests with data for 1970-1972.

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<tr>
<th></th>
<th>1970</th>
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<th>1972</th>
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<td>Surface</td>
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<tr>
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<td>Diversion</td>
<td>Effluent</td>
<td>Outflow</td>
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<td>Salt</td>
<td>(1000 API)</td>
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<td>Baseline</td>
<td>597</td>
<td>161</td>
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</table>
Figure 5. Predicted water and salt outflows resulting from irrigation management alternatives applied to the Grand Valley subbasin for the water years 1970-1972.
Duchesne study area. The Duchesne area is typical of some Upper Basin irrigation projects where sequential recycling of the diverted water makes up a significant part of the total diverted water. There are many diversion works for which only partial flow records are kept as well as some small storage facilities that cause regulatory effects downstream for which sufficient data are not available for incorporating in the model. The base line condition, as calibrated, indicates that a salt imbalance may exist in the area; however, this may be the result of poor simulation of the quality of the seepage return flows that make up a significant portion of the canal diversions. The research referred to in Grand Valley may help answer this question, but without additional research and testing of the model hypotheses of salt pickup, a definitive assessment cannot be made. If the model assumptions are valid, then the trends indicated by the management runs for the Duchesne basin would be valid even though the absolute numbers depicted may not be. The management results are shown in Figure 6 and show a reduction in salt loading for all three alternatives tested. These results are consistent with those of the one-dimensional model.
Figure 6. Predicted water and salt outflows resulting from irrigation management alternatives applied to the Duchesne subbasin for water year 1972.
SALINITY CONTROL THROUGH ON-FARM WATER MANAGEMENT IN GRAND VALLEY

by

Gaylord V. Skogerboe and Wynn R. Walker*

Introduction

In April of 1972, the seven basin states sharing the water resources of the Colorado River Basin (Fig. 1) and the U.S. Environmental Protection Agency (EPA) responsible for the quality of such flows agreed to the necessity of maintaining the concentrations of salts in the Lower Basin at or below existing levels (U.S. EPA, 1972). Further, the necessity to allow Upper Basin users to proceed with the development of waters apportioned to them under the Colorado River Compact of 1922 was realized. These two segments of the statements emanating from the enforcement conference are, however, contradictory without accompanying each new development with sufficient reductions in existing salt loads to compensate for the effects of the new water use.

The collective decisions regarding the control of salinity in the basin have been induced by the mounting damages incurred by downstream users. Salinity problems are also of international concern owing to the detriments being experienced in the Mexicali Valley of the Republic of Mexico. The methods available for controlling salinity include phreatophyte eradication, reducing evaporation, desalination, elimination of mineralized point sources, importing supplemental water, and improving agricultural, municipal, and industrial water use practices. While certain of these alternatives may be either technologically impractical or politically unacceptable, they represent the array of

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Fig. 1. The Colorado River Basin.
alternatives from which an overall strategy must be generated. Because of the limited effectiveness of each measure, such a strategy for salinity control must be a combination of several feasible alternatives. The first task is, therefore, to develop the costs and the effectiveness of the individual salinity control measures.

Input-Output Analysis

The increasing salinity problem in the Colorado River Basin has necessitated the collection and analysis of data on water and salt flows in order to evaluate the contributions from various sources. Although several interested governmental agencies have conducted short term studies in the basin, the primary source of data is the stream monitoring system of the U. S. Geological Survey. One of the most comprehensive efforts to summarize and analyze these data was made by Iorns, Hembree, and Oakland (1965) for the period between 1914 and 1957 and adjusted to the 1957 conditions. The study was inclusive of the entire Upper Colorado River Basin, but for the purposes of this paper only the section dealing with the Grand Valley area has been extracted. The total salt loading to the Colorado River from the Grand Valley averaged about 750,000 tons during the period.

The 1963-1967 water years were selected by the Colorado River Board of California (1970) in conjunction with various governmental agencies to appraise the salinity sources in the basin and to evaluate the future impact of water resource developments on mineral water quality. The results pertaining to the Grand Valley in particular indicated the salt pickup to be about 8 tons per acre per year, which is the results Hyatt (1970) established for the 1963-1968 years. Both of these references are useful data sources for examination of the Upper Colorado River System. Also, both studies utilized salinity data collected by the Federal Water Pollution Control Administration (now the U.S. Environmental Protection Agency).
One examination of the sources of salinity in the basin, shown in Fig. 2, reveals that of man-made contributions, irrigated agriculture has the largest effect. Consequently, the major aspect of salinity control in the region must be the effective use of irrigation diversions by improving the efficiency of conveyance, farm and wastewater systems. One of the several important efforts funded by the U.S. Environmental Protection Agency to develop salinity control technology is the Grand Valley Salinity Control Demonstration Project in western Colorado (Fig. 3). The reason for selecting Grand Valley for intensive study is because the annual salt pickup per acre is greater in this particular irrigated area than any other irrigation system in the Upper Basin.

**Study Area**

The Colorado River enters the Grand Valley from the East, is joined by the Gunnison River at Grand Junction, Colorado, and then exits to the West. The contribution to the total salt flows in the basin from this area, illustrated in Fig. 4, is highly significant. The primary source of salinity is from the extremely saline aquifers overlaying the marine deposited Mancos shale formation. The shale is characterized by lenses of salt in the formation which are dissolved by water from excessive irrigation and conveyance seepage losses when it comes in contact with the Mancos shale formation. The introduction of water through these surface sources percolates into the shallow ground water reservoir where the hydraulic gradients it produced displace some water into the river. This displaced water has usually had sufficient time to reach chemical equilibrium with the salt concentrations of the soils and shale. These factors also make the Grand Valley an important study area, since the conditions encountered in the valley are common to many locations in the basin.
Fig. 2. Major sources of salinity in the Colorado River Basin (U.S. EPA, 1971).
Fig. 3. The Grand Valley of Colorado.
Fig. 4. Relative magnitude of agricultural salt sources in the Colorado River Basin (U.S. EPA, 1971).
The study area, shown in Fig. 5, was chosen as an intensive study area in which the bulk of the investigation was to be conducted and also includes most of the construction and demonstration efforts. This area was designated for detailed investigations regarding various salinity control measures on the water and salt flow systems in an irrigated area. The intensive study area was selected for its accessibility in isolating most of the important hydrologic parameters, but had the important advantage that it allowed five irrigation companies to participate in one unit.

**Hydro-Salinity Model**

In undertaking the Grand Valley Salinity Control Demonstration Project, one of the first tasks was to conceptualize a hydro-salinity model of the intensive study area. This model had to have sufficient sensitivity to detect the effects of various salinity control measures upon the salt pickup reaching the Colorado River. Then, the model could be used to design the field data collection system. Finally, the model could be used to extrapolate results from the intensive study area to the entire Grand Valley.

A difficulty often encountered while preparing water and salt budgets is the variability in the accuracy and reliability with which the hydrologic and salinity parameters are measured. Usually, the measurement precision varies with the scope of the research and the area of the study. The intensive study area on this project has been observed in great detail.

Since the hydrologic system is difficult to monitor and predict, it is impractical to expect their models to operate without applying some adjustments in order that all components will be in balance. In short, the budgeting procedure is usually the adjustment of the segments in the water and salt flows according to a weighting of the most reliable data until all parameters represent the closest approximation of
Fig. 5. Intensive study area, Area I, of the Grand Valley Project.
the area that can be achieved with the input data being used. The vast and lengthy computation procedure of calculating budgets is facilitated by a mathematical model programmed for a digital computer. A complete listing and explanation of its operation has been previously reported (Walker, 1970). For the purposes of this paper, the more important aspects will be extracted for discussion. A schematic diagram of a general hydro-salinity model is shown in Fig. 6.

The model of the intensive study area was developed in three general sections:

1. All diversions from the canals through small turnouts into the lateral network are distributed onto the farmland after taking into account lateral seepage losses;
2. Flow within the root zone including evapotranspiration, tailwater runoff, and deep percolation losses; and
3. Groundwater return flows resulting from seepage and deep percolation return to the river system with their large salt loads through both surface and subsurface drainage routes.

**Cropland diversions**

The irrigation supply is diverted from the Colorado River by means of large check-type dams and then conveyed through the Grand Valley with water being lost by seepage, spilled into wasteways, evaporated, and discharged through turnout structures into laterals. Two of these alternate routes, spillage, and the lateral diversions, will be examined further.

Natural washes and drains located throughout the valley serve as wasteways for canal regulation operation. The Grand Valley Canal dumps water into Lewis Wash to supply the Mesa County Ditch. These flows are mixed with a considerable drainage flow and a noticeable water quality degradation occurs.

Diversions into the lateral system in the test area are also reduced by seepage. Evaporation is insignificant. Most of these
Fig. 6. Schematic generalized hydro-salinity model.
small conveyance channels carry less than 5 cfs but may serve as many as 100 farmers. Both maintenance and management of laterals below the canal turnout is poor. Of the flows reaching the cropland, only about 60 to 70 percent of the water actually enters the root zone and the remaining flow is field tailwater and returns directly to the river via the open drainage system. Comparison of drainage discharges throughout three irrigation seasons indicated that about 80 percent of the surface drain flows are field tailwater.

**Root zone flows**

The goal of an irrigation is to recharge the soil moisture reservoir with sufficient water to meet the growing crops needs until the next irrigation, as well as to maintain an acceptable salt concentration in the root zone. The tendency to over-irrigate has produced high water tables and salinity problems. The purpose of the root zone submodel was to separate the various flows occurring within the root zone in sufficient detail to quantify the salinity problem.

The important water movements within the root zone are evapotranspiration and deep percolation, with water storage changes also occurring. The separation of these flows by measurement is impractical on a large scale. Consequently, empirical computational methods were employed. The model developed for this study accounts for these basic water and salt flows only by a budgeting process. The assumptions made regarding the operation of this model include that the diversions are applied uniformly over each acre of cropland. Phreatophyte vegetation in the area was assumed to extract water only from the groundwater flows or to use only precipitation entering the root zone of these plants. A generalized flow chart of the root zone budgeting procedure is presented in Fig. 7.

Several applicable methods of estimating evapotranspiration could have been used in this study. However, because the shortest time period employed in the study was one month, the Blaney-Criddle
Fig. 7. Illustrative flow chart of the root zone budgeting procedure.
Method provided an acceptable degree of accuracy. This method determines consumptive use as a function of mean monthly temperature and the percentage of daylight hours occurring during the month. As the study progressed, a comparison was made between the Blaney-Criddle Method and some of the more sophisticated energy balance relationships. Comparisons indicated that the Blaney-Criddle Method was somewhat conservative. Studies are presently underway to improve the estimates of evapotranspiration.

With the evapotranspiration data and field measurements of moisture holding capacity, texture, infiltration rates, and rooting depths, the budgeting scheme proceeded with computation of deep percolation losses from the root zone. The calculations were initiated by assuming that the crops use soil moisture at the potential rate until the wilting point is reached. The calculated potential use is then limited to the water added by irrigation and the existing available soil moisture storage. If the supply to the root zone from irrigation is insufficient to meet the crop demands but the available soil moisture storage is sufficient to make up the difference, then the crop demand is satisfied. It was assumed that while the soil moisture reservoir is below field capacity, no deep percolation occurs. If the total available moisture in a period is insufficient to meet the total demand, the crops use all water available. A term called "consumptive use deficit" is defined as the difference between the potential and actual uses. Deep percolation losses and leaching occur when the supply is more than enough to meet the crop demands and fill the soil moisture reservoir to field capacity.

The salts in the applied water move with the water into the root zone where they are concentrated by the evapotranspiration process. The behavior of specific ions is complex and has not been considered in this particular study. However, additional research is underway in Grand Valley to provide prediction equations for specific ions. The assumption has been made that the salts acquired from the intensive
study area occur as salt pickup below the root zone. This assumption allows for a simplified computational procedure in evaluating the demonstration area based upon an input-output model.

**Groundwater model**

Most of the water in the soils and aquifers in the test area originate as seepage from canals and laterals, as well as deep percolation from the irrigation of croplands. The groundwater discharges eventually reach the river as surface drainage interception or subsurface return flows. The flows in the surface drainage system were measured by installing flow measuring devices at the outflow points. The subsurface return flows were not measured but were estimated from water table elevation data and the hydraulic gradients in the aquifers. Considerable effort was made to evaluate the necessary parameters to use in the groundwater computations. For purposes of this study, Darcy's steady state equation was used (Luthin, 1966).

\[
Q = AK \frac{dh}{dk}
\]  

(1)

in which \( Q \) is the discharge, \( A \) is the cross-sectional area of flow, \( K \) is the hydraulic conductivity, and \( \frac{dh}{dk} \) is the hydraulic gradient in the direction of flow.

The groundwater analysis, illustrated in Fig. 8, begins by comparing the values for subsurface return flow obtained from a mass balance of the area to the values obtained by calculation using the field data. It was possible to formulate two estimates of the subsurface return flows and then by adjusting the model until both methods yielded the same values, a satisfactory alignment between the hydrologic and salinity parameters was obtained. Because the model only focuses attention on the relative magnitude of hydraulic conductivities, the cross-sectional areas of the strata need only be in proper proportion with respect to depth, and the width can be any convenient value. Then, the values for cross-sectional area can be adjusted with the known
Fig. 8. Illustrative flow chart of the groundwater modeling procedure.
hydraulic conductivities. The model adjusts the values of strata hydraulic conductivity until both estimates of the flows are equal. Since this is done on a monthly basis, the model calculates twelve values of hydraulic conductivity for each strata for each year. When adjustments in the model finally result in homogeneous values of hydraulic conductivity, the model represents the "best fit" between monitored and estimated data.

The groundwater modeling procedure can also be described mathematically. The form of Eq. 1 for a number of strata can be written,

\[ Q = \sum_{i=1}^{n} A_i K_i \frac{dh_i}{dx_i} \]  

(2)

where \( A_i \) is the cross-sectional area of the \( i \)th strata, \( K_i \) is the actual measured conductivity of the \( i \)th strata, and \( \frac{dh_i}{dx_i} \) is the gradient in the flow direction acting on the \( i \)th strata. The value obtained from Eq. 2 is then used to adjust the model values of hydraulic conductivity,

\[ K_i = \frac{TGWOF}{Q} K_i \]  

(3)

where \( K_i \) is an adjusted hydraulic conductivity encompassing adjustments for units and strata areas, \( Q \) is the value obtained from Eq. 2, and TGWOF is the subsurface return flow estimate from the mass balance analysis.

**Generalizing the model**

The mathematical model derived for this study attempted to simulate the hydrologic conditions of the agricultural system in Grand Valley, but the concepts are general and can be extended with modification to other areas that are similar in nature. The program was written in individual but interconnected subroutines that give the program a measure of flexibility during operations by separating the calculation phase from either input or output phases. Thus, several of the subroutines become optional if their functions can be replaced by input data, or if certain outputs are not desired.
The main portion of the program is used to read necessary input data and to control the order of water and salt budget calculations. There are certain advantages in separating the input, output, and computational stages of a program including:

1. Input order is not important as the data are completely available at all stages of computation.
2. Variable sets of data can be utilized in the model when several budgets are desired, or when some form of integration is desired. This is especially useful when an area can be broken down into smaller dependent areas.
3. The functions of the subroutines are independent of input, thereby making each subroutine a unit that can be implemented in other programs.
4. Corrections and adjustments are easily made without detailed consideration to other segments of the program.

In controlling the computational order of the program, the main program separates the calculation of the water and salt budgets. Consequently, the modeling procedure involves only the water phase of the flow system. This has been possible in this study because of the detail in which data have been collected. Once the water flow system has been simulated, the individual flows are multiplied by measured salinity concentrations and converted to units of tons per month. At this point in the formation of the budgets, careful attention must be given to the salt flow system since irregularities may be present, thereby necessitating further model adjustments. Thus, when the final budgets have been generated, the salt system, groundwater system, and surface flow system must be reasonably coordinated and additional reliability is assured.

Summary of model results

In this section, a summary of the input-output analysis and results from the hydro-salinity model will be presented. Inflows to
the Grand Valley occur as flows in the Colorado River, Gunnison River, and precipitation. In addition, a small quantity of water is imported for domestic and industrial purposes, and a possibility exists that precipitation on the watershed adjacent to the valley may contribute via diffuse groundwater inflows. Neither of these latter flows are deemed significant, especially the inflow from surrounding lands because of the low annual precipitation (8-10 inches) and high evaporative demands (40-45 inches).

As a means of better identification, data for the 1968 water year from the U.S. Geological Survey (USGS) and U.S. Weather Bureau can be utilized. Inflows passing the USGS gaging stations "Colorado River near Cameo" (2,413,000 acre-feet), "Plateau Creek near Cameo" (112,000 acre-feet), and "Gunnison River near Grand Junction" (1,444,000 acre-feet) totaled 3,968,000 acre-feet carrying an estimated salt load of 3,070,500 tons. The outflows passing the station "Colorado River at Colo-Utah State Line" totaled 3,722,000 acre-feet and approximately 3,771,000 tons of salt. These figures represent either published data or interpolations thereof. It should be noted that the state line station collects only limited quality data.

A comparison of the inflows and outflows indicates that 246,000 acre-feet of water were depleted from the system and 701,000 tons of salt added. Precipitation records indicate that approximately 75,000 acre-feet fell on the land encompassed by the irrigated boundaries of which it is estimated that 25,000 acre-feet could be classed as "effective on the irrigated acreages." These estimates are congruent with similar computations presented by Iorns et al. (1965), Hyatt (1970 and U.S. Environmental Protection Agency (1971).

Another check on these numbers can be made from land use data collected by Walker and Skogerboe (1971). A somewhat more definitive breakdown is presented in Table 1. Westesen (1974) estimated that the consumptive use based on the pan evaporation data from the U.S. Weather Bureau and calculations using the Modified
Jensen-Haise method amounted to about 295,000 acre-feet annually, including almost 25,000 acre-feet of effective precipitation on other vegetative uses. Thus, the inflow-outflow data for this particular year regarding water flow is acceptable. An examination of the salt flows will be noted for comparison in the following paragraphs.

Table 1. Agricultural land use in the Grand Valley.

<table>
<thead>
<tr>
<th>Land Use</th>
<th>Acreages</th>
<th>Percent of Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>Irrigated</td>
<td>60,844</td>
<td>53</td>
</tr>
<tr>
<td>Idle</td>
<td>9,706</td>
<td>8.5</td>
</tr>
<tr>
<td>Dwelling &amp; Premises</td>
<td>10,678</td>
<td>9.3</td>
</tr>
<tr>
<td>Open Water</td>
<td>1,699</td>
<td>1.5</td>
</tr>
<tr>
<td>Phreatophyte</td>
<td>15,174</td>
<td>13.2</td>
</tr>
<tr>
<td>Natural Terrain</td>
<td>16,607</td>
<td>14.5</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>114,708</strong></td>
<td><strong>100.0</strong></td>
</tr>
</tbody>
</table>

1Roads and railways have been omitted.

The second approach to establishing the effects of water use in the Grand Valley is to model the complex inter-relationships associated with irrigation and drainage. Several parameters are added to the analysis to account for the various flows which take place.

The first segment encountered is the delineation of the canal diversions. As the water is diverted from the rivers into the canals and ditches, a certain portion of the flow seeps or evaporates from the conveyance surfaces, while still another fraction is spilled into wasteways as a means of regulating capacity. The remainder of the flow is diverted through small headgates into an extensive lateral system leading to the fields. It is important in this type of analysis that each flow path be defined, because each results in a different salinity effect. For example, the evaporative losses concentrate the salts in the remaining flows, whereas the seepage enters the saline groundwater basin and results in salt pickup.
Lateral diversions eventually become seepage, field tailwater, root zone additions or evaporation, as was the case above. In a similar manner, the root zone additions result in cropland consumptive use or deep percolation. When deep percolation is combined with seepage losses, a groundwater flow segment is begun which results in the severe salt loadings common in the valley. A great deal of the groundwater is consumed by water-loving phreatophytes abundant in the area and some of the flows are intercepted by the open-ditch drainage system. A substantial amount returns to the rivers through aquifers making precise measurement difficult.

Westesen (1974) examined the 1968 water year in some detail and combined many of the principles discussed by Walker (1970) into an accounting of the flows derived for irrigation in the Grand Valley. His results, shown in Tables 2, 3, and 4, compare very well with data collected by the authors in recent years.

Much of the local water table problems are due to over-irrigation, especially along the higher northern lands in the area. Lower areas and isolated trouble spots are affected by excessive groundwater flows trying to leave the area. If the local canals and laterals were lined (including farm head ditches), Table 4 indicates that 77,000 acre-feet annually, which amounts to 55 percent of the groundwater inputs, would be prevented from contributing to local drainage problems. (If only the canals were lined, the groundwater would be decreased by only 18 percent.) Such improvements may also reduce the evapotranspiration from phreatophytes and result in significant water savings as well. Canal linings appear to be the initial program for controlling salinity in the Grand Valley although it is the least effective alternative available.

Probably the greatest potential for salinity control lies in on-farm water management which directly includes the lateral conveyance system. Together, deep percolation and lateral seepage contribute 82 percent of the groundwater flows. If effective irrigation

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Table 2. Grand Valley water budget for 1968 water year.

<table>
<thead>
<tr>
<th>Budget Item</th>
<th>Acre-feet</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Surface Inflows</strong></td>
<td></td>
</tr>
<tr>
<td>Colorado River near Cameo, Colorado</td>
<td>2,413,000</td>
</tr>
<tr>
<td>Plateau Creek near Cameo, Colorado</td>
<td>112,000</td>
</tr>
<tr>
<td>Gunnison River near Grand Junction, Colorado</td>
<td>1,443,000</td>
</tr>
<tr>
<td><strong>TOTAL</strong></td>
<td>3,968,000</td>
</tr>
<tr>
<td><strong>Effective Precipitation</strong></td>
<td></td>
</tr>
<tr>
<td>Cropland</td>
<td>25,000</td>
</tr>
<tr>
<td>Phreatophytes</td>
<td>5,400</td>
</tr>
<tr>
<td><strong>TOTAL</strong></td>
<td>30,400</td>
</tr>
<tr>
<td><strong>System Depletions</strong></td>
<td></td>
</tr>
<tr>
<td>Water Surface Evaporation</td>
<td></td>
</tr>
<tr>
<td>Canals</td>
<td>8,000</td>
</tr>
<tr>
<td>Rivers</td>
<td>8,000</td>
</tr>
<tr>
<td>Phreatophyte Consumption</td>
<td></td>
</tr>
<tr>
<td>Along Canals and Drains</td>
<td>64,000</td>
</tr>
<tr>
<td>Adjacent to Rivers</td>
<td>21,400</td>
</tr>
<tr>
<td>Cropland Consumption</td>
<td>175,000</td>
</tr>
<tr>
<td><strong>TOTAL</strong></td>
<td>276,400</td>
</tr>
<tr>
<td><strong>Surface Outflows</strong></td>
<td></td>
</tr>
<tr>
<td>Colorado River at Colorado-Utah State Line</td>
<td>3,722,000</td>
</tr>
</tbody>
</table>
Table 3. Grand Valley distribution of canal flows in 1968.

<table>
<thead>
<tr>
<th>Budget Item</th>
<th>Acre-feet</th>
<th>Acre-feet</th>
</tr>
</thead>
<tbody>
<tr>
<td>Canal Diversions</td>
<td>560,000</td>
<td>103,000</td>
</tr>
<tr>
<td>Spillage</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Seepage</td>
<td>25,000</td>
<td></td>
</tr>
<tr>
<td>Evaporation</td>
<td>8,000</td>
<td></td>
</tr>
<tr>
<td>Lateral Diversions</td>
<td></td>
<td>424,000</td>
</tr>
<tr>
<td>TOTAL</td>
<td></td>
<td>560,000</td>
</tr>
<tr>
<td>Lateral Diversions</td>
<td>424,000</td>
<td></td>
</tr>
<tr>
<td>Seepage</td>
<td>51,000</td>
<td></td>
</tr>
<tr>
<td>Field Tailwater</td>
<td>162,000</td>
<td></td>
</tr>
<tr>
<td>Root Zone Diversions</td>
<td>211,000</td>
<td></td>
</tr>
<tr>
<td>TOTAL</td>
<td>424,000</td>
<td></td>
</tr>
<tr>
<td>Root Zone Diversions</td>
<td>211,000</td>
<td></td>
</tr>
<tr>
<td>Evapotranspiration</td>
<td>150,000</td>
<td></td>
</tr>
<tr>
<td>Deep Percolation</td>
<td>61,000</td>
<td></td>
</tr>
<tr>
<td>TOTAL</td>
<td>211,000</td>
<td></td>
</tr>
<tr>
<td>Groundwater Return Flows</td>
<td>137,000</td>
<td></td>
</tr>
<tr>
<td>Phreatophyte Consumption</td>
<td>60,000</td>
<td></td>
</tr>
<tr>
<td>Subsurface and Drain Flows</td>
<td>77,000</td>
<td></td>
</tr>
<tr>
<td>TOTAL</td>
<td>137,000</td>
<td></td>
</tr>
</tbody>
</table>

Table 4. Salt budget for Grand Valley during 1968.

<table>
<thead>
<tr>
<th>Budget Item</th>
<th>Flow (acre-feet)</th>
<th>Concentration (ppm)</th>
<th>Salt Load (tons)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Inflows</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Colorado River near Cameo</td>
<td>2,413,000</td>
<td>454</td>
<td>1,490,000</td>
</tr>
<tr>
<td>Plateau Creek near Cameo</td>
<td>112,000</td>
<td>454</td>
<td>69,000</td>
</tr>
<tr>
<td>Gunnison River near Grand Jct.</td>
<td>1,443,000</td>
<td>769</td>
<td>1,511,000</td>
</tr>
<tr>
<td>TOTAL</td>
<td></td>
<td></td>
<td>3,070,000</td>
</tr>
<tr>
<td>Cutflows</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Colorado River near Colo-Utah</td>
<td>3,722,000</td>
<td>745</td>
<td>3,771,000</td>
</tr>
<tr>
<td>State Line</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Salt Pickup</td>
<td></td>
<td>701,000</td>
<td></td>
</tr>
</tbody>
</table>

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scheduling programs are incorporated locally, which means accompanying the scheduling services with rehabilitation of the irrigation systems, the need for field drainage will be diminished.

Thus, the first steps in a salinity control program are to minimize: (a) deep percolation losses from croplands (ideally, the deep percolation losses would not exceed the leaching requirement); (b) seepage losses from canals and laterals. By minimizing the amount of moisture reaching the groundwater, the requirements for field drainage will also be minimized. As higher levels of salinity reduction are sought, field drainage becomes a more feasible component of a valley-wide salinity control program.

The results to date clearly show that the key to reducing the salt load contribution to the Colorado River from Grand Valley is improving on-farm water management practices in order to minimize deep percolation losses and consequent salt pickup.

Predicting Chemical Quality

The hydro-salinity model describes the present situation in the study area regarding water and salt flows. However, the only method for predicting the reduction in salts returning to the river through implementation of any salinity control measure(s) is by assuming a one-to-one relationship between water and salt. That is, if the subsurface return flow is reduced by 50 percent, the salt is also reduced by 50 percent. In order to overcome this limitation, a project "Irrigation Practices, Return Flow Salinity, and Crop Yields" was initiated.

Three adjacent fields containing 23 acres was leased for this study. The area has been divided into 54 plots which are 100 feet by 100 feet in size, two plots which are 40 feet by 200 feet, two plots which are 40 feet by 300 feet, and five plots which are 40 feet by 500 feet. Each plot is used for a different replication of the crop, fertilizer, and irrigation treatments. They have been constructed so that
each plot performs as a large lysimeter. A trench was excavated slightly into the shale along the lines dividing the plots. A plastic curtain was then placed vertically in the center of the trench to divide the individual plots. The lower edge of the curtain is "sealed" to the shale by back-filling to the original elevation of the shale with compacted clay (Fig. 9).

The drainline encased in a gravel filter material was then placed inside the curtain and continued around the periphery of the plot. Upon leaving the plot area, the water is transported via solid pipeline to a measuring station where water quality and quantity is monitored.

The irrigation system is designed to deliver water through a closed conduit to each plot and allow measurement of the flows onto each plot. Since furrow irrigation is used almost exclusively throughout the valley, this method has been employed on the project area.

The crops being grown are corn, grass, alfalfa, and winter wheat, since these are the main crops grown commercially in the valley. By varying irrigation timing and amounts, crops, and nitrogen fertilizer levels on the different plots, and by monitoring quality and quantity of both inflow and outflow waters, the effects of these parameters on return flow salinity and crop yields can be evaluated.

One of the primary objectives of this research is to model the transport of salts in the soils in this area. The first portion of the flow of water and consequent transport of salts is through the root zone which is usually a zone of partial saturation. A numerical model of the moisture flow and chemical and biological reactions occurring in the root zone has been developed by Dutt et al. (1972). This is the basic model which will be used in this study to describe the salt transport being observed in the field.

The model consists of three separate programs. The first program describes the soil moisture movement and distribution with time. The second program interfaces the soil moisture movements with the chemical biological model. This is needed because the horizons used
Fig. 9. Plot cross-section with drain details.
in the calculations of soil moisture and chemistry differ. The third program computes the chemical and biological activity occurring in the soil profile. Fig. 10 is a block diagram of the overall model. A brief description of the moisture flow and chemical-biological models is included to serve as a basis for understanding the data collection requirements.

The flow is one dimensional and was developed using the Richards equation with a sink term. Schematically, the model is given in Fig. 11. Mathematically, the flow is described using Richards equation in the form:

\[
\frac{\partial \theta}{\partial t} = \frac{\partial}{\partial x} \left( \frac{D \partial \theta}{\partial x} - K \right) - S \quad \ldots \quad (4)
\]

where

- \( \theta \) = volumetric water content
- \( t \) = time
- \( x \) = length
- \( K \) = hydraulic conductivity
- \( S \) = sink term
- \( D \) = diffusivity

This is the diffusivity form of the equation which means that only flow in the partially saturated zone of the soil profile can be described. The sink term \( S \) is computed using the Blaney-Criddle equations for evapotranspiration with the loss due to evapotranspiration being distributed through the soil profile by assuming a specific root distribution for the crop. The root distribution and coefficients for the Blaney-Criddle equations are supplied by the user. Actual values of evapotranspiration can be used in the sink term when they are known. In this research, the necessary field data is being collected on-site that will utilize either Penman or Jensen-Haise evapotranspiration equations.

Salt transport is described by the following equation in one dimension.
Fig. 10. Generalized block diagram of the model.
START MOISTURE FLOW PROGRAM

PROGRAM MOISTRE

READ CONTROL AND INPUT DATA

COMPUTE MOISTURE CONTENT AND FLUX FOR EACH DEPTH NODE AND TIME STEP

WRITE ON MAGNETIC TAPE OR PRINT OUTPUT

SUBROUTINE THEDATE

COMPUTE CALENDAR DATE FROM DAY NUMBER

SUBROUTINE CONUSE

COMPUTE VALUE OF MACROSCOPIC SINK TERM

STOP MOISTURE FLOW PROGRAM

Fig. 11. Generalized block diagram of Moisture Flow Program.
\[
\frac{\partial c}{\partial t} = \frac{\partial}{\partial z} \left( D \frac{\partial c}{\partial z} \right) - \nu \frac{\partial c}{\partial z} \quad \ldots \quad (5)
\]

where

- \( c \) = solute concentration
- \( t \) = time
- \( D \) = dispersion coefficient
- \( z \) = depth
- \( \nu \) = flux or darcy velocity

By assuming the term \( \frac{\partial}{\partial z} \left( D \frac{\partial c}{\partial z} \right) \) is negligible compared to \( \nu \frac{\partial c}{\partial z} \), the equation reduced to \( \frac{\partial c}{\partial t} = - \nu \frac{\partial c}{\partial z} \). This assumption implies that transport due to dispersion in partially saturated soils is negligible compared to the convective transport which occurs. This is generally a good assumption.

The model computes the moisture flow (\( \nu \)) and couples the flow with the chemical changes \( \frac{\partial c}{\partial z} \) computed in the biological-chemical program to give the salt transport. This technique is the basis for the mixing cell concept.

The chemical exchange model computes the equilibrium chemistry concentrations for calcium, magnesium, gypsum, sodium, bicarbonates, carbonates, chlorides, and sulfates. The nitrogen chemistry including ammonium, nitrates, and urea-nitrogen uses a kinetic instead of an equilibrium approach. The kinetic approach is needed since microbial activity involved in nitrogen transformation occurs over a period of weeks and days instead of minutes and seconds. The equilibrium chemistry for inorganic salts is a good approximation since the reactions describing their chemistry occur in a matter of minutes or seconds in a flow regime which is changing very slowly.

A block diagram of the biological chemical model is given in Fig. 12.

Preliminary studies have been made with this model to evaluate its capabilities and to insure compatibility with the available computer facilities. Analysis of last year's field data allowed extensive testing of the model. Modifications have been made to more accurately model existing field conditions.
Fig. 12. Generalized block diagram of Biological-Chemical Program.
Since the shale floor and plastic membrane walls act to create a box around each plot, the plot acts as a large lysimeter. A salt and water budget will be developed for each plot and compared to those developed for the other plots. From these data, equations can be developed to predict the variation in chemical quality (including ionic constituents) of the moisture movement through the soil profile, as well as the salt pickup resulting from movement of subsurface irrigation return flows over the Mancos shale beds. These results combined with the hydro-salinity model will allow an evaluation of various salinity control measures upon salinity reaching the Colorado River.

Salinity Control Measures

Channel lining

The results of channel lining studies indicate that canal and lateral lining in the study area reduced salt inflows to the Colorado River by about 4700 tons annually (Skogerboe and Walker, 1972). The bulk of this reduction is attributable to the canal linings, but clearly indicated is the greater importance of lateral linings. The length of laterals, including farm head ditches, is about ten times greater than the length of canals. The economic benefits to the Lower Basin water users alone exceed the costs ($350,000 construction plus $70,000 administration) of this project. Consequently, it seems justifiable to conclude that conveyance lining in areas such as the Grand Valley, where salt loadings reach 8 tons or more per acre, are a feasible salinity control measure. The local benefits accrued from reduced maintenance, improved land value, and other factors add to the feasible nature of conveyance linings as a salinity management alternative.

The first and most important consideration in improving farm water use is control. Implied in this realization is the requirement
of sound water measurement at the farm turnout and again at critical division points among farmers below the turnout. This would necessitate a considerable rehabilitation of both the canal and lateral system, and the implementation of a "call period" to allow canal operators more time for flexible water handling. In addition, it is an important requirement that the canal companies extend their control of the water below the canal turnout structure to include key division points within the lateral system to insure equitable allocation of water among users.

Irrigation scheduling

The irrigation of agricultural lands in the Colorado River Basin is a significant cause of the salinity concentrations encountered in the Colorado River. Emphasis towards stemming further salinity increases has logically centered upon improving the quality of irrigation return flows. This emphasis, especially in the high salt contributing areas like the Grand Valley in western Colorado, focuses upon reducing the flows which pass through the saline soils and aquifers, thereby reducing the salt pickup that occurs by dissolution. Since a major fraction of the water contacting local soils in this manner comes from over-irrigation, measures aimed at improving irrigation efficiencies promise good potential for controlling salinity. Among the methods for achieving higher water use efficiencies on the farm, "scientific" irrigation scheduling is possibly the most important (Skogerboe, Walker, Taylor, and Bennett; 1974).

Irrigation scheduling consists of two primary components; namely, evapotranspiration and available root zone soil moisture. Evapotranspiration is calculated by using climatic data. The other major category of required data pertains to soil characteristics. First of all, field capacity and wilting point for the particular soils in any field must be determined. More importantly, infiltration characteristics of the soils must be measured. Only by knowing how soil
intake rates change with time during a single irrigation, as well as throughout the irrigation season, can meaningful predictions be made as to: (a) the quantity of water that should be delivered at the farm inlet for each irrigation; and (b) the effect of modifying deep percolation losses. With good climatic data and meaningful soils data, accurate predictions as to the next irrigation date and the quantity of irrigation water to be applied can be made. In order to insure that the proper quantity of water is applied, a flow measurement structure is absolutely required at the farm inlet.

The results of this demonstration project indicate that irrigation scheduling programs have a limited effectiveness for controlling salinity in the Grand Valley under existing conditions. Excessive water supplies, the necessity for rehabilitating the irrigation system (particularly the laterals), and local resistance to change preclude managing the amount of water applied during successive irrigations. To overcome these limitations, irrigation scheduling must be accompanied by flow measurement at all the major division points, farm inlets, and field tailwater exits. In addition, it is necessary for canal companies and irrigation districts to assume an expanded role in delivery of the water. Also, some problems have been encountered involving poor communication between farmer and scheduler, as well as certain deficiencies in the scheduling program dealing with evapotranspiration and soil moisture predictions. These latter problems can be easily rectified, however. Correcting these conditions will make irrigation scheduling much more effective and acceptable locally.

Water budgets from which the study results were generated resulted from intensive investigation on two local farms. The selection of the two study farms was intended to be representative of conditions valley-wide. Analysis of the budgets reveal that approximately 50 percent of the water applied to the fields came during the April and May period when less than 20 percent of the field evapotranspiration potential has been experienced. Salt pickup estimates during
this early part of the season amounted to about 60 percent of the annual total for each field. Another indication of the importance of early season water management is presented in an analysis of irrigation efficiencies. As the season progressed, the soils became less permeable and the crop water use increased, causing marked improvements in irrigation efficiency. Thus, if irrigation scheduling is employed in its optimal format, salt pickup from the two fields could have been reduced as much as 50 percent or more.

The results of this demonstration project show that irrigation scheduling is a necessary, but not sufficient, tool for achieving improved irrigation efficiencies. The real strides in reducing the salt pickup resulting from over-irrigation will come from the employment of scientific irrigation scheduling in conjunction with improved on-farm irrigation practices. This combined effect could result in reduction of 300,000 tons annually of salt pickup from the Grand Valley, depending upon the degree of improvement in present on-farm irrigation practices.

Drainage

Drainage investigation in the Grand Valley began shortly after the turn of this century when local orchards began failing due to high saline water tables. Study showed the soils to be not only saline but also having low permeabilities. At the time, the future development of the Bureau of Reclamations "Grand Valley Project" loomed as a severe threat to the low lying lands between it and the Colorado River. In answer to these drainage needs, the solutions were clearly set forth but never fully implemented. Rather, a local drainage district was formed to construct open ditch drains and some buried tile to correct trouble spots. All of these efforts barely stagnated the rise in water tables, and today more than fifty years later, the local conditions remain essentially unchanged.
This study was undertaken with the history of local drainage well in mind, but for a different purpose—that being the skimming of water from the top of the water table before it reaches equilibrium with the highly saline soils and aquifers. A farm owned by Mr. Wareham was used in the study to demonstrate the skimming effect by installing field relief drains on forty foot centers. The field had been under poor irrigation management for several years, so the results are not immediately discernable. However, analysis of water quality throughout the study area indicated that relief drainage if effective would interrupt flows with a salinity concentration as much as 3000 ppm lower than existing groundwater concentrations.

In viewing the results of this study, it is obvious that field drainage is a curative rather than preventative measure. High costs of such programs illustrate the need of first minimizing the flows passing through the root zone or seeping from canals and laterals. The small amount of water then entering the groundwater could then be effectively removed by drainage systems located at selected locations. Thus, field drainage as it pertains to objectives of salinity control is a remedy which must be considered but will probably not be orderly implemented until the later stages of salinity control in the valley (Skogerboe, Walker, Bennett, Ayars, and Taylor, 1974).

As part of the study, an alternative use of drainage was considered. This involved the collection of and desalting of drainage effluents. During the 1940's, pump drainage from a deep cobble aquifer was tested and proved most effective. In determining the costs of pump drainage and desalting as well, it became apparent that these alternatives are also too costly to be feasible in the immediate future. However, with the recent advances in desalination technology, this alternative method of removing salts from irrigation return flows is certain to become increasingly feasible as time progresses.
Demonstration of Salinity Control Measures

The principal study area in Grand Valley, which has been used for evaluating the effectiveness of canal and lateral lining, as well as irrigation scheduling and tile drainage, in reducing the salt load entering the Colorado River, is now being used as a demonstration project beginning in February, 1974. The advantage in continuing to utilize this study area is that the hydrology is already known. In addition, there has been considerable expenditure of funds in both equipment and personnel for instrumenting this particular demonstration area. The wealth of available information provides a strong basis for evaluating the effectiveness of salinity control measures.

With the available knowledge regarding the study area, a lateral including the associated land served by the lateral water supply can be used as a subsystem for evaluating the salinity reduction in the Colorado River resulting from the implementation of a salinity control technology package. The study area was originally selected because of being fairly representative of the Grand Valley, while having five canals traverse the area, thereby allowing greater participation by the majority of irrigation entities in the valley.

In order to facilitate continued participation by most irrigation interests in Grand Valley, this demonstration project will utilize laterals under each of the five canals in the study area. These particular laterals have been selected to represent a wide variety of conditions. A few of the laterals have already been extensively lined with concrete under the previous demonstration project. The lands selected represent a variety of irrigation and drainage problems.

The laterals have been selected to capitalize on previous work regarding canal and lateral lining, as well as irrigation scheduling and drainage studies. The hydrologic knowledge already gained in this demonstration area allows routine surface water and groundwater monitoring to evaluate the overall effectiveness of the salinity control
Table 5. Completed and scheduled improvements by laterals for Grand Valley.

<table>
<thead>
<tr>
<th>Project Improvements</th>
<th>HL C</th>
<th>HL E</th>
<th>PD 177</th>
<th>GV 95</th>
<th>GV 160</th>
<th>MC 3</th>
<th>MC 10</th>
<th>MC 30</th>
<th>TOTAL</th>
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<tr>
<td>Concrete Ditch (LF)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<td></td>
<td>735</td>
<td>6700</td>
<td>6495</td>
<td></td>
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<tr>
<td>Buried Plastic Pipeline (LF)</td>
<td>1200</td>
<td>6829</td>
<td>5500</td>
<td>6880</td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>Gated Pipe (LF)</td>
<td>610</td>
<td>1870</td>
<td></td>
<td></td>
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<td></td>
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<tr>
<td>Drip Irrigation (Acres)</td>
<td>3.4</td>
<td>6.3</td>
<td>2.7</td>
<td>12.4</td>
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<td></td>
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<tr>
<td>Sprinklers (Acres)</td>
<td>12.2</td>
<td>15.0</td>
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<td></td>
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<tr>
<td>Concrete Drain Tile (LR)</td>
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<td></td>
<td>800</td>
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<td></td>
</tr>
<tr>
<td>Plastic Drain Tile (LF)</td>
<td></td>
<td>7200</td>
<td>12100</td>
<td></td>
<td>6425</td>
<td>16250</td>
<td></td>
<td></td>
<td>41975</td>
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<tr>
<td>Additional Labor Input (Acres)</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td>6.5</td>
</tr>
<tr>
<td>8&quot; x 3' Cutthroat Flumes</td>
<td>1</td>
<td>3</td>
<td>2</td>
<td>12</td>
<td>16</td>
<td></td>
<td>13</td>
<td>3</td>
<td>50</td>
</tr>
<tr>
<td>3&quot; x 3' Cutthroat Flumes</td>
<td>1</td>
<td>1</td>
<td>2</td>
<td>2</td>
<td></td>
<td>1</td>
<td></td>
<td>8</td>
<td></td>
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<tr>
<td>12&quot; φ Propeller Meter</td>
<td>1</td>
<td></td>
<td></td>
<td>1</td>
<td></td>
<td></td>
<td>3</td>
<td></td>
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</tr>
<tr>
<td>10&quot; φ Propeller Meter</td>
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<td></td>
<td></td>
<td>2</td>
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<td></td>
<td>4</td>
<td></td>
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</tr>
<tr>
<td>8&quot; φ Propeller Meter</td>
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<td>3</td>
<td>2</td>
<td>1</td>
<td>4</td>
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<td>7</td>
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<td>Other Propeller Meters</td>
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<td>1</td>
<td>1</td>
<td>1</td>
<td></td>
<td>5</td>
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<td></td>
</tr>
<tr>
<td>Irrigation Scheduling (Acres)</td>
<td>6.5</td>
<td>84.3</td>
<td>39.9</td>
<td>134.4</td>
<td>60.4</td>
<td>6.3</td>
<td>95.9</td>
<td>34.7</td>
<td>461.7</td>
</tr>
<tr>
<td>Total Acres</td>
<td>32.4</td>
<td>88.6</td>
<td>68.8</td>
<td>195.7</td>
<td>194.3</td>
<td>6.3</td>
<td>133.4</td>
<td>34.7</td>
<td>754.2</td>
</tr>
</tbody>
</table>

1. The number of measurement structures is not final.

2. This lateral was part of a previous drainage study and contains approximately 11,000 LF of agricultural drain tile.

3. These laterals were part of a previous lateral lining study and contain an additional 4,000 LF of concrete ditches.
technology package. Fortunately, the portion of lands to undergo treatment under this demonstration project, along with previously constructed channel lining and drainage facilities, will provide a significant impact upon salinity leaving the demonstration area.

The experimental design for the pre-evaluation will be primarily aimed at providing specific information for the 750 acres undergoing treatment. The field data collection program will allow the design of irrigation and drainage facilities, as well as providing sufficient data to allow predictions of salinity benefits that should result from each specific salinity control measure. Although the post-evaluation will include the monitoring of water and salts entering and leaving the demonstration area, the primary emphasis will be the on-site evaluation of each specific salinity control measure. The on-site evaluation can then be compared with the results of the demonstration area monitoring program, which in turn can be expanded to a valley-wide evaluation. The laterals being utilized in this program area shown in Fig. 13, while Table 5 lists the improvements that have been constructed.

The selection of a lateral as a subsystem, rather than an individual farm, has a tremendous advantage in allowing control at the lateral turnout. In this way, both the quantity of flow and the time of water delivery can be controlled, thereby facilitating improved water management throughout the subsystem.

A variety of irrigation methods will be demonstrated, including "tuning up" present irrigation methods being used in the study area. Considerable experience has been gained in improving the existing irrigation methods while evaluating irrigation scheduling as a salinity control measure in Grand Valley. However, more advanced irrigation methods have not been evaluated as to salinity benefits in Grand Valley. The irrigation systems to be constructed under this proposed project include automated farm head ditches, border irrigation, sprinkler irrigation, and trickle irrigation. Thus, one of the significant results from this project will be the preparation of a report, "Evaluation of Irrigation Methods for Salinity Control in Grand Valley."

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Fig. 13. Lands for which irrigation improvements are being made in Grand Valley.
The most significant aspect of this particular demonstration project is the employment of a salinity control technology "package," rather than a single control measure. Experience in Grand Valley has shown that the most significant progress is made when the gamut of questions can be answered regarding the interrelationships between water management and agricultural production. Thus, the concept of a technology package, along with an understanding of the "system" including other agricultural inputs, provides the necessary base for providing sound advice to the farmer, which in turn facilitates the development of credibility and consequently farmer acceptance. A report, "Composite Evaluations of Salinity Control Measures in Grand Valley," will be prepared describing the results from this demonstration project.

A two-day "Field Days" will be conducted during the third year (1976) of this project, probably during the month of August. This event will be primarily directed towards the farmers in Grand Valley and secondly to irrigation leaders (mostly farmers) throughout the Upper Colorado River Basin. Undoubtedly, some state and federal agency personnel throughout the West will also attend.

The currently funded EPA research project, "Irrigation Practices, Return Flow Salinity, and Crop Yields," which is being conducted in Grand Valley, will be utilized in developing the cost-effectiveness of each salinity control measure. In addition, the results from the research project will provide valuable information regarding increased crop yields that can be expected from improved water management practices. The combined results of the research project and this demonstration project are extremely important in establishing the benefits to be derived from implementing a salinity control technology package.

**Implementation**

The results from the demonstration project will be projected to valley-wide conditions in preparing the report, "Best Practicable
Salinity Control Technology for Grand Valley. This report, which will integrate several years of concentrated study in the valley, will serve as a basis for an action salinity control program. Such a program would detail the optimal strategy for implementing various levels of individual salinity control measures into a comprehensive technology package. To develop this kind of policy, cost-effectiveness functions relating the reductions in the system salt loadings resulting from a specified investment would be individually assessed in an optimizational format to arrive at the least cost combination for achieving a desired level of salinity control. Since salinity control in Grand Valley must evolve with the development of water resources in the Upper Colorado River Basin, this report will describe the time-varying characteristics of salinity control strategies. As other critical regions require salinity management, this report would serve as a procedural document illustrating analytical methodologies, data requirements, and strategy structures.

However, one question still remains--"How do we implement salinity control technology?" A significant portion of the answer to this question is related to institutional problems. The first step in institutional analysis is the study of local administrative controls.

As a part of the demonstration project, the effects of various institutional influences upon salinity control will be analyzed. For example, the effects of tailwater runoff control will be evaluated, along with the requirements for implementing a permit system, as well as the alternative of setting "influent" standards. The information necessary for analyzing the effects of each of the above alternatives will be collected as a part of the demonstration project. In addition, to allow the analysis to be projected valley-wide, field data will be collected on a sample basis throughout the valley. Although not all of the alternatives for implementing salinity control technology will be thoroughly analyzed under the demonstration project, every attempt will be made to collect the necessary "field" data for assessing alternatives. Thus, any remaining
alternatives must be analyzed on a much larger scale (e.g., regional, state, or federal).

Acknowledgments

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References


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MEASURING ECONOMIC SURPLUS CHANGES USING MATHEMATICAL PROGRAMMING MODELS: UTAH WATER ALLOCATIONS

by

John E. Keith, Jay C. Andersen, and Calvin G. Clyde*

Introduction

Since Utah is considered an arid state, the allocation of water among uses and regions in Utah is of considerable importance to Utah, and to other states. Several in state reallocation problems or plans were in evidence in the late 1960's and early 1970's and these reallocations appeared to be subject to analysis using systems analysis techniques. The modeling effort reported here for Utah was directed toward examining various parts of the reallocation question, and finally took the form of a complete allocation model.

The Policy Questions

Among the problems and plans which gave impetus to the model construction, the allocation of water to the growing Wasatch Front area in Utah from the Colorado River Basin via the various parts of the Bureau of Reclamation's Central Utah Project was of primary importance. The policy questions which were to be dealt with centered on three areas. First, was it economically feasible to import water for use in agriculture; second, was there an alternative to interregional water transfers for meeting use requirements which would be economically more feasible than transfers; and finally, if industrial activity and municipal growth continued or accelerated in various regions, what could be expected in terms of agricultural use given the costs of water delivery?

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Inherent in these policy questions were considerations of food output and energy development. Initially, in a project funded by the Office of Water Resources Research (now Office of Water Research and Technology) of the Department of Interior, the model for allocations between various fixed requirements for water was constructed. This model included the inter-basin transfers which were either in operation or proposed, including the Central Utah Project, storage and delivery systems, and water treatment. The model was constructed to minimize costs of meeting the fixed requirements for agricultural, municipal and industrial, and wetland (recreation, refuges, etc.) uses.

A second contract to generate regional allocation models based on economic efficiency was made with the Institute for Water Research, Corps of Engineers. The questions of water allocations between regions and uses relative to both water costs and water productivity was the focus of this model. This paper is a summary of the results of these research projects (Keith et al., 1973; King et al., 1972).

The Modeling Approach

The modeling method was mathematical programming, since such methods generate optimal solutions with which alternative policies can be compared. The mathematical programming format is:

\[
\begin{align*}
\text{minimize (maximize)} & \quad Z = CX \\
\text{subject to} & \quad AX = B \\
& \quad X \geq 0
\end{align*}
\]

where \(X\) is a vector of variables \(X_i\), \(c\) is a vector of costs (or returns), \(A\) is a matrix of coefficients for the constraint equations, \(a_{ij}\), and \(B\) is a vector of right-hand-side values for the constraints, \(b_j\).

The first models were constructed so that the objective function \((CX)\) was the cost of meeting requirements, which was minimized. The second, or allocation, model was constructed so that the objective function was net return (revenue-cost), which was maximized.
The models were then run under alternative values for specific constraints to generate optimal solutions under different assumptions. In addition to these solutions, shadow prices were generated which relate a one unit change in a given constraint to the resultant change in the objective function value; that is, the cost or benefit which results from a one unit (incremental) change in a given constraint can be found in what is known as the "dual" solution (which results from a restructuring of the model so that B is the objective function coefficient and C is the right-hand-side).

By using the shadow price from the cost model for alternative levels of water requirements (from zero to maximum water available), supply curves were constructed. This curve is analogous to a marginal cost curve for supplying water. By using the shadow price from the return model for alternative water availabilities (from zero to maximum), a demand curve was constructed, analogous to a "value of marginal product" curve. By combining the demand and supply models, the economically efficient solution was obtained (where marginal cost equals marginal benefit, or demand equals supply).

The specific models

The supply and demand models for Utah were developed separately. Submodels for supply were constructed for each of ten hydrological study units (HSUs) in Utah as indicated in Figure 1, and linked so that the total supply model would include inter and intrabasin water uses and/or transfers. Demand models were developed for each HSU. The allocation model was composed of the demand and supply models for each HSU, linked through downstream flows and other transfers of water.

The supply model

For the supply model, the objective function to be minimized was the total statewide cost of meeting agricultural, municipal and industrial (M & I), and wetland requirements, given the various sources of water supply and their respective costs of development, transport, and use. Each of these sources of water had variables which identified the
The subareas are identified as:

<table>
<thead>
<tr>
<th>Hydrologic Study Unit</th>
<th>Area Explanation</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>Columbia River</td>
</tr>
<tr>
<td>1</td>
<td>Great Salt Lake Desert</td>
</tr>
<tr>
<td>2</td>
<td>Bear River</td>
</tr>
<tr>
<td>3</td>
<td>Weber River</td>
</tr>
<tr>
<td>4</td>
<td>Jordan River</td>
</tr>
<tr>
<td>5</td>
<td>Sevier River</td>
</tr>
<tr>
<td>6</td>
<td>Cedar-Beaver</td>
</tr>
<tr>
<td>7</td>
<td>Uintah Basin</td>
</tr>
<tr>
<td>8</td>
<td>West Colorado</td>
</tr>
<tr>
<td>9</td>
<td>South and East Colorado</td>
</tr>
<tr>
<td>10</td>
<td>Lower Colorado</td>
</tr>
</tbody>
</table>

Figure 1. Hydrologic study units of Utah.
source and factors associated with that source. These variables are in eight general categories:

1. Variables indicating amounts of local surface water used in each of the HSUs;
2. Variables indicating amounts of groundwater used in those HSUs having sufficient groundwater to feasibly pump;
3. Variables dealing with storage of local surface water;
4. Variables accounting for evaporation losses from storage reservoirs;
5. Variables which associated return flows with each water use;
6. Variables indicating water used in recharging groundwater basins;
7. Variables dealing with water transfers existing or planned between HSUs (including the major transfer through the Central Utah Project); and
8. Variables indicating outflows from each of the HSUs to the appropriate receptor.

Each of these variables were included in constraints which placed limits on water use (either as equalities or definitions, or as maximum availabilities). These constraints were essentially of six types:

1. Constraints on the availability of surface water for allocation in a given HSU;
2. Draft requirements and evaporation are identical for storage projects in a given HSU;
3. Delivery, recharge, and treatment costs are constant in a given HSU; and
4. Return flows are constant for a given use in a given HSU.

Results from the Supply Model

Since the supply model was incapable of determining economic feasibility, the results which it generated were only partial, but some of the results were indicative of policy alternatives which substantially
alter allocation patterns. As water requirements were increased to indicate change over time, using parameterizations of appropriate variables, utilization of higher cost sources were indicated. Imposing limitations on low cost sources forced the use of higher cost sources more quickly. Timing of the development of the Central Utah Project was dependent upon the allowable use of groundwater and recharge activities along the Wasatch Front, and the inflows to the Great Salt Lake. As required inflows to the lake diminished (representing a lowering of lake levels), and as ground water pumping was increased (representing a relaxation of present groundwater use restrictions which preserve head pressure of current users), the Bonneville Unit transfers of the Central Utah Project were postponed and the Ute Indian Unit was not required.

Figure 3. Supply curve for agriculture in HSU 4, given M & I and wetland requirements for 1965.
In addition, supply curves for water were generated for each HSU. These supply functions were generated for each use, given alternative levels of diversions to other uses. For agricultural supply curves, various levels of M & I use represented projected water requirements for future years, so that changes in water availability to agriculture could be determined.

**The demand model**

The demand model uses returns net of production and development costs as the maximized objective function. Since the returns to water use for municipal, industrial, and wetland uses are not readily available nor easily researched, M & I and wetland demands were treated as requirements, just as in the supply model. These requirements had to be satisfied from existing available water before agricultural demand could be met. Thus, the demand model was based on agricultural net returns. For agricultural production, production was represented by variables which identified the activity and its output, costs, and revenue. The variables are grouped into seven categories:

1. Variables which identified crops to be produced;
2. Variables which related crops grown to rotation patterns;
3. Variables which indicated land classes available for production;
4. Variables which indicated water requirements by crop;
5. Variables associated with crop production and harvesting;
6. Variables which related development of new land for crops with new land preparations; and
7. Variables which related crop production to sales.

These variables were included in constraints which placed limits on agricultural production, again as equalities or inequalities. These constraints were of four general types:

1. Constraints on land availability;
2. Constraints on rotation of crops;
3. Constraints on production input requirements; and
4. Constraints on water availability.

The demand model can be illustrated using another schematic matrix as in Figure 4.

Several assumptions were made about the demand model:
1. Municipal, industrial, and wetland diversion requirements are fixed;
2. Agricultural productivity is fixed at 1980 projections for an average manager;
3. Agricultural prices will rise at the same relative rates as input costs; and
4. Timing of water delivery is irrelevant to water value.

<table>
<thead>
<tr>
<th>VARIABLE COSTS</th>
<th>AVERAGE COSTS</th>
<th>SELLING PRICES</th>
</tr>
</thead>
<tbody>
<tr>
<td>( C_1 )</td>
<td>( C_2 )</td>
<td>( C_3 )</td>
</tr>
<tr>
<td>( A_{11} )</td>
<td>( A_{21} )</td>
<td>( A_{31} )</td>
</tr>
<tr>
<td>( A_{12} )</td>
<td>( A_{23} )</td>
<td></td>
</tr>
<tr>
<td>( A_{13} )</td>
<td>( A_{22} )</td>
<td></td>
</tr>
<tr>
<td>( A_{24} )</td>
<td>( A_{25} )</td>
<td>( A_{35} )</td>
</tr>
</tbody>
</table>

Figure 4. Demand model diagram.
Results of the demand model

The demand model indicated that the value of water in agriculture was low relative to the municipal and industrial prices currently paid for all but the most productive lands. Seldom did the value of water in agriculture exceed $20.00 per acre-foot, and at current rates of application, $2.00 to $3.00 was an average value. Clearly, the value of water in use in agriculture is not as high as the cost of development of high cost water sources.

The demand curves were generated for agriculture using the parameterization of water availabilities previously described. Figure 5 is the agricultural demand curve for HSU 4.

Figure 5. Agricultural demand curve for HSU 4.
The allocation model

The economically efficient allocation of water is generated by combining the supply and demand models. M & I and wetland diversions are given as requirements and must be met from the supply of water; the agricultural demand is then equated to the remaining supply curve for the efficient solution. Figure 6 diagrammatically illustrates the allocation model.

As M & I demand increases over time, the available supply for agriculture decreases and the allocations are altered. Figure 7 illustrates the changes in agricultural water use over time. As M & I requirements increase, new supplies are developed to provide sufficient water. These new supplies include new groundwater, new surface water (storage), and transfers. The timing of these developments was indicated by the time-related M & I requirements which caused new water to be produced. Further, the inclusion of energy development was accomplished by adjusting M & I requirements appropriately in those basins in which energy development was proposed. At present, a moderate rate of growth in shale oil and fossil-fuel fired electrical plants is assumed, although work is currently under way to explicitly include energy as a water demand.

Results of the allocation model

The allocation model confirmed the implications of both the supply and demand models. First, no water transferred by the Central Utah Project would be used for agricultural purposes. Only a substantial subsidization could induce agriculturalists to use the water. Therefore, the development of the Bonneville Unit of the Central Utah Project (the Ute Indian Unit was not developed to 2000) is dependent upon the M & I requirements, the amount of inflow to the Great Salt Lake which was required in the model, the water salvage which might be undertaken for M & I use, and the utilization of groundwater resources which might be allowed. Under the most stringent assumptions (more than one
Figure 6. The allocation model.
Figure 7. The efficient allocations for alternative M & I requirements for HSU 4.
millon acre-feet annual inflow to the Great Salt Lake, no water salvage, no increase in groundwater use), the Bonneville Unit would reach maximum capacity by 1995. Under the least stringent assumptions (500,000 acre-feet inflow to the Great Salt Lake, salvage of 50,000 acre-feet, and 50,000 acre-feet use of the groundwater), the Bonneville Unit would not profitably reach full capacity until after 2020. Clearly, salvage and groundwater use are alternatives to the Central Utah Project, at least for the next thirty years. (See Figures 8 and 9.)

The model also indicated that another transfer which was in the planning stages by private agents would not be economically feasible. Negotiations have subsequently been abandoned. While this is not a statistical test of the model's reliability, it does indicate that the models are reasonable approximations of actual situations. As M & I diversions increased, particularly in some of the regions of anticipated energy development, agricultural activity was reduced in order to provide additional water. Even though M & I requirements were met by the model regardless of cost, intuitively M & I users could be expected to bid water away from agriculturalists.

The model's sensitivity to price changes for agricultural products was also examined, and indicated that while cropping patterns changed rapidly with price changes, development of new irrigated land in Utah would be minimal. This appears to be the case at present. Substantial increases in prices, relative to production costs, would be necessary to induce significant irrigation developments.

In addition to the allocation questions, the model is used to include calculations of surplus changes (welfare measures) which result from various policies concerning water allocations. Since those policies typically affect availability of water (supply), shifts in the supply curves can be examined using the model constrained to represent alternative restrictions. Areas between these supply curves and bounded by the demand curve, equal the losses of consumers' and producers' surplus.
Figure 8. Cup diversions
Inflo GSL = 1,014,000 (no salvage)
Medium to high projections
Figure 9. Cup diversion
Inflo CSL ≤ 850,000 (w/salvage)
Medium to high (HSU 5) projections
due to the restrictive policies. An example of such a calculation is given below.

Cost of groundwater pumping constraints in the Jordan River HSU

An example of using the study's methodology to determine the cost of institutional constraints can be illustrated by the restriction of groundwater pumping. Costs of providing water and the losses suffered by agriculturalists increased as a result of institutional constraints curtailing any groundwater pumping. Such curtailment is presently practiced along the Wasatch Front to protect head pressures of present wells and preserve maximum groundwater storage. For inflows to the Great Salt Lake greater than or equal to 850,000 acre-feet/year and no salvage, increased low-cost recharge was necessitated and full development of the Bonneville Unit was required in 1995. As a result, two kinds of losses were incurred. First, the users of water suffered higher costs, or losses in producers' surplus. Second, returns to new agricultural development were foregone.

Figure 10 illustrates the annual loss of producers' and consumers' surplus in HSU 4, the appropriate measure for this study since it was in HSU 4 that the timing of the "take off" and full development of the Bonneville Unit were determined. Given the assumptions of inflows to the Great Salt Lake greater than or equal to 850,000 acre-feet/year, no salvage, and groundwater pumping was limited to present quantities, full annual loss of producers' surplus occurred by 2000; the demand curve intersects the supply curve (S4 in Figure 10) above the price of transferred water at that time. Estimates of annual losses of surplus were made for each 10-year period, beginning in 1980 and ending in 2020, after which all annual losses were equal. Since there was no groundwater applied to present agricultural production in HSU 4, only

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M & I uses suffered increased costs. The supply curve without restrictive constraints is the $S^4$ curve and supply curve with restrictive constraints is the $S'^4$ curve. The crosshatched areas define the losses in producers' and consumers' surplus in HSU 4 as a result of the higher marginal cost curve. Table 1 is a tabulation of the losses of producers' surplus as indicated in Figure 10. The calculation of the losses of producers' surplus to M & I uses for a given period, therefore, is:

\[
(1) \quad (MC^4_{LRECH} - MC^4_{GW})(Q^4_{LRECH}) + (MC^4_{HRECH} - MC^4_{LRECH})
\]

\[
(Q^4_{HRECH}) + (MC^4_{TRANS} MC^4_{HRECH})(Q^4_{TRANS})
\]

The model indicated that a significant amount of new agricultural production would accompany exploitations of groundwater. This production would be gradually decreased as population centers encroach on rural areas until, by 2000, production would return to its current (1965) level. Income streams foregone to new agriculture would be approximately $12,000,000 at 5 percent and $8,500,000 at 7 percent.

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2 The following symbols used in Figure 14 are defined as:

- $MC_{TRANS}^4$ = Marginal cost of transferred water
- $MC_{LRECH}^4$ = Marginal cost of low-cost recharge in HSU 4
- $MC_{HRECH}^4$ = Marginal cost of high-cost recharge in HSU 4
- $MC_{GW}^4$ = Marginal cost of new groundwater in HSU 4
- $Q_{LRECH}^4$ = Quantity of low-cost recharged water to replace new groundwater
- $Q_{HRECH}^4$ = Quantity of high-cost recharge to replace low-cost recharge
- $Q_{TRANS}^4$ = Quantity of water transferred to replace high-cost recharge
- $Q_{GW}^4$ = Quantity of new groundwater used in HSU 4 on M & I requirements
Figure 10. Losses in consumers' and producers' surplus in HSU 4.

Table 1. Present value of producers' and consumers' surplus losses.

<table>
<thead>
<tr>
<th>Interest Rate</th>
<th>Period Beginning</th>
<th>Present Value at Period Beginning</th>
<th>Present Value Discounted to 1972</th>
</tr>
</thead>
<tbody>
<tr>
<td>5%</td>
<td>1980</td>
<td>9,521,000</td>
<td>6,446,000</td>
</tr>
<tr>
<td></td>
<td>1990</td>
<td>10,773,000</td>
<td>4,482,000</td>
</tr>
<tr>
<td></td>
<td>2000</td>
<td>12,118,000</td>
<td>3,090,000</td>
</tr>
<tr>
<td></td>
<td>2010</td>
<td>12,702,000</td>
<td>1,994,000</td>
</tr>
<tr>
<td></td>
<td>TOTAL</td>
<td></td>
<td>16,012,000</td>
</tr>
<tr>
<td>7%</td>
<td>1980</td>
<td>8,652,000</td>
<td>5,035,000</td>
</tr>
<tr>
<td></td>
<td>1990</td>
<td>9,790,000</td>
<td>2,898,000</td>
</tr>
<tr>
<td></td>
<td>2000</td>
<td>11,012,000</td>
<td>1,652,000</td>
</tr>
<tr>
<td></td>
<td>2010</td>
<td>11,543,000</td>
<td>877,000</td>
</tr>
<tr>
<td></td>
<td>TOTAL</td>
<td></td>
<td>4,329,000</td>
</tr>
<tr>
<td>12%</td>
<td>1980</td>
<td>6,961,000</td>
<td>2,812,000</td>
</tr>
<tr>
<td></td>
<td>1990</td>
<td>7,876,000</td>
<td>1,024,000</td>
</tr>
<tr>
<td></td>
<td>2000</td>
<td>8,859,000</td>
<td>372,000</td>
</tr>
<tr>
<td></td>
<td>2010</td>
<td>9,286,000</td>
<td>121,000</td>
</tr>
<tr>
<td></td>
<td>TOTAL</td>
<td></td>
<td>4,329,000</td>
</tr>
</tbody>
</table>
Conclusions

The study in general and development of the model in particular have led to several conclusions with respect to the general research approach:

1. The inclusion of demand and supply analyses as separate components avoids the problems involved in least-cost planning for projected demands. While this study did project M & I demands, using demands in the marginal, or least productive, activity did indicate that agricultural use changed as costs rose. The writers suggest inclusion of demand studies in all planning and feasibility studies where possible. The "requirements" approach to water planning lacks consideration of one-half the problems.

2. Multiple demands can be usefully included in a mathematical programming model so that efficient allocations among uses can be determined directly. In this model, the trade-offs among water uses (agricultural, municipal and industrial, and wetlands) were evaluated.

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

4. Hydrologic modeling can be effectively included in a mathematical programming allocation model, although some of the relationships must be generalized. The accuracy of the reproduction of the hydrologic system relationships is determined by the scope of the mathematical programming modeling effort.

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

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

7. Interdisciplinary research can be productive, particularly when a model such as this is the focus of study. Information exchange and cooperation can develop from developing such models, in part because of the requirements for structuring the model.

Some specific conclusions were reached concerning allocations of water in Utah:

1. The timing of development of the Bonneville Unit of the Central Utah Project is dependent upon the growth of M & I requirements for water in the Jordan River area, and upon the use of locally available alternative water sources, such as interception of inflows to the Great Salt Lake.

2. The cost of mistiming investment of public monies in the Bonneville Unit is of sufficient magnitude to warrant careful and explicit consideration of alternatives and requirements by public officials. If goals other than economic efficiency dictate inefficient allocations, then the costs which occur must be imputed to those goals.

3. In general, the value of water in agriculture is apparently too low to warrant development of elaborate and expensive transfer systems.

Model improvements

There appear to be at least four areas in which the model and the research approach in general could be improved.
First, the cooperation between public officials, responsible for decisions concerning water or other resource planning, and researchers could be improved. The benefits will be two-fold. The research and model will include the variables and coefficient values which decision-makers feel are appropriate, as well as those chosen by researchers. Modifications of the model using public decision-makers' inputs should lead to better understanding and utilization of the output of research efforts in public policy formulation.

Second, while quantity of water available was of course critical, quality of water may effectively limit water availability and, therefore, efficient allocations. For example, if quality standards are established by the Colorado River Compact for the outflow of water from Utah, treatment of industrial and agricultural return flows may be required, adding to costs and/or lessening demands. Quality standards for return flows in the Great Basin HSUs may similarly be reflected in allocations. The addition of quality constraints and alternative standards should be a prime goal of further research.

Third, the inclusion of value marginal product curves for M & I uses would make the model more truly allocative. Until the demand schedule for M & I water is known, the effect of the increased costs of M & I and agricultural transfers and quality requirements cannot be accurately judged. Further research is definitely required if the model is to indicate efficient allocations. The inclusion of such demand curves could enable more precise establishment of trade-offs between various sectors of the economy. Further, multiple goals could be added to the objective function or the constraint system to generate more information for decision-makers.

Finally, the coefficients used in the model were taken as constants, even though they are drawn from stochastic distributions. The effect of the variability (uncertainty) of the coefficients on the solution is not known. Stochastically programming at least portions of the model in which large variability occurs is a desirable goal for further research,
and should provide a better knowledge of the model's applicability to problems in resource allocation.

**Literature Cited**


King, A. B., J. C. Andersen, C. G. Clyde, and D. H. Hoggan. 1972. Development of regional supply function and a least-cost model for allocating water resources in Utah: A parametric linear programming approach. PRWG100-2, Utah Water Research Laboratory, Utah State University, Logan, Utah.

EVAPORATION SUPPRESSION BY DESTRATIFICATION
OF DEEP RESERVOIRS**

by

Trevor C. Hughes*

Introduction

Reservoir destratification has been practiced for several years at more than 20 impoundments in the United States for various water quality objectives. There is a growing body of literature on the results of artificial thermal mixing on various water quality parameters such as dissolved oxygen, taste and odor control algae, and plankton production. A collection of papers resulting from a series of related destratification research projects is included in a Public Health Service publication (Symons, 1969). Symons, Carswell, and Robeck have also written a state-of-the-art paper (1969). Both of these publications contain valuable information on the efficiency of the mixing process, the costs involved, the water quality impacts, and an approach to estimating destratification equipment capacity. The literature indicates that two principal methods for artificial mixing are feasible; pumping of cold deep water to the surface and pumping compressed air to the reservoir bottom. Both methods create a continuous mixing current which is capable of thermally mixing the entire reservoir, provided that sufficient mixing energy is used.

The physical limnology of a typical reservoir produces a significant difference in temperature between its shallow and deep portions as energy from the sun is absorbed during the spring and summer months. This phenomenon has been described in detail by several authors (Vallentyne, 1957, and Kittrell, 1965, for example) and only a brief summary of the aspects pertinent to the thermal mixing concept will be repeated here.

A cross section of a typical reservoir is shown in Figure 1. The wind mixes the top layer (epilimnion) but because of the difference in

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density which develops as the epilimnion absorbs energy from the sun, an increasingly large amount of energy is required to mix this warm lighter water with the cooler more dense hypolimnion as the spring and summer seasons progress. The density transition zone (the thermocline) acts as a seal which prevents the wind from mixing the water at lower depths.

The theoretical minimum amount of energy (the stability) required to destroy the thermocline has been defined as that energy necessary to lift the entire body of water the vertical distance between the center of gravity when the water is in any given state of stratification and the center of gravity when the water is isothermal (which is the minimum energy required to completely mix the stratified water). The actual energy required to accomplish such mixing has been found to be on the order of 100 times this theoretical minimum (Symons, 1969).

Typical temperature profiles for Lake Powell at various seasons are shown in Figure 2. The temperature profile is almost vertical (isothermal) in February, but the strength of the thermocline increases until in August there is a difference of 20°C between the surface and the bottom. During the fall months the thermocline decreases due to the loss of stored energy when the air is colder than the water.

The thermocline on smaller Utah reservoirs is not this pronounced but is still very significant on most impoundments over 50 feet in depth. Porcupine Reservoir, for example, with a depth of 140 feet has a thermocline of 13°C during July.

Although much research has been addressed to the water quality aspects of destratification, very little work has been done on the potential for evaporation suppression by this method. As a reservoir with a significant thermocline is mixed, it is apparent that some cooling of the surface and therefore a reduction in evaporation should occur.

The first suggestion in the literature that destratification has an evaporation suppression potential was apparently made by Abraham Streiff (1957). This brief paper included no estimate of the potential, but did recognize the concept. The only field trial where suppression
Figure 1. Summer thermal stratification pattern.

Figure 2. Lake Powell temperature profiles.
by thermal mixing was considered is described by Koberg and Ford (1965). This USGS Water Supply Paper is addressed primarily to the mechanics of the thermocline itself and of destratification techniques for water quality purposes. However, four pages of the paper are devoted to an estimate of changes in the evaporation rate on Lake Wohlford, California, during 1962 due to artificial destratification. This paper will be discussed later.

At the Utah Water Research Laboratory a state funded project for estimating the potential for water salvage has just been completed. One phase of this project was devoted to developing and applying a model which simulates the evaporation suppression potential of artificial destratification of deep reservoirs (Hughes, Richardson, and Franckiewicz, 1975). This paper will summarize the results of the UWRL study.

The objective of the UWRL project was to develop a mathematical model which:

(a) Simulates the change in reservoir surface temperature caused by destratification; (b) estimates the expected change in evaporation rate due to such thermal mixing; and (c) integrates the effect of the resulting changes in energy budget parameters over time.

In order to develop such a model, the following approach was used:

1. A theoretical method of expressing evaporation change as a function of water surface temperature change was developed.

2. This approach to calculating evaporation suppression was verified empirically by constructing a specially instrumented group of evaporation pans which included two artificially cooled pans.

3. Water temperature profile data were measured at several Utah reservoirs at monthly intervals for four summer months. Additional water temperature data were obtained for other major reservoirs from previous studies.

4. An evaporation suppression model was developed by combining the basic evaporation/temperature relationship with other parameters which are necessary to simulate surface temperature changes over time which are caused by thermal mixing.
5. The detailed model was applied to the 10 Utah reservoirs at which temperature profile data were available. The results on these reservoirs were used to develop a regression model by which suppression potential was estimated for all other impoundments in Utah.

The Suppression Model

Evaporation/water temperature relationship

It has long been recognized that evaporation is strongly correlated with water surface temperature and that this correlation is closely related to the well defined monotonic function relating water temperature to saturation vapor pressure. Evaporation is normally computed as a function of the vapor pressure deficit as follows:

\[ E = (e_{sw} - e_a)K \]

Where \( e_{sw} \) is saturation vapor pressure of a temperature equal to that of the water surface; \( e_a \) is the actual vapor pressure of the air; and \( K \) represents all of the non-temperature related parameters which influence evaporation such as wind (which are not of concern in this discussion). When the equation is applied to historic climatological data, air temperature is the parameter which is normally available. The saturation vapor pressure air temperature \( (e_{sa}) \) is used but with the implicit assumption that over a long period of time (May to October for example) the average water and air temperatures are very close and therefore that \( e_{sw} = e_{sa} \).

This procedure gives good results for seasonal evaporation estimates but may cause significant error in short term estimates when the two temperatures do not approximately balance.

In order to determine evaporation suppression as a function of change in water surface temperature, a form of the evaporation equation is desired which includes \( e_{sw} \) as a factor rather than an additive component in the evaporation equation. This revised form of the function would allow quantification of the change in evaporation as a function of
change in temperature without ever determining the actual evaporation magnitudes and therefore eliminating the need to determine wind and other unchanged parameters which are aggregated into $K$. The following derivation accomplished this desired modification of the equation:

Relative Humidity (R.H.) is defined as follows:

$$R.H. = 100 \frac{e_a}{e_{sa}}$$

therefore $e_a = (R.H.) e_{sa} / 100 \approx (R.H.) e_{sw} / 100$

If one accepts the substitution given above then:

$$E = e_{sw} (1 - \frac{R.H.}{100}) K$$

which is the desired form of the function. With an additional approximation, that R.H. is not changed by lowering the water temperature; the following ratios hold:

$$\frac{E_c}{E_n} = \frac{e_{swc}}{e_{swn}} = \frac{f(T_c)}{f(T_n)}$$

and suppression $= 1 - \frac{E_c}{E_n} = 1 - \frac{f(T_c)}{f(T_n)}$

where the c and n subscripts refer to cooled (thermally mixed) and normal conditions of the reservoir and $f(T_i)$ is the known function relating temperature to $e_{sw}$.

As mentioned previously, for short term measurements (a model with monthly time increments is anticipated) the substitution of $e_{sw}$ for $e_{sa}$ in the relative humidity definition may introduce a significant error. The size of this error and any others resulting from using the evaporation equation in this manner was investigated for the small scale situation as part of this project by using specially cooled evaporation pans. The results of this research are discussed later. The conclusion of the cooled pan experiment was that a suppression model based on the equations developed previously clearly gives a conservative estimate of suppression for periods when the temperature of the air averages not less than the water surface temperature. This was the case for the evaporation pans even during October and this inequality should clearly hold for reservoirs during the summer when the majority of evaporation occurs.

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Extension of model from pan to
lake evaporation

There appears to be no error introduced by treating wind as a constant and thereby eliminating it from the ratios derived in the previous section because thermal mixing has no measurable effect on wind. The elimination of relative humidity, as being independent of water temperature, however, requires some qualification. It is no doubt true that in the case of an evaporation pan the relative humidity or $e_a$ is independent of a change in water temperature. The air mass passes over the pan too quickly to be effected by the lower boundary temperature. This may also be true of small reservoirs but probably will not be true in the case of larger reservoirs.

In order to understand the effect of artificially cooling the water surface (by thermal mixing) and the error introduced by the simplified model, it is necessary to consider evaporation in for hypothetical situations; both natural and thermally mixed conditions at both an upwind and downwind point on a large reservoir.

Figure 3 shows the change in vapor pressure for assumed temperature and humidity conditions representing the four situations described above. The following assumptions and parameter values were used in developing Figure 3: because of the huge difference in specific heat of water compared to air, the air temperature is effected (cooled) by the water between points 1 and 2, but the water temperature is not changed by this temperature gradient. The air and water temperatures and humidity shown represent conditions on a typical summer day at the canyon mouth in Logan, Utah. The indicated changes in temperatures and humidity are arbitrary but reasonable relative values.

Suppression at point 1 (representing a small reservoir or the upwind section of a large reservoir) by the Dalton equation is:

$$\text{Supp}_1 = 1 - \frac{\Delta e_{cl}}{\Delta e_{nl}}$$

Suppression at this point as computed by the model is:
Figure 3. Comparison of evaporation by Dalton equation and by simplified model.
It is apparent from the figure that the model adds the same constant to both the numerator and denominator of a fraction which is less than unity, thereby insuring a conservative model for the small reservoir (19 percent lower in this case). It is also apparent that while the model estimates suppression adequately for the parameter values shown, its error increases as vapor pressure increases. This is not considered to be a serious problem since evaporation suppression is likely to be worthwhile only in arid regions and the time when model accuracy is most important is the dry summer period.

Suppression at point 2 (representing a large reservoir) by the Dalton equation is as follows:

$$\text{Supp}_{2} = 1 - \frac{\Delta e_{c2}}{\Delta e_{n2}}$$

Model suppression at this point is the same as at point 1 because of the constant R.H. assumption. In this case the model adds a smaller constant to $e_{c2}$ than to $e_{n2}$ thereby simulating the true suppression more accurately than for the small reservoir. The model still appears to be conservative (8 percent lower than the Dalton equation for the particular situation depicted in the figure) but a rigorous mathematical proof of error on the conservative side is not possible for this case.

Energy budget consideration

The previous discussion of potential suppression has been essentially in context of the relationship between parameters at any given point in time. The integration of these effects on a reservoir during a season or a series of years requires the consideration of heat addition and loss sources over time.

A destratification study by the U.S. Geological Survey on Lake Wohlford, California, included the only previously published attempt to analyze the effect of thermal mixing on evaporation (Koberg, 1965).
Even though the reservoir was less than 50 feet deep, a 5 percent net saving in evaporation was computed. The researchers had expected a negative effect in the fall which equaled the suppression effect during the early summer. They explained the apparently unexpected net saving as being due to drawdown during the fall. In addition to drawdown, however, the model for Utah reservoirs indicates that a related but more important factor may be the increased temperature of released water (which may be operative even under negative drawdown conditions). The hypothesis of the author in this regard (which anticipates a net suppression rather than an annual balance) is two-fold.

1. Thermal mixing achieves a significant increase in temperature of water flowing from the reservoir outlet (assuming the outlet is near the reservoir bottom or at least below the thermocline) and therefore a net decrease in reservoir heat is accomplished by flow from the reservoir.

2. On reservoirs for which winter carryover storage is a minor part of annual storage, residual winter heat is unimportant. In this situation, a May to October suppression of evaporation (the irrigation season) for example, is all that may be of concern because spring runoff will fill the impoundment anyway (and will dominate subsequent water temperatures).

On reservoirs which have large carryover storage factors however, the comparison between heat added by suppression and increased heat lost from the outlet will be of key importance in determining the net annual evaporation suppression. This can best be visualized by considering the significant sources of heat flux in a reservoir energy budget. U.S. Geological Survey researchers have defined nine such variables as constituting energy budget parameters on studies of Lake Mead (Harbeck et al., 1958), Lake Colorado City (Harbeck et al., 1959) and the Salton Sea (Hughes, 1967). In these papers the energy budget was defined as follows:

\[ Q_s - Q_r + Q_a - Q_{ar} - Q_{bs} + Q_v - Q_e - Q_h - Q_w = \text{Net Change} \]

The first four terms are respectively the incoming and reflected solar radiation and atmospheric long-wave radiation. Although these
four are major items in the energy budget, they can be ignored in the
proposed model because they are independent of changes in surface tem­
perature (except in the very minor portion of the surface area where the
albedo may be changed by air bubbles).

Qbs is the long wave radiation emitted by the body of water. This
is an important component of the energy budget which is a function of
surface temperature. This heat loss will be decreased by destratifica­
tion during the summer (a negative effect on suppression) and increased
due to residual heat during the winter (a positive effect). The order of
magnitude of the change in this parameter can be estimated by examining
the Lake Mead heat budget cited previously. The range of variation in
this parameter was +12% and -9% from the mean during the summer and
winter respectively. The temperature changes summer and winter caused
by mixing and by residual heat, however, will be perhaps 10 percent of
the natural temperature extremes giving an artificial variation in this
parameter of about 1 percent and the net annual difference due to mixing
should therefore be negligible.

Qv is the net energy advected into the reservoir. The inflow is
unchanged by destratification but the outflow is significantly affected be­
cause of the increased temperature below the thermocline. This is a
beneficial effect both during summer (due to mixing) and during winter
(due to the residual heat added by suppression). This is therefore poten­
tially a very important beneficial parameter which is a function of the
outflow/storage ratio. The mixed reservoir temperature decrease due
to this parameter during any time period is equal to the increase in out­
let temperature multiplied by the outflow/storage ratio.

Qe is the heat removed from the reservoir by evaporation. When
evaporation is decreased this becomes a primary source of added heat
and possibly subsequent above normal evaporation. The quantity Qe is
the latent heat of vaporization which varies slightly with temperature at
which the process occurs but is close to 585 calories per gram of water
evaporated at typical reservoir temperatures.
Qh is the energy conducted out of the reservoir as sensible heat. This represents a very small portion of the heat budget (Harbeck et al., 1958) which will become a negative influence on suppression in summer and positive in winter. Analysis of the size of this parameter compared to other factors in the energy budgets of the USGS studies indicates that only negligible error is introduced by ignoring the change in this parameter caused by thermal mixing.

Qw is the energy advected by evaporated water. This is the smallest of all items in the USGS energy budgets and may be ignored (or could easily be added to the latent heat computation Qe).

In summary, two of the nine energy budget parameters appear to dominate the calculation of evaporation suppression secondary effects; these are Qe (evaporation latent heat) and Qv (the outflow component). It would appear that without any outflow, there would be no net annual suppression; that is, the savings during the summer season would be essentially completely dissipated by increased evaporation later during the decay of residual heat added by suppression. However, manmade impoundments do typically have large outflows and therefore the increased outflow temperature should represent a significant net savings.

The importance of the beneficial effect of outflow temperature increases due to destratification is interesting in relation to previous research on monolayer suppression. The fact that deep outlet temperatures are not increased during monolayer treatment suggests that the added heat from reduced evaporation would tend to limit suppression by that method to a net seasonal amount which would tend to approach zero when analyzed on an annual basis. In addition to this advantage of thermal mixing over the monolayer concept in the long term, a similar advantage occurs with daily suppression rates. During the monolayer operation, above normal evaporation begins immediately upon wind stripping of the chemical film because of added heat which accumulated near the surface during film coverage. Although a similar amount of heat is added to the water during suppression by thermal mixing, it is continuously mixed and distributed equally throughout the reservoir rather
than concentrated above the thermocline. At any point in time therefore
the surface temperature increase due to the parameter is much less than
for the monolayer method.

Summary of model conceptualization

The suppression by thermal mixing model which has been described
and justified in general terms here is developed in detail in the UWRL
report (Hughes et al., 1975). It can be conceptualized in abbreviated
form as follows.

Basic concept. Idealized suppression is calculated as a function of
change in water surface temperature (ergo change in vapor pressure)
caused by perfect mixing (constant temperature profile).

Secondary effects. The secondary effects which are sufficiently
important to be included in the model are: 1) heat added due to the de-
crease in heat of vaporization caused by suppression during a previous
period and 2) heat loss from the reservoir due to warmer than normal
outflow from the mixed reservoir.

Time resolution. Model parameters are determined on a monthly
average basis. Heat flux is accumulated between months up to six
months for reservoirs on which a seasonal (May to October) analysis is
appropriate and for annual or multiyear periods where carryover storage
is important.

Errors in the model. The model structure includes assumptions
which tend to make the model conservative as discussed previously and
as verified by the pan experiment.

A source of error which has not been discussed previously is that
the model assumes perfect thermal mixing; that is, conversion of the
normal thermocline into a perfectly vertical temperature profile. Re-
results on many reservoirs which have been mixed for quality objectives
indicate that this is feasible except for a very minor diurnal variation on
the order of 1°C. Many of the empirical results did not achieve this
degree of mixing because pumping was limited to that necessary for de-
sired dissolved oxygen levels. But those projects on which pumping witl
sufficiently large energy sources were operated continuously seemed to produce almost perfect mixing. No correction for this non-conservative error was incorporated into the model.

**Evaporation pan experiment**

In order to empirically determine evaporation change as a function of water temperature change only, an experiment was designed with evaporation pans at different water temperatures but at the same location so that identical wind and humidity conditions were acting upon each pan. Three evaporation pans were used at the station. Two of the pans were cooled by running water through coils beneath the water surface. The third pan had no temperature control; it was a normal evaporation pan serving as a standard reference. Air and water temperatures, dew point, precipitation, and wind were all recorded and converted to daily averages.

This phase of the research is described in detail in an M.S. Thesis by James Frankelwicz (1975). The data and analysis are also included in the UWRL report.

Empirical suppression values were consistently higher than the suppression predicted by the model for the temperature changes involved (as expected from the simplified model discussion). A comparison of the model suppression to ten day averages of the daily measured suppression is shown in Figure 4. The conservative nature of the model is apparent from the difference between the two best fit linear functions. The multiple points at each model value of \( \Delta T \) indicate the slight variation of suppression with original (natural) water temperature over the expected range. This variation is better shown in Figure 5.

**Increased evaporation below impoundments**

Since the annual net suppression is closely related to the removal of excess heat from the reservoir an important question in applying the model to some reservoirs is how much of this claimed net benefit is ultimately lost by the resulting increased evaporation in the river, canal, or other downstream reservoirs.
Figure 4. Comparison of model and empirical equations.

Figure 5. Model suppression as a function of water surface temperature.
On most Utah impoundments the reservoir outflow after high spring runoff is transported almost entirely in either pipelines or canals to the point of use. The travel time to the point of use is relatively short so that additional evaporation from canals is negligible.

For the major Colorado River impoundments (Lake Powell and Flaming Gorge) the increase in river losses appears also to be minor. For example, flow from Lake Powell travels through 295 miles of narrow river before entering Lake Mead. The travel time is less than 3 days and the additional evaporation in the river due to the added heat is estimated at 3 to 4 percent of the annual volume of water salvaged (Hughes et al., 1975).

When this warmed flow enters Lake Mead it will still be colder than the water above the thermocline and will be stored at a depth well below the surface. It was beyond the scope of this study to model the long term impact of this heat on Lake Mead. It would appear that a multiple year destratification operation on Lake Powell would eventually cause a slight increase in the surface temperature of Lake Mead (1°C or less). Part of the additional heat would be dissipated by increasing evaporation at Lake Mead and part would be removed via the river. Additional research is needed to determine the fraction of the Lake Powell (or Flaming Gorge) suppression that would be lost downstream, but it appears to represent a minor fraction of the volume claimed for the upstream reservoir.

Results of Model Application

Detailed model

The suppression model was applied to 10 Utah reservoirs for which several months of summer temperature profile data were available. The results of the May to October model are summarized in Table 1.

Residual heat is the temperature increase above natural water surface temperature at the end of October. The negative values in this column for 5 of the 10 reservoirs indicate that the beneficial effect of
Table 1. Six month model results.

<table>
<thead>
<tr>
<th>No.</th>
<th>Reservoir</th>
<th>Elev.</th>
<th>Maximum Depth</th>
<th>Average Depth</th>
<th>Suppression Idealized (A)</th>
<th>Final (C)</th>
<th>Residual Heat (°C)</th>
<th>Flow Index</th>
<th>C/A</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Bear Lake</td>
<td>5925</td>
<td>225</td>
<td>224</td>
<td>28.8</td>
<td>19.0</td>
<td>2.7</td>
<td>.01</td>
<td>.66</td>
</tr>
<tr>
<td>2</td>
<td>Deer Creek</td>
<td>5420</td>
<td>137</td>
<td>123</td>
<td>19.0</td>
<td>19.5</td>
<td>0.1</td>
<td>.29</td>
<td>1.03</td>
</tr>
<tr>
<td>3</td>
<td>Flaming Gorge</td>
<td>6000</td>
<td>439</td>
<td>421</td>
<td>26.8</td>
<td>22.7</td>
<td>1.0</td>
<td>.05</td>
<td>0.85</td>
</tr>
<tr>
<td>4</td>
<td>Hyrum</td>
<td>4885</td>
<td>72</td>
<td>61</td>
<td>14.4</td>
<td>19.6</td>
<td>-1.7</td>
<td>.79</td>
<td>1.40</td>
</tr>
<tr>
<td>6</td>
<td>Lake Powell</td>
<td>3700</td>
<td>561</td>
<td>480</td>
<td>43.3</td>
<td>33.9</td>
<td>3.0</td>
<td>.04</td>
<td>.78</td>
</tr>
<tr>
<td>10</td>
<td>Pineview</td>
<td>4900</td>
<td>82</td>
<td>70</td>
<td>16.9</td>
<td>14.7</td>
<td>-0.3</td>
<td>.21</td>
<td>.87</td>
</tr>
<tr>
<td>11</td>
<td>Porcupine</td>
<td>5800</td>
<td>141</td>
<td>105</td>
<td>19.0</td>
<td>24.8</td>
<td>-1.0</td>
<td>.71</td>
<td>1.30</td>
</tr>
<tr>
<td>12</td>
<td>Scofield</td>
<td>7580</td>
<td>45</td>
<td>37</td>
<td>4.8</td>
<td>4.6</td>
<td>-1.1</td>
<td>.09</td>
<td>.96</td>
</tr>
<tr>
<td>13</td>
<td>Sevier Bridge</td>
<td>5015</td>
<td>72</td>
<td>62</td>
<td>6.0</td>
<td>4.5</td>
<td>1.1</td>
<td>.20</td>
<td>.75</td>
</tr>
<tr>
<td>14</td>
<td>Starvation</td>
<td>5800</td>
<td>147</td>
<td>122</td>
<td>10.3</td>
<td>11.2</td>
<td>-1.0</td>
<td>.19</td>
<td>1.09</td>
</tr>
<tr>
<td></td>
<td><strong>Average</strong></td>
<td></td>
<td></td>
<td></td>
<td><strong>18.9</strong></td>
<td><strong>17.5</strong></td>
<td><strong>0.4</strong></td>
<td><strong>.26</strong></td>
<td><strong>.35</strong></td>
</tr>
</tbody>
</table>

1 Average of all reservoirs

2 Average without high carryover reservoirs (1, 3, and 6)
releasing additional heat at the outlet exceeded the negative effect of heat added due to suppression for these reservoirs. In general, the low residual heat is strongly correlated with flow index. This index is computed as a ratio of monthly outflow to total storage. These May to October ratios are weighted according to evaporation rate for each month and combined to produce the seasonal flow index.

Bear Lake is the only natural lake included in the table, and therefore the only one which does not have an outlet below the thermocline. This results in a relatively low 6 month suppression (relative to its depth) which would actually become negative in the annual model because the outflow is being cooled rather than warmed by thermal mixing. The obvious conclusion is the natural lakes (or man made reservoirs with high outlets) should never be destratified for the purpose of evaporation suppression.

The long term model was applied to Lake Powell (the only reservoir for which year around temperature data were available). The results are summarized in Table 2. The initial year's suppression of 27.3 percent reduces to 22.7 percent (140,200 ac. ft.) for continuous operation.

Table 2. Sequential mixing of Lake Powell.

<table>
<thead>
<tr>
<th>Year</th>
<th>Percent Suppression</th>
<th>Residual Heat (°C)</th>
<th>Salvage During Year (Acre feet)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>27.3</td>
<td>2.39</td>
<td>171,500</td>
</tr>
<tr>
<td>2</td>
<td>23.2</td>
<td>2.82</td>
<td>145,800</td>
</tr>
<tr>
<td>3</td>
<td>22.4</td>
<td>2.90</td>
<td>141,100</td>
</tr>
<tr>
<td>4</td>
<td>22.3</td>
<td>2.91</td>
<td>140,200</td>
</tr>
<tr>
<td>over 4</td>
<td>22.3</td>
<td>2.91</td>
<td>140,200</td>
</tr>
<tr>
<td>N+1*</td>
<td>-6.0</td>
<td>0.2</td>
<td>-36,000</td>
</tr>
</tbody>
</table>

* The year after mixing is stopped evaporation of 6% over normal will occur.

Multiple regression model

The results of the 6 month model application to 10 Utah reservoirs (see Table 2) were used in a multiple regression analysis to develop a
model for estimating suppression on reservoirs for which temperature profiles are not available. The only parameters other than water temperature which appear to be significant in determining suppression are depth, flow index, and possibly elevation. The best fit equation is:

\[
\text{Suppression} = 5.434 + 10.908 \log(D) - 5.341 \log(E) + 15.479 \text{Flow}
\]

in which \(D\) is maximum depth in feet, \(E\) is elevation in feet and Flow is the flow index described previously. The correlation coefficient \(R\) for this function is 0.940. The dominant correlation parameter is depth. Both maximum depth and average depth during the suppression season were compared and they both gave equally good correlations.

The parameter which contributed the second highest improvement in correlation is the flow index. In fact, almost the same correlation (\(R = 0.939\)) can be achieved by eliminating elevation as follows:

\[
\text{Supp.} = -42.49 + 11.291 \log(D) + 16.248 \text{Flow}
\]

Figure 6 represents this function.

**Economic feasibility**

The best source of cost information on reservoir destratification is contained in an AWWA committee report (Symons, 1971). This paper summarized the results of a survey of water suppliers who have used artificial destratification for water quality reasons. The report includes three figures showing energy capacity capital investment costs and operating and maintenance costs, each as a function of reservoir volume. The figures include data from 24 reservoirs upon which various air and water pumping systems were used. The cost/volume regression plots include considerable variability but all figures show substantial economies of scale.

Destratification costs on Utah reservoirs were estimated (in 1970 dollars) by using the Symons cost data with equipment life of 15 years and 7 percent interest. The analysis indicates four major reservoirs with costs under $5 per ac. ft. of water salvaged; nine with costs under $10 and nineteen with costs under $20. The total under $20 Utah annual net salvage estimate is 170,000 ac. ft.
Lake Powell benefit/costs

The single impoundment which dominates the potential in terms of total water salvaged (140,200 ac. ft.) also has the lowest salvage cost ($2 per ac. ft.). Destratification of this huge impoundment also has some interesting energy related implications.

If the salvaged water is valued at $10 per acre foot and salvage costs are $2, the net profit of such an operation is $1.1 million annually. But in addition to the water revenue, the impact of energy supply and demand should be considered. The equipment required to destratify Lake Powell is estimated at 6,000 HP operating continuously for 6
months \((19.3 \times 10^6 \text{ KWH per year})\). If the 140,000 ac. ft. of salvaged water is then used to produce hydropower as it leaves the reservoir at 484 ft. average head and .905 overall efficiency (USBR, 1970) this represents \(63.5 \times 10^6\) KWH per year of added generating capacity.

This suggests that 3.3 times as much power would be generated from the salvaged water as it took to "create" the water. The power revenue profit from this operation would of course depend upon whether wholesale or retail costs are used in the analysis. If \$0.01\) per KWH is selected as a value of electrical power, the potential annual revenue from hydropower generation at Glen Canyon Dam alone would be \$635,000

After generating hydropower, the salvaged water would still be available for other uses such as cooling of fossil fuel fired generators. Using 15 ac. ft. of water per MW of power capacity as the cooling requirement (Western States Water Council, 1974), 142,000 acre feet of additional water could provide the cooling for 9,500 MW of generating capacity.

Another aspect of the potential benefits on Colorado River impoundments is related to the salinity problem in the Lower Colorado Basin. Ti damages to agricultural production in the lower basin due to increased salinity in the river has been estimated by the USBR at \$230,000/ppm. Evaporation suppression in effect adds water with zero salinity to the reservoir. On Lake Powell for example the ratio of water salvaged to average reservoir storage (16 maf) is 0.9 percent. The TDS of Lake Powell at Wahweap is 600 to 700 ppm (USBR unpublished Lake Powell Quality Data). An addition of 0.9 percent of pure water annually should lower the TDS of the Lake by 6 ppm. Since flow through Powell represents almost the entire flow to the lower basin this indicates an annual dilution benefit of \$1,380,000\) to irrigated agriculture for quality improvement in addition to the value of the ultimate use of the salvaged water.
Conclusions

The use of thermal mixing by either mechanical pumping or compressed air appears to have important potential for evaporation suppression on deep reservoirs. This concept does not involve many of the problems associated with monolayer suppression. It is independent of wind; it does not produce a concentration of excess heat near the surface; it provides a mechanism for net suppression on an annual rather than only a seasonal basis (warmed outflow); and it does not involve environmental problems associated with adding chemicals to the reservoir surface.

There are, in fact, several environmental benefits claimed for the destratification procedure. Significant improvement in dissolved oxygen, taste and order, algae production, and many other quality parameters occur in the hypolimnion water. In addition to these human related benefits, fish habitat may be improved both in the reservoir (because of increased DO) and downstream from it. Below Flaming Gorge Dam, for example the water being released is presently so cold that native species of fish in the area are becoming endangered. The warmed outflow from a thermally mixed reservoir would help this situation.

The necessary conditions for significant evaporation suppression appear to be as follows:

1. Sufficient depth (usually more than 60 feet) to produce a marked natural thermocline and to provide a relatively large volume of cold hypolimnion water for mixing.

2. An outlet that is below the thermocline and sufficient outflow in relation to storage to transport a significant amount of excess heat from the reservoir.

The model developed in connection with this research appears to produce reasonably accurate but conservative estimates of suppression for impoundments in an arid climate but the model error increases rapidly as average humidity increases.

A major potential for water conservation in general and water for energy production and salinity control in particular exists at Lake Powell.
Additional research addressed to the actual destratification equipment capacity, design and costs at Lake Powell as well as other smaller reservoirs should produce very cost effective results.

Selected Bibliography


BAYESIAN DECISION ANALYSIS APPLIED TO RIVER BASIN STUDIES

by

Donald R. Davis and Rick Patten*

Decision theory addresses the problem of making and evaluating decisions based on methods, models, and information which, to varying degrees are uncertain and where there may be substantial economic and social loss if the decision is incorrect. Among the advantages of this type of analysis is the ability to use, in a constructive manner, less than precise data. When uncertainty does not mean total ignorance, but implies something in between ignorance and precise and accurate knowledge, then decision theoretic techniques allow the use of uncertain knowledge for the determination of better decisions. This can be done because the uncertain knowledge is treated as a probability distribution and the entire decision-making process is handled in a probabilistic framework. It amounts to an extended probabilistic sensitivity analysis.

Decisions must often be made for projects to be located at places where there is insufficient data for satisfactory design to be determined in the usual manner. The remedy is usually accomplished by the use of other data, which is transformed through some model to the data desired, or by the transference of data from an area with like characteristics (regionalization). The transformation of rainfall to runoff is an example of the first method. The concept of regionalized skew coefficients for modeling peak river flow is an example of the second. The transference of data in form or space, or both, is often done within a river basin because of the homogeneity of the relations governing the transference.

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One might think that with time, as data banks get larger, the need for techniques of transference and regionalization might diminish. However man's activities have changed the nature of parts of the river basin so historical records may not reflect the current situation. Man's activities have also created the need for data that was not of interest to a previous generation. This is especially true of environmental concerns. Transformation and translation of data is a necessary part of environmental study because large scale collection of data concerning environmental quality is a recent phenomenon.

use of models, such as rainfall runoff models, regression models, etc.

It is hoped that these models adequately reflect conditions throughout the river basin. Conditions in the basin are not uniform; there is some heterogeneity which may make the models inaccurate and lead to uncertainty in the use of the outputs from these models.

The purpose of this paper is to discuss the use of decision theory in the context of data transformation and translation and review some of the work in the area. First we will review some of the fundamentals of decision theory and then by way of illustration, and for perspective, we will make a brief analysis of the Colorado River Compact of 1922 from the viewpoint of decision theory.

**Decision Theory**

This theory is concerned with the uncertainties in the knowledge the engineer has concerning the outcomes to be obtained by the various decision alternatives available. The word "outcome" is used to describe those output(s) of the system or project which indicate to the decision-maker how well the goals of the project are being reached. The desirability of various outcomes are quantified by a goal function which may range from project capacity through project profitability to an index of "social goals." Thus, the effective use of decision theory requires a systems approach to the decision problem at hand.
The essential steps in the application of decision theory (to hydrologic design), as given by Davis et al. (1972) are given below:

A. Define the goal.

B. Define the decision to be made and identify the alternatives.

C. Analyze the project.
   1. Define the goal function.
      a. Select the state and decision variables.
      b. Set a time preference.
      c. Include a risk aversion.
   2. Make a sensitivity analysis.
   3. Develop the stochastic properties of the knowledge of values of the state variables as a probability density function.
   4. Calculate the outcomes of the various alternatives and determine the stochastic properties of these outcomes.
   5. Eliminate the dominated alternatives.

D. Make the decision.
   1. Calculate the expected value of the goal function for each alternative.
   2. Choose an alternative to minimize the expected value of the goal function.

E. Evaluate the decision.
   1. Determine the expected opportunity loss due to uncertain parameters in the problem.
   2. Evaluate information-gathering programs.
      a. Determine the expected reduction in the expected opportunity loss with further information.
      b. Determine the full cost of obtaining further information.
      c. Obtain further information if warranted, and repeat the analysis.
It should be noted that the measure used to rank various outcomes is the most expected value of a goal function. The expected value criteria is most commonly used but is not mandatory. Decision theory is discussed at greater length in Benjamin and Cornell (1970) and Raiffa and Schlaifer (1961).

From this summary, it may be seen that decision theory may be used to evaluate decisions, as well as to make decisions. The evaluation is accomplished by calculating the loss to be expected due to the uncertain information available to the engineer and decision-maker. (The use of decision theory minimizes this loss but does not eliminate it.) Knowledge, such as additional data or improved models, which reduces the uncertainty in the information available, will enable a better decision to be made, that is, a decision in which less loss is to be expected due to uncertainty. The expected reduction in this loss is a measure of the value of this knowledge. We term this a reduction in the expected opportunity loss.

**Colorado River Compact**

We would like to illustrate some aspects of decision theory by examining the background of the Colorado River Compact of 1922. The Colorado River System was providing plenty of uncertainties for those whose lives and well being were intertwined with it. Was there going to be flooding? Would there be enough water at the Imperial Valley diversion points throughout the season? Would the upper basin states have their future development stymied because California might get rights to most of the Colorado's water by the doctrine of prior usage.

The Colorado River Commission met in 1922 to lay the foundation for solving these problems by dividing the waters of the Colorado at Lee Ferry, Arizona. For a while it might have seemed that they merely traded one set of uncertainties for another. How would the flow at Lee Ferry be determined; there was no gaging station there.
What was a fair share of how much water? How would the natural variation in the river's flow affect delivery schedules? The commissioners knew they were dealing with uncertainty, they talked of averages and means and minimum flows, and later in their deliberations of three year minimums and average means and minimum averages. They faced natural uncertainty in the river's yearly flow and sample uncertainty in their estimate of its mean annual flow. As practical men, if not decision theorists, they knew that to talk of using the mean flow they had to talk of storage, so the high flows could be kept for use in low flow years.

Their data base was about 20 years of reliable record and a few additional years of less reliable data. These figures were for Laguna and the virgin flow at Lee Ferry was reconstructed from these figures and from estimates of upper basin consumptive use. The data did not seem to have been subject to much statistical analysis, they worked with the yearly flows individually and the means of the whole record and various portions of the record.

Decision theory requires a probability distribution to describe the natural uncertainty, so over 50 years later we give the Colorado a chi-square test and settle a normal distribution on its annual flows. We calculate the mean and variance, based on the record from 1900 to 1921. For a loss function we draw an analogy with reservoir operating rules and the concept of target yield. The decision variable is the target yield, a yearly gain is accrued in proportion to the target yield, but if the target yield is not met a loss is incurred which is proportional to $K$ times the deficit:

$$B(y, x) = \begin{cases} 
  y & \text{if } x > y \\
  y-K(y-x) & \text{if } y \geq x
\end{cases}$$

where $y$ is the target yield and $x$ is the water delivered.
The water delivered is a random variable depending on the river flows. The optimal decision alternative is the target yield which maximizes the expected benefits:

\[
E[B(y, x)] = \int_0^\infty B(y, x) f(x \mid \theta) \, dx
\]

\[
B(y) = y - \int_0^y K(y-x) f(x \mid \theta) \, dx,
\]

where \( f(x \mid \theta) \) is the probability distribution function of the random river flow \( X \). The optimal value of \( y \) when there is no storage from year to year is found by equating the first derivative of \( B(y) \) to zero:

\[
\frac{dB(y)}{dy} = 0 = 1 - \int_0^y K f(x \mid \theta) \, dx
\]

\[
= 1 - K F(y),
\]

\[
y = F^{-1}(1/K),
\]

where \( F^{-1}(\cdot) \) is the inverse of the cumulative distribution function. That is the optimal target yield is that amount of water for which the probability of shortage in any year is \( 1/K \).

When there is storage from year to year the optimal value of the target yield is higher. Figure 1 shows the optimal values of target yield plotted against storage capacity for various values of \( K \), assuming the operating value is to release water to the target yield if water is available. The results were obtained by 1000 simulated years of operation. Large amounts of storage enables the surplus water from one year to be used to meet the deficits of another year. This credit should be reflected in the benefit function. The credit can be no bigger than necessary to compensate for expected deficits nor bigger than the expected excess. The benefit function is now:

\[
B(y) = y - \int_0^y K(y, x)f(x \mid \theta) \, dx + \min \left[ \int_0^y K(y-x)f(x \mid \theta) \, dx, \int_0^y K(x-y)f(x \mid \theta) \, dx \right]
\]
OPTIMUM TARGET YIELD AS A FUNCTION OF K
FOR VARIOUS STORAGE CAPACITY ON
THE COLORADO RIVER

Figure 1.
The credit is maximized when
\[ y \int_{y}^{\infty} K(y-x)f(x)dx = \int_{0}^{y} K(x-y)f(x)dx \]
In this case \( B(y) = y \). To find the value of \( y \) that enables the equality above to hold we rewrite the equation as
\[
0 = -\int_{0}^{y} K(y-x)f(x)dx + \int_{0}^{\infty} K(x-y)f(x)dx
\]
\[
= \int_{0}^{\infty} K(x-y)f(x)dx = K(m_{x} - y).
\]

\[ \therefore y = m_{x} \]

With arbitrarily large amounts of storage we can set the target yield equal to the mean flow.

Large amounts of storage were not available on the Colorado, though it was expected that Boulder Dam would be constructed with an effective storage of from 20-30 million acre-feet. The commissioners thought enough storage would be available so that the water to be delivered to the lower basin could be specified in terms of a 10 year running average delivery. What is the optimal value for this delivery? We can test this question in a manner analogous to the development for yearly target yield. The variable \( y \) is the 10 year average target yield, the random variable \( X \) is the 10 year average, and \( f_{10}(x|\theta) \) is the pdf for this random variable. The same description of benefits is used. The optimal decision alternative is \( y = F_{10}^{-1}(1/K) \), that is the optimal 10 year average delivery is that amount for which the probability of the delivery being short is \( 1/K \).
These calculations take care of the natural uncertainty if the distribution \( f(x \mid \theta) \) is specified. Usually an estimate of the mean, the variance and the coefficient of skew is required. What about the sampling uncertainties in these estimates? The specified effect of these uncertainties is to introduce uncertainty into the parameters of the probability distribution function \( f(x \mid \mu, \sigma^2, \gamma) \) and into the decisions derived by use of this function. Bayes Theorem enables the quantification of the sampling uncertainties as a probability distribution function \( g(\mu, \sigma^2, \gamma \mid D) \), conditioned on the sample data. An average \( f(x \mid \cdot, \cdot, \cdot) \) is obtained,

\[
f(x) = \iiint f(x \mid \mu, \sigma^2, \gamma) g(\mu, \sigma^2, \gamma \mid D) \, d\mu \, d\sigma^2 \, d\gamma
\]

and used in the analysis previously developed.

The optimal values calculated, \( y^* \), are optimal in face of the uncertainty; if the true values of the mean, variance and coefficient of skew were known, true optimal values could be calculated, \( y^T \). The difference between \( B(y^*) \) and \( B(y^T) \) measures the cost of the uncertain knowledge used to calculate \( y^* \). This cost is called the opportunity loss and of course cannot be calculated. However, knowledge of the pdf \( g(\mu, \sigma^2, \gamma \mid D) \) describing the uncertainties enables us to calculate the expected opportunity loss (XOL). The XOL is a measure of the worth of perfect information as the XOL would be reduced to zero with perfect information.

Optimal values of \( y \), the expected benefits and the XOL caused by sample uncertainty were calculated and are shown in Table 1. The normal pdf was used to describe the random variation of the yearly river flows: \( f(x \mid \theta) = N(x \mid \bar{x}, s^2) \). The mean and variance were estimated by the sample mean and variance based on the flows from 1900-1921. The sample mean was 17.38 maf, the sample standard deviation was 4.49 maf and the sample coefficient of skew was 0.13. The 10 year average flow is also normal: \( N(x \mid \bar{x}, s^2/10) \). Sample uncertainty is considered for the mean only: \( g(\mu \mid \bar{x}, s^2) = N(\mu \mid \bar{x}, s^2/22) \); uncertainty in the mean is described by a normal distribution. The pdf corresponding to \( f(x) \),
Table 1. Optimal values for Colorado River usage considering uncertainties (based on virgin flow estimates at Lee Ferry, 1900-1921).

<table>
<thead>
<tr>
<th>Uncertainty</th>
<th>K = 5</th>
<th>K = 10</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$y^*$</td>
<td>$B(y^*)$</td>
</tr>
<tr>
<td>(MAF) (Unit Benefit/year/MAF)</td>
<td>(MAF) (Unit Benefit/year/MAF)</td>
<td>(MAF) (Unit Benefit/year/MAF)</td>
</tr>
<tr>
<td>Natural, optimal with no storage</td>
<td>13.80</td>
<td>11.23</td>
</tr>
<tr>
<td>Natural, 10 year average</td>
<td>16.25</td>
<td>15.44</td>
</tr>
<tr>
<td>Natural and Sample (mean)</td>
<td>15.94</td>
<td>14.98</td>
</tr>
<tr>
<td>Sample (mean), uncertainty</td>
<td>---</td>
<td>0.40</td>
</tr>
<tr>
<td>XOL</td>
<td>---</td>
<td>0.40</td>
</tr>
</tbody>
</table>
which describes the natural and sample uncertainty for annual flows is normal: $N(x|\bar{x}, s^2(1+1/22))$ (Raiffa and Schlaifer, 1961, p. 296), for the 10 year average flow it is $N(x|\bar{x}, s^2(1/10+1/22))$. Results are given for $K = 5$ and 10.

Uncertainty in the variance and coefficient of skew were not considered. Uncertainty in the sample variance would yield an $f(x)$ in the form of the t-distribution with 20 degrees of freedom (Benjamin and Cornell, 1971, p. 650) which differs little from the normal. The effect would be to lower the optimal value slightly. The treatment of the sampling uncertainty in the coefficient of skew is a research problem; in the context of our discussion the effect would be to raise the optimal value slightly.

Looking at Table 2, notice how the different values for the penalty ratio for water deficit change the optimal target yield and the XOL. The standard deviation of the sampling uncertainty of the mean flow is 957,272 a.f. yet when it is considered in the decision-making, it only reduces the target yield by about 300-400 thousand a.f. In this case the sampling uncertainty shows in the XOL (expected opportunity loss). The XOL must be thought of in terms of the loss function. An XOL of 400,000 for a penalty ratio of 5 indicates that the expected cost of sampling uncertainty is equivalent to underestimation of 400,000 a.f. or overestimation by 80,000 a.f.

The results indicate that from the stance of 1922, the Colorado River Commissioners made an optimal choice when they allocated 16 million acre-feet per year. The value of XOL indicates that enough data were available to reduce the effect of sampling uncertainty to a tolerable level. That choice was on the high side if one considers that they knew that sooner or later Mexico would have to be provided with 1.5 million acre-feet. This analysis is predicated on the assumption that the normal distribution, with parameters fixed over time, described the random variation in the river flows. Information available now indicates the model and its parameters are not fixed in time.
Table 2. Sensitivity analysis* on return period moments as function of uncertain parameters for rural watershed with only 10 years of data.

<table>
<thead>
<tr>
<th>Uncertain parameters</th>
<th>Var C</th>
<th>Moments of return period $T_Q$</th>
<th>Reciprocal return period</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Mean $\hat{T}_Q$ (years)</td>
<td>Var $\hat{T}_Q$</td>
</tr>
<tr>
<td>only m</td>
<td>0</td>
<td>41.82</td>
<td>538.</td>
</tr>
<tr>
<td></td>
<td>.0005</td>
<td>39.00</td>
<td>442.</td>
</tr>
<tr>
<td></td>
<td>.005</td>
<td>26.33</td>
<td>153.</td>
</tr>
<tr>
<td></td>
<td>.05</td>
<td>6.30</td>
<td>2.35</td>
</tr>
<tr>
<td>only u</td>
<td>.0005</td>
<td>35.36</td>
<td>.830</td>
</tr>
<tr>
<td></td>
<td>.005</td>
<td>23.41</td>
<td>3.89</td>
</tr>
<tr>
<td></td>
<td>.05</td>
<td>6.20</td>
<td>.19</td>
</tr>
<tr>
<td>only m</td>
<td>.0005</td>
<td>37.61</td>
<td>383.</td>
</tr>
<tr>
<td></td>
<td>.005</td>
<td>24.14</td>
<td>110.</td>
</tr>
<tr>
<td></td>
<td>.05</td>
<td>6.52</td>
<td>1.96</td>
</tr>
</tbody>
</table>

* Conditions for the analysis: $A = 0.11$ inches, mean $C = 0.3$ for beta distribution, $Q = 0.7$ inches on the average; rainfall is distributed on basis of an exponential distribution for amounts above 0.3 inches with an average of 14.0 storms/season and an average of 0.39 inches/event.

** Coefficient of variation of $\hat{T}_Q$. 

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Application

The expected loss (gain), considering the natural uncertainty in the random variable $X$ is called risk and depends on the decision alternative chosen, $a$, and the parameter(s), $\theta$, of the underlying probability distribution $f(x \mid \theta)$, and a loss function $L(a, x)$, depending on the alternative chosen and the realization of the random variable:

$$r(a, \theta) = \int L(a, x) f(x \mid \theta) \, dx \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

When the sampling uncertainty in the parameter $\theta$ is considered the expected loss (gain) is called the Bayes Risk:

$$R(a, \theta) = \int r(a, \theta) g(\theta \mid s) \, d\theta \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

where $g(\theta \mid s)$ is the probability distribution describing the sampling error given sample information $s$. The alternative, $a^*$, is chosen to optimize the value of the Bayes Risk. Equations 1 and 2 may be written together

$$R(a, \theta) = \int \int L(a, x) f(x \mid \theta) \, g(\theta) \, dx \, d\theta$$

$$= \int L(a, x) f(x) \, dx \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

where $f(x)$ is the Bayes distribution of the random variable $X$; the Bayes distribution includes both the natural and sampling uncertainties. Furthermore is often do to give the worth of perfect information and the worth of addition information.

Regionalization

Logarithmic regression of basin parameters is used to provide streamflow information (Benson and Matalas, 1967). The streamflow information is often in the form of $T$-year flows and is used to design
small structures such as culverts. Culverts are not now designed by
decision theoretic methods but the application of similar methods has
been recommended (Young et al., 1974). Regression estimates are
estimates, they represent the mean of a distribution of possible values,
hence the calculation of the standard error of estimate and prediction
intervals. The use of regression estimates can be examined from the
viewpoint of Eq. (3). The Bayes distribution for this case is the t-
distribution, the same as used to determine the prediction interval.
The loss function is not generally known. The alternative chosen is
determined by the regression estimate; this estimate is the median and
the mean of the Bayes distribution f(x). Using the regression estimate
as the alternative chosen would be optimal where the loss for over esti-
mation is the same as the loss for equal under estimation. The median
is optimal when the error is measured in terms of the absolute value
of the linear difference, the mean is optimal when the error is measured
as the square of the difference.

The regression estimate is the estimate for the log of the informa-
tion desired (T-year flow). It is, however, the antilog of this value
that is used for design. f(x) is now the log t-distribution and the mean
is no longer the median as the distribution is skewed to the right. If
losses are a function of differences in flow rather than differences in the
log of the flow, the antilog of the regression estimate is now optimal for
losses proportional to the absolute value of the linear error but not for
losses proportional to the square of the error. In cases where the stand-
ard error of estimate is small, the nature of the loss function is not too
important; however if the standard error of estimate is not small the
nature of the loss function does become important.

The probability density function g(θ | s) used in Eq. (2) represents
the distribution of θ after the receipt of information concerning θ, which
usually is in the form of a sampled outcome of the random variable X.
It is calculated by the use of Bayes Theorem,
\[ g(\theta \mid s) = \frac{g(\theta) f(s \mid \theta)}{K} \]  

where \( x \) is the data collected, \( K \) is a normalization constant and \( g(\theta) \) is the prior distribution of \( \theta \), that is the distribution describing the uncertainty in \( \theta \) prior to using the sample data. Usually \( g(\theta) \) is chosen to represent complete ignorance. Vicens et al. (1974, 1975) use regression methods to obtain a more informed prior distribution by the use of regional information. They use as an example, for the parameters \( \theta \), the mean and the variance of the annual flow of the Pemigewasset River at Plymouth, New Hampshire. Physiographic and meteorologic information such as basin area slope, precipitation, etc. were used in the regression equation which was developed from river data throughout New England. The regional information was shown to be the equivalent of many years of river flow data.

Regression methods may be used directly to augment a short record from a related longer record. If the correlation between the two sources of data is not sufficiently high there may be no advantage to the augmentation procedure (Jacobs and Matalas, 1964). However this limitation is not present if the uncertainty, represented by the low correlation, is quantitatively considered and handled in a Bayesian manner. Peterson et al. (1974), in an example concerning the depth to drive bridge piers, show that rainfall data has about 50 percent the value of peak river flow data even though the correlation between the two is too low to use the standard augmentation procedures.

Regression methods are not necessary to obtain a more informative prior distribution. Regional information may be encoded directly into the pdf \( g(\theta) \). Lenton et al. (1974) does this in estimating the parameter \( \rho \), the first-order autoregression coefficient used in some stream flow simulation models. \( g(\theta) \) was constructed by smoothing a histogram constructed from coefficients obtained from over 140 rivers in the region; in this case the region was the world. They obtained estimates based on several different loss functions. Comparisons with traditional methods of estimation show the range of Bayesian superiority to be less
than 40 percent of the possible range for $p$, the range of superiority was from -0.2 to 0.6. Since the first-order correlation coefficient for most rivers is also in this range, this presents no problem but is an advantage.

Rainfall data may be used to obtain information about peak annual runoff. Such information may be necessary for the design of structures on small watersheds. Advantages come from the larger data base using an event based rainfall model, but the necessity of using a rainfall runoff relation may prove to be a disadvantage. In the arid regions of Southern Arizona most runoff at the lower elevations is from summer convective storms. Return periods for the yearly maximal rainfall and runoffs can be calculated given appropriate models for storm frequency and depth. Sampling uncertainty is present in estimating the parameters of the rainfall model and some sample and natural uncertainty in the parameters of the runoff model. Davis et al. (1973) calculated the runoff volume by the relationship:

$$Q = C(R - A) \quad 0 \leq C \leq 1$$

where $R$ is rainfall per storm, $A$ is the abstraction depending on the watershed and $C$ is a coefficient depending on rainfall characteristics such as the maximum 15-minut intensity. The return period of maximum season runoff is a function of number of storms per season, $m$, and the average rainfall per storm, $u$. The parameters $m$ and $u$ are subject to sampling uncertainty, which is reduced with long historical records. The sample uncertainty leads to uncertainty in the return period. The posterior distributions of the return period for 0.7 inch of runoff for historical records of 10 and 20 years is shown in Figure 2.

The coefficient $C$ is a random variable which depends on the particular storm. It introduces a natural uncertainty into the calculation. The effect of this uncertainty is to drastically change the mean of the posterior distribution for the return period of maximum seasonal runoff. Table 2 illustrates the effects of these uncertainties.
Figure 2. Posterior probability density function of return periods for 0.7-inch runoff as function of record length.
Bayesian Decision Theory is used for decision-making and decision evaluation when there is uncertainty in the design process. In river basin studies it has great potential in the area of the regionalization of information, both in conceptualization and efficiency.

Bayesian Decision Theory is a method of handling statistical analysis in relation to the problem or project at hand. If a "best" estimate is required it produces one that is best for the use at hand, it need not be best for all the readers of the Imperial Mathematical Statistics Quarterly. This can be a disadvantage if one cannot produce a loss function that indeed does define what is best.

BDT is not easy to work with. Loss functions may not be known in the first stages of an investigation. Bayesian mathematics can get complicated; closed form solutions are the exceptions used to illustrate textbooks and conference papers. Large amounts of computer time can be consumed for medium sized problems. These disadvantages may serve to limit the use of BDT for small projects and problems. For problems and projects of consequence, the information provided by BDT will justify the effort required for analysis.

Acknowledgment

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References


APPLICATION OF THE "SSAM" MODEL
TO THE GREEN RIVER, UTAH

by

William J. Grenney and Donald B. Porcella*

Introduction

As indicated by most of the other papers presented at this seminar, salinity is the most significant basin-wide water quality problem in the Colorado River System. However, in local situations a variety of parameters are of concern because of their potential to create public health hazards or to degrade the aquatic environment. Figure 1 summarizes the sources, types, effects and controls of pollutants which may cause localized problems. Mathematical modeling is an important methodology for evaluating these complex interactions.

In this study a mathematical water quality model was applied to the Green River and its major tributaries between Jensen and Green River City, Utah (Figure 2). The model was calibrated to water quality conditions occurring during the summer of 1973. Changes in this base condition were predicted by the model for population projections to the year 2000 and for the implementation of proposed waste effluent standards.

The Study Area

The study area is composed of six subbasins: (1) the Green River reach extends from Jensen to Green River City, Utah and can be considered in two separate sections with demarcation occurring at Ouray, Utah between the junctions of the Duchesne and the White Rivers; (2) Ashley Creek is an identifiable subbasin; (3) the Duchesne River has been subdivided into two subbasins at Duchesne, one of

* Utah State University, Logan, Utah.
Figure 1. Sources, types, effects, and controls of pollutants from societal activities in the river basin.
Figure 2. Map of study area.
which includes the upper reaches of the Duchesne River and the Straw-
berry River and the other of which contains the lower reach of the
Duchesne and the Uinta River; (4) the Price Rivers and the White River
are identifiable subbasins.

Hydrology in the study area was determined from 303e studies,
USGS Gaging Station records and from a report by Hyatt et al. (1970).
EPA permit system (NPDES) data were used to identify point
sources. These were compared with population data, State 303e reports,
and other state data to insure completeness. One important data lack
was the adequacy of discharge flow and quality data. Operation of
small treatment plants is spotty and information often is not obtained;
thus spot estimates are often the only available data and longterm or
cumulative values cannot be used.

Those dischargers which actually discharge to streams in the study
area are listed in Table 1; other communities and industries are listed else-
where (UWRL, 1975b). No agricultural wastewater sources are identifi-
able point agricultural sources in the study area. No permits have been
issued in the Colorado River Basin so far.

In analyzing the significant point sources, it was necessary to make
some judgments about effluent quantity and loads. Consequently, six sites
in the study area were defined as having a high enough wastewater quan-
tity or potential quantity to be dealt with in detail. These are the munici-
palities of Vernal, Duchesne, Price, and Bonanza (projected wastewater
source), Utah, and Rangely and Meeker, Colorado. Industrial and
energy developmental uses of water will likely not have an impact on
water quality except insofar as flow and dilution are decreased as a
result of diversion. This is because most industry is using no discharge
as a goal to be achieved, i.e., complete containment of wastes.

Irrigation return flows reenter rivers in this region primarily
as groundwater inflows. It is often difficult to identify agricultural or
natural components of groundwater inflows.
Table 1. Point sources affecting Green River study area (source: NPDES).

<table>
<thead>
<tr>
<th>Point Loads (NPDES)</th>
<th>OBERS Subregion</th>
<th>Flow (MGD)</th>
<th>BOD</th>
<th>COD</th>
<th>SS</th>
<th>Coliforms</th>
<th>Remarks (RM = river mile)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Industrial</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Moon Lake Electric (C00000302)</td>
<td>1402</td>
<td>0.025</td>
<td>5</td>
<td>-</td>
<td>394</td>
<td></td>
<td>Moon Lake Electric Assoc., Inc., Rio Blanco County, Colo. Power generation--overflow from</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>cooling tower, backwashing filters, softeners, White River. No residuals.</td>
</tr>
<tr>
<td>Utah Power and Light (UT-0000096-001, 002, 003)</td>
<td>1403</td>
<td>NA</td>
<td>-</td>
<td>-</td>
<td>220</td>
<td></td>
<td>Carbon Plant at Castle Gate--Willow Creek and Price River, Carbon County, UT. Cooling</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.005</td>
<td>-</td>
<td>-</td>
<td>105</td>
<td></td>
<td>tower slowdown, chemicals, clarifier, evaporation, two settling ponds. Residuals to</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.018</td>
<td>-</td>
<td>-</td>
<td>265</td>
<td></td>
<td>landfill (PRM104)</td>
</tr>
<tr>
<td>Whiterocks Fish Hatchery (UT0000191)</td>
<td></td>
<td>3.554</td>
<td>1.3</td>
<td>1.5</td>
<td>18.9</td>
<td></td>
<td>Utah Division of Wildlife, Uintah County, UT. Uinta River, no residuals. (URM25)</td>
</tr>
<tr>
<td>Polumbus Corp. (UT0000183)</td>
<td></td>
<td>0.1</td>
<td>37</td>
<td>13</td>
<td>2</td>
<td></td>
<td>Near Vernal, Ashley Valley oil field, sludge ponds, and lagoon. Ditch to Ashley Creek, no</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>residuals (ACRM12)</td>
</tr>
<tr>
<td>Carbon/Emery Animal By-Products (UT0000043)</td>
<td>1403</td>
<td>0.0008</td>
<td>1070</td>
<td>1300</td>
<td>1850</td>
<td></td>
<td>Drunkards Wash and then to Price River. No residuals (PRM50.0)</td>
</tr>
<tr>
<td>Maryland Air Products (UT0000078)</td>
<td></td>
<td>0.2</td>
<td>4.5-12</td>
<td>-</td>
<td>-</td>
<td></td>
<td>Wellington, Flood wash to Price River. (PRM78.15)</td>
</tr>
<tr>
<td>Major Oil Corp. (UT0022527)</td>
<td></td>
<td>0.015</td>
<td>-</td>
<td>-</td>
<td></td>
<td></td>
<td>Roosevelt, North branch of Dry Gulch. Septic tank and evaporation pond. (URM5.99)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.005 total</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Clinton Oil Co. (UT0000124)</td>
<td>1403</td>
<td>2.46</td>
<td>15-50</td>
<td></td>
<td>-</td>
<td></td>
<td>Vernal, Ashley Creek, sludge ponds, oil well. (ACRM12)</td>
</tr>
<tr>
<td>R. Lacy Inc. (UT0021768)</td>
<td>1403</td>
<td>0.3</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td></td>
<td>Vernal, Ashley Creek, oil well. (ACRM12)</td>
</tr>
<tr>
<td>Hollandsworth &amp; Travis Operating Co.</td>
<td>1403</td>
<td>0.084</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td></td>
<td>Vernal, ditch to Ashley Creek, oil well. (ACRM12)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Municipal</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Maeker Sanitary Dist. (C00026972)</td>
<td>1402</td>
<td>0.27</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td></td>
<td>Rio Blanco, Colo. White River, 2 cell extended aeration. (WRM160)</td>
</tr>
<tr>
<td>Rangely Sanitary District (C00026972)</td>
<td>1402</td>
<td>0.05-.099</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td></td>
<td>Rio Blanco, Colo. White River. Secondary 2 stage aerated lagoon. (WRM82)</td>
</tr>
<tr>
<td>Vernal City (UT0020028)</td>
<td>1403</td>
<td>1.0-4.9</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td></td>
<td>Ashlley Creek, trickling filter, includes two industrial dischargers. (ACRM11.4)</td>
</tr>
<tr>
<td>Duchessa City Corp. (UT0020093)</td>
<td>1403</td>
<td>0.15-.275</td>
<td>1.2-20</td>
<td>-</td>
<td>5-4.5</td>
<td>4-43000</td>
<td>Lagoons, Duchesnea River. (DRM64.8)</td>
</tr>
<tr>
<td>Roosevelt City Sewer Dept. (UT0020320)</td>
<td>1403</td>
<td>0.05-.099</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td></td>
<td>Dry Gulch Creek lagoon (6 ponds) to Uinta River. (URM5.9)</td>
</tr>
<tr>
<td>Price River Water Improvement Dist.</td>
<td>1403</td>
<td>1.6-1.2</td>
<td>22-20</td>
<td>15-20</td>
<td>4300</td>
<td></td>
<td>Wellington, trickling filter, to Price River. (PRM78.15)</td>
</tr>
</tbody>
</table>
Feedlots are small and so far have not been under the permit system. Grazing is common but is a diffuse source of BOD, coliform and nutrients.

Combined sewer overflows do not exist in the study reach. Septic tank (as are used in the small communities) contributions to water quality problems probably are unimportant if not unmeasurable (e.g., see Meyers et al., 1972). Agriculture nonpoint sources would include overland flow where it occurs; data from Hyatt et al. (1970) indicate this is unlikely. Groundwater flow is most likely the return flow mechanism for much of irrigated agriculture in the study area. Small feedlots and grazing apparently were responsible for most of the observed coliform problems (State of Utah, 1974). Control of trailer dumpout wastes, garbage and refuse disposal (including dead animals), proper wastewater treatment plant operation, and sewage in small communities would eliminate some of the observed problems (State of Utah, 1974).

**Water Quality Model**

**Introduction**

A river water quality model, "SSAM" (Stream Simulation and Assessment Model), was developed at UWRL for use in water quality management studies. The model has been successfully applied to the Weber, Bear, Virgin, and Sevier Rivers in Utah (for example; Utah Water Research Laboratory, 1974, and Grenney et al., 1975). The model was used in wasteload allocation studies to evaluate various management alternatives for dealing with projected wasteloads during periods of low river flow (7-day 10-year low flows).

The model, SSAM, can simulate nine constituents simultaneously: two conservative constituents (for example, salinity); a nonconservative substance; coliform bacteria (MPN); ammonium (NH$_4$); nitrate (NO$_3$);
biochemical oxygen demand (BOD); and dissolved oxygen (DO). User options are available to run any combination of constituents. Reaeration and biological rate coefficients in the model are automatically adjusted for temperature. Saturation concentrations of dissolved oxygen are calculated as a function of temperature and elevation. The model includes the following sources for constituent input: (1) headwater flow, (2) diffuse surface runoff, (3) diffuse subsurface runoff, (4) point loads, and (5) leaching from bottom deposits. Provisions for point diversions and stream flow to groundwater are included in the model. Biochemical interactions are represented by first order, linear differential equations and advection is modeled for conditions of steady flow.

The solution technique for SSAM provides exact solutions for the system of differential equations and, therefore, eliminates numerical errors from the model responses. The model was developed with the user in mind and provides convenient input formats to facilitate both calibration and management runs.

Model equations

Models of the first-order of resolution, such as the Utah Steady-State River Model (USSRM), have been most popular for practical applications. USSRM can be applied to a river system with any reasonable number of tributaries, point loads, and point diversions. The river channels must be divided into "reaches" representing lengths of river which can be assumed to have uniform physical characteristics. The program was designed for ease of user operation; for example, the system layout may be changed at any time (e.g., a point load or diversion added, a new reach defined) simply by inserting the appropriate punched card into the data deck.

Basically, the model simulates the reactions and interactions among constituents occurring in a slug of water (see Figure 3) as it
Figure 3. Model conceptualization of a slug of water moving downstream.
travels downstream at a velocity, $\bar{V}$. It is assumed that mixing with adjacent slugs (dispersion) is negligible. Mass can be added to the slug by lateral inflow and by leaching from the stream bottom. Oxygen can enter the slug by diffusion across the air-water interface and by the photosynthetic oxygen production of benthic and planktonic algae.

The model starts at the first headwater, where water quality constituent concentrations are known. These concentrations provide the initial conditions for the differential equations describing the system and are used in conjunction with the river characteristics for the downstream reach to obtain a closed-form solution to the differential equations. Then the concentrations can be calculated at any point of interest in the downstream reach. The concentrations at the end of one reach become the initial conditions for the next reach.

Water quality equations

Flow is assumed to be steady (invariant with time). The water quality equations represent two phenomena occurring in a slug of water as it travels downstream (Figure 3):

1. Mass being added or removed from the water due to sources or sinks distributed along the stream channel.
2. Biochemical reactions and interactions among constituents.

Descriptions of the symbols used in the equations are shown in Table 2.

The expression for rate change in constituent concentration due to lateral surface and subsurface flow can be expressed as follows:

$$\frac{dC_i}{dt} = \begin{cases} 
\frac{-C_i(Q_S + Q_G)/A + (Q_S C_{Si} + Q_G C_{Gi})/A}{S_i} & \text{when subsurface flow is into the stream} \quad (1a) \\
\frac{-C_i Q_S/A + Q_S C_{Si}/A}{S_i} & \text{when subsurface flow is out of the stream} \quad (1b)
\end{cases}$$
Table 2. Description of symbols used in the water quality equations.

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>B_A</td>
<td>Benthic load for NH4N</td>
<td>(mg/sq m/sec)</td>
</tr>
<tr>
<td>B_B</td>
<td>Benthic load for CBOD</td>
<td>(mg/sq m/sec)</td>
</tr>
<tr>
<td>B_N</td>
<td>Benthic load for NO3N</td>
<td>(mg/sq m/sec)</td>
</tr>
<tr>
<td>B_O</td>
<td>Oxygen production from the benthic algae for DOXY</td>
<td>(mg/sq m/sec)</td>
</tr>
<tr>
<td>B_P</td>
<td>Benthic load for PHOS</td>
<td>(mg/sq m/sec)</td>
</tr>
<tr>
<td>B_U</td>
<td>Oxygen uptake by the benthic BOD for DOXY</td>
<td>(mg/sq m/sec/mg/l oxygen)</td>
</tr>
<tr>
<td>CBOD</td>
<td>Carbonaceous biochemical oxygen demand</td>
<td>(mg/l)</td>
</tr>
<tr>
<td>C_{Gi}</td>
<td>Concentration of the i^{th} constituent in the lateral groundwater inflow</td>
<td>(mg/l)</td>
</tr>
<tr>
<td>C_i</td>
<td>Concentration of the i^{th} constituent</td>
<td>(mg/l)</td>
</tr>
<tr>
<td>COLI</td>
<td>Coliform bacteria</td>
<td>(MPN/100 ml)</td>
</tr>
<tr>
<td>CON1</td>
<td>Conservative constituent</td>
<td>(mg/l)</td>
</tr>
<tr>
<td>CON2</td>
<td>Conservative constituent</td>
<td>(mg/l)</td>
</tr>
<tr>
<td>C_{Si}</td>
<td>Concentration of the i^{th} constituent in the lateral surface inflow</td>
<td>(mg/l)</td>
</tr>
<tr>
<td>DOXY</td>
<td>Dissolved oxygen</td>
<td>(mg/l)</td>
</tr>
<tr>
<td>E_{sat}</td>
<td>Dissolved oxygen saturation concentration</td>
<td>(mg/l)</td>
</tr>
<tr>
<td>E_L</td>
<td>Elevation of element</td>
<td>(m)</td>
</tr>
<tr>
<td>K_2</td>
<td>Reaeration rate at 20°C for DOXY</td>
<td>(per sec)</td>
</tr>
<tr>
<td>K_3</td>
<td>First-order decay coefficient for NCON</td>
<td>(per sec)</td>
</tr>
<tr>
<td>K_4</td>
<td>Removal rate at 20°C for COLI</td>
<td>(per sec)</td>
</tr>
<tr>
<td>K_5</td>
<td>Removal rate for PHOS</td>
<td>(per sec)</td>
</tr>
</tbody>
</table>
Table 2. Continued.

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>$K_6$</td>
<td>Decay rate at $20^\circ C$ for NH$_4$N</td>
<td>(per sec)</td>
</tr>
<tr>
<td>$K_{A6}$</td>
<td>Removal rate (other than biochemical decay) for NH$_4$N</td>
<td>(per sec)</td>
</tr>
<tr>
<td>$K_{A7}$</td>
<td>Removal rate for NO$_3$N</td>
<td>(per sec)</td>
</tr>
<tr>
<td>$K_8$</td>
<td>Decay rate at $20^\circ C$ for CBOD</td>
<td>(per sec)</td>
</tr>
<tr>
<td>$K_{A8}$</td>
<td>Removal rate (other than biochemical decay) for CBOD</td>
<td>(per sec)</td>
</tr>
<tr>
<td>NCON</td>
<td>Nonconservative constituent</td>
<td>(mg/l)</td>
</tr>
<tr>
<td>NH$_4$N</td>
<td>Ammonium</td>
<td>(mg/l)</td>
</tr>
<tr>
<td>NO$_3$N</td>
<td>Nitrate</td>
<td>(mg/l)</td>
</tr>
<tr>
<td>PHOS</td>
<td>Available phosphorus</td>
<td>(mg/l)</td>
</tr>
<tr>
<td>$F_r$</td>
<td>Net photosynthetic oxygen production by phytoplankton</td>
<td>(mg/l/sec)</td>
</tr>
<tr>
<td>$Q_G$</td>
<td>Lateral groundwater inflow</td>
<td>(cu m/s/m)</td>
</tr>
<tr>
<td>$Q_S$</td>
<td>Lateral surface inflow</td>
<td>(cu m/s/m)</td>
</tr>
<tr>
<td>$R$</td>
<td>Hydraulic radius</td>
<td>(m)</td>
</tr>
<tr>
<td>$S_i$</td>
<td>Source (or sink) for $i^{th}$ constituent due to lateral inflow</td>
<td>(mg/l/sec)</td>
</tr>
<tr>
<td>$t$</td>
<td>Time</td>
<td>(sec)</td>
</tr>
<tr>
<td>$T$</td>
<td>Temperature</td>
<td>($^\circ$C)</td>
</tr>
<tr>
<td>TDS</td>
<td>Total dissolved solids</td>
<td>(mg/l)</td>
</tr>
<tr>
<td>$T_f$</td>
<td>Temperature</td>
<td>($^\circ F$)</td>
</tr>
</tbody>
</table>
where

\[ Q_s = \text{lateral surface flow (cm/m)} \]

\[ Q_G = \text{lateral subsurface flow (cm/m)} \]

\[ A = \text{average cross-sectional area (sq m)} \]

The total rate changes in the various constituent concentrations in the main channel are expressed by the following system of equations.

**Constituents 1 and 2. Conservative constituents (CON1 and CON2).**

The rate change in concentration is influenced only by mass input from lateral inflow.

\[
\frac{dC_i}{dt} = S_i, \quad i = 1, 2
\]

**Constituent 3. Nonconservative constituent (NCON)**

The rate change in concentration is influenced by first-order decay and by mass input from lateral inflow.

\[
\frac{dC_3}{dt} = -K_3 C_3 + S_3
\]

**Constituent 4. Coliform bacteria (COLI)**

The rate change in concentration, MPN (most probable number per 100 ml), is influenced by first-order decay (death) and by mass input from lateral inflow. The decay rate \( K_{4a} \) increases with temperature.

\[
\frac{dC_4}{dt} = -K_{4a} C_4 + S_4
\]

\[
K_{4a} = K_4 1.047(T-20)
\]

**Constituent 5. Available phosphorus (PHOS)**
The rate change in concentration is influenced by first-order removal (algae uptake, precipitation, etc.), leaching from bottom deposits, and mass input from lateral inflow.

\[
\frac{dC_5}{dt} = -K_5 C_5 + \frac{B_F}{1000 R} + S_5 \quad \ldots \quad \ldots \quad \ldots \quad (5)
\]

Constituent 6. Ammonium (NH₄N)

The rate change in concentration is influenced by first-order decay (biochemical oxidation to nitrate), first-order removal (uptake by algae, etc.), leaching from bottom deposits, and mass input from lateral inflow.

\[
\frac{dC_6}{dt} = -K_{6a} C_6 - K_{A6} C_6 + \frac{B_A}{1000 R} + S_6 \quad \ldots \quad \ldots \quad \ldots \quad (6a)
\]

\[
K_{6a} = K_6 1.047^{(T-20)} \quad \ldots \quad \ldots \quad \ldots \quad (6b)
\]

Constituent 7. Nitrate (NO₃N)

The rate change in concentration is influenced by the accumulation of oxidized ammonia, first-order removal (uptake by algae, etc.), leaching from bottom deposits, and mass input from lateral inflow.

\[
\frac{dC_7}{dt} = K_{6a} C_6 - K_{A7} C_7 + \frac{B_N}{1000 R} + S_7 \quad \ldots \quad \ldots \quad \ldots \quad (7)
\]

Constituent 8. Carbonaceous biochemical oxygen demand (CBOD)

The rate change in concentration is influenced by first-order decay (biochemical oxidation), first-order removal (adsorption, settling), leaching from bottom deposits, and mass input from lateral inflow. CBOD is modeled as the ultimate demand. Five-day BOD can be input to the model and is converted to ultimate BOD by a user-supplied conversion factor (BODCON) as follows: \( BOD_U = BOD_5 \times BODCON \). Output is converted back to five-day BOD to be consistent with the input.
Constituent 9. Dissolved oxygen (DOXY)

The rate change in concentration is influenced by reaeration across the surface, carbonaceous oxygen demand, nitrogenous oxygen demand, photosynthetic production by benthic algae, photosynthetic production by phytoplankton, uptake by bottom deposits, and mass input from lateral inflow.

\[
\frac{dC_9}{dt} = -K_{8a} C_8 - K_{8b} C_8 + \frac{B}{1000 R} + S_8 \quad \ldots \ldots \quad (8a)
\]

\[
K_{8a} = K_8 1.047^{(T-20)} \quad \ldots \ldots \quad (8b)
\]

Solution Technique

The general purpose of this algorithm is to construct the closed-form solution for a system of constant coefficient linear ordinary differential equations which can be solved in sequence.

All possible solution forms to this type of system of equations have been grouped into the five categories shown in Table 3. For
Table 3. Solutions for term by term integration of model equations.

<table>
<thead>
<tr>
<th>Category Number</th>
<th>Differential Equation</th>
<th>Solution depending on values of the coefficients</th>
</tr>
</thead>
<tbody>
<tr>
<td>(1)</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Left hand side</td>
<td>Right hand term</td>
</tr>
<tr>
<td></td>
<td>(2)</td>
<td>(3)</td>
</tr>
<tr>
<td></td>
<td>$\frac{dX}{dt} + \beta_1 X + \beta_2 + \ldots$</td>
<td>$\beta_2 t + C_1$</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>$\frac{dX}{dt} + \beta_1 X + \beta_2 e^{t_2 t} + \ldots$</td>
<td>$\frac{\beta_2}{t_2} e^{t_2 t} + C_1$</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>$\frac{dX}{dt} + \beta_1 X + \beta_2 e^{t_2 t} + \ldots$</td>
<td>$\frac{\beta_2}{t_2} e^{t_2 t} f(t) + C_1$</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>$\frac{dX}{dt} + \beta_1 X + \beta_2 e^{t_2 t} + \ldots$</td>
<td>$C_1 e^{-\beta_1 t}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>$\frac{dX}{dt} + \beta_1 X + \beta_2 e^{t_2 t} + \ldots$</td>
<td>$C_1 e^{-\beta_1 t}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

- $C_1$ is a constant of integration which incorporates the initial conditions of the system.
- $f(t) = \sum_{m=1}^{k+2} \frac{k! t^{m-1} (-1)^{k+m-1}}{(m-1)! \beta_k (k+2-m)}$

Go to Category #2

Go to Category #1

Not Applicable

Not Applicable

Not Applicable

Not Applicable

Not Applicable

Not Applicable

Not Applicable
a particular left-hand-side (column 2) and a particular term on the	right-hand-side (column 3), solutions are shown in columns 4 through
7 depending on the values of the coefficients. The solution for each
of the differential equations can be expressed in the general form:

\[ X_i = \sum_{j=1}^{n_i} \beta_{i,j}^{t,i,j} \xi_{i,j}^{t} \]  \( \ldots \ldots \ldots \ldots \ldots \)  \( \ldots \ldots \ldots \ldots \ldots \) \( (10) \)

where \( i \) identifies the dependent variable, \( n_i \) is the total number of
terms in the solution, and \( \beta, k, \) and \( \xi \) are coefficients.

The algorithm operates on each equation in sequence. The
first equation in the system is expressed in the form
\[ \dot{X}_1 + G_{1,1} X_1 = G_{1,2} \]
where the dot indicates the time derivative and values for the
\( G \)'s are constant coefficients. The proper solution is selected from
Table 3 and values of \( \beta, k, \) and \( \xi \) are calculated and stored for each
term in the solution. The solution to the first equation is then sub­
stituted into the second equation resulting in the expression:

\[ \dot{X}_2 + G_{2,1} X_2 = G_{2,2} \left( \sum_{j=1}^{n_1} \beta_{1,j}^{t,1,j} \xi_{1,j}^{t} \right) + G_{2,3} \]  \( \ldots \ldots \ldots \ldots \ldots \)  \( \ldots \ldots \ldots \ldots \ldots \) \( (11) \)

This equation is then solved by superposing the solution (as shown in
Table 3) for each term on the right-hand-side. Thus, each equation
is operated on in sequence, first involving the substitution of appro­
priate preceding solutions and then conducting a term by term integra­
tion.

The algorithm is basically one of accounting for all of the terms
in a particular differential equation, identifying its form, and select­
ing the proper solution from a table. If a term becomes zero, it is
dropped from the equation and eliminated from future calculations.
Once a closed-form solution has been constructed by the algorithm,
it can be used to calculate values for the dependent variables in
later calculations. This type of approach is much more efficient
than using a numerical technique and avoids distortions which may
be significant in numerical approximations.
Typical model responses

A testing of the response of the model was made using a uniform stream and streamflow with three waste point load inputs and a tributary (Figure 4). Loadings of $\text{BOD}_5$, $\text{NH}_4^-\text{N}$, and $\text{DO}$ were used in conjunction with varying parameters associated with processes of oxygen metabolism and moss transfer. The factors studied were $K2A$ (reaeration coefficient, $\text{days}^{-1}$), $PR$ (photosynthetic coefficient, $\text{mg/l-1day}^{-1}$), $K9A$ (CBOD assimilation coefficient, $\text{day}^{-1}$), $BO$ (benthic oxygen input from benthic plants, $\text{mg m}^{-2}\text{day}^{-1}$), and $BB$ (benthic oxygen demand, $\text{g m}^{-2}\text{day}^{-1}$). Values of $K2A$ were 1.0 for every run of the single factors, where the single factors had a zero value or a specific rate value (see Figure 5). Then a minimal sensitivity study was done for $K2A$ (where $K2A = 0.1, 1.0, 5.0 \text{days}^{-1}$) and using all other factors at the specific rate value as used in the single factor studies. These results are shown in Figure 5 where $\text{DO}$ variations are shown to respond sensitively and logically.

Model Application

Model input data

A schematic of the hydrologic system studied (Figure 6) shows several basic sources of water: headwaters (H), loads (streams and point sources, L), checkpoints (C) to compare results with observed data, and junctions (J) between tributaries and terminations of the two major study reaches (T). The 1973 flows were determined for each of these points, a flow balance made, and then the calibration of the model performed. Hydrologic data and effluent quality data were obtained from USGS reports and the 303e reports.
Figure 4. Schematic of unit response study of SSAM.
Figure 5. Unit response output of SSAM for dissolved oxygen.
Figure 6. Schematic representation of water quality model (SAAM) of study area.
The effluent quality data are described in Table 4 for the various alternative futures. The first grouping in Table 4 is for existing effluent flows and qualities as measured in 1973 303e studies (State of Utah, 1974; State of Colorado, 1974). Parameters for Bonanza were considered zero because at the present time there is no discharge into the actual drainage system (White River). Where data were not available, estimates were made. Estimates were based on McGauhey and Middlebrooks (1972) and were as follows: \( \text{NH}_4 - N = 10 \text{ mg/l} \), \( \text{PO}_4 - P = 10 \text{ mg/l} \), other parameters judged on basis of surrounding water quality.

The next groupings were for the years specified by PL 92-500 (1977, 1983, 1985) and a future benchmark year (2000). In 1977 the basis of State of Utah standards, BPT, is defined as 25 mg/l \( \text{BOD}_5 \), SS, and 2000 MPN/100 ml coliform. Utah specifies that in 1980 an effluent standard appropriate to BAT will be defined; to relate to Colorado it is assumed that those 1980 standards will still be in effect in 1983. In 1985 two alternative EOD conditions were considered: (I) zero discharge of pollutants and (II) zero discharge of flow and pollutants. EOD(I) effluent levels were estimated by assuming that \( \text{BOD}_5 \), \( \text{NH}_4 - N \), coliform, and SS were zero and that other parameters were equal to influent to the water system, i.e., the water quality at an upstream station for TDS, \( \text{Cl}^- \), \( \text{PO}_4 - P \), \( \text{NO}_3 - N \), and DO. EOD(II) was simply a matter of flow adjustment to zero so that no effluent entered the stream.

Effluent flow estimates as affected by population increase were obtained by assuming (1) 90 percent of the population increase in an OBERS subregion would enter the larger communities, (2) the population increase in a subregion could be determined for a specific county by determining the ratio of the county population to the subregion population, and (3) that the population increase in the target cities (Table 4) would be the cities in the subregion to increase in population. Bonanza was the exception to these sets of assumptions; it was
Table 4. Effluent flow and quality of major wastewater treatment plants in the Green River reach study area.

<table>
<thead>
<tr>
<th>WWTP</th>
<th>Year</th>
<th>Flow (CFS)</th>
<th>TDS</th>
<th>CI'</th>
<th>SS</th>
<th>COLI</th>
<th>NH3-N</th>
<th>PO4-P</th>
<th>NO3-N</th>
<th>CBOD</th>
<th>DO</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vernal</td>
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* Analytical data Reference

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<td>White River 160</td>
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<td>White RM 51.2 (actually Coyote Wash)</td>
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No data; values were estimated.
assumed that the population of Bonanza would increase because of oil shale development and would attain 8000 people in 2000, all of which population would increase during 1985 to 2000. These assumptions can be stated in equation form as follows:

\[
\text{(Projection - 1970 population)} \cdot (0.9) - \frac{\text{(county population, 1970)}}{\text{(subregion population, 1970)}} + \text{urban population in county, 1970} = \text{projected urban population}
\]

Because no population increase occurs under OBERS E projections and because energy impacts on population projections are minimal until 1983 (Table 5), only population increases for 1983 and 2000 were studied. The population increase was apportioned to the towns on the basis of population (Table 5). Effluent flows were calculated based on an additional 100 gpd per capita. This last assumption is unrealistic in terms of likely technology for the study area (lagoons) but errs conservatively.

Model calibrations

As can be seen in Figures 7, 8, 9, and 10, the calibration between predicted and true values for DO, CBOD, coliforms, and suspended solids was adequate. Considerable difficulty arose in calibration due to the few check points available (9 in the basin during August, the critical flow period), the lack of good data on the water quality parameters and the relative inadequacy of flow data. However, based on experience with streams in the study area and elsewhere in the intermountain region it is felt that the data and the model as developed represent a reasonable estimate of the water interactions in the study area.
Table 5. Increases in population for target towns.

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592
Figure 7. Comparison between predicted and measured DO concentrations.
Figure 8. Comparison between predicted and measured BOD concentrations.
Figure 9. Comparison between predicted and measured coliform concentrations.
Figure 10. Comparison between predicted and measured suspended suspended solid (SS) concentration.
Management Alternatives

In the study area it was assumed that the restrictions of effluents by state laws would supercede the requirements of PL 92-500 for WWTP effluents. In a practical sense Utah's effluent standards are utilized because they are the most stringent in the study area so far. Thus, the deadlines of 1977 (BPT), 1983 (BAT), 1985 (EOD) would be applied to effluents of the six communities described previously; in addition, these same treatment levels would be applied to effluents from the communities after adjustment of populations for the year 2000.

Projected Effects of Quality PL 92-500 on Water Quality

In obtaining those results, the calibrated model was utilized and changes in flow and loadings for the six communities were made as shown previously in Table 4. Values are plotted for different alternate futures only for the first downstream checkpoint because the effluent loads most likely have greatest impact at that point. The alternate futures shown are 1977 (OBERS, BPT), 1983 (OBERS, BAT), 1983 (high energy, BAT), 1985 (EOD), 2000 (high energy, BPT), 2000 (high energy, BAT). Other combinations would have essentially the same effect as these alternate futures.

Oxygen

As can be seen, no significant impacts from the municipal effluents on dissolved oxygen (DO) could be observed. Essentially

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1 BPT is Best Practicable Treatment and represents 25/25 for \( \text{BOD}_5 \) and suspended solids, 2000 MPN/100 ml for coliform; BAT is Best Available Treatment and represents 10/10 for \( \text{BOD}_5 \) and suspended solids, 200 MPN/100 ml for coliform; EOD is Elimination of Discharge; I represents 0/0/0/0 for \( \text{BOD}_5 \), suspended solids, \( \text{NH}_4^-\text{N} \), coliforms and water intake concentrations for other parameters; II represents zero effluent flow as will likely be practiced wherever possible.
BOD and NH₄ discharges had no measurable effect on DO (Figure 11). Thus, even at high energy development when BOD loads might be expected to increase reaeration was adequate to replace the oxygen used in degrading the BOD before reaching the downstream checkpoint. No violations of stream water quality standards of Utah (6 mg/l) or EPA (5.5 mg/l) were observed.

**BOD**

Essentially no impacts on BOD concentrations were observed from the Duchesne or the White River communities (Figure 12). BOD concentrations below Price are already high and would be decreased by implementation of BPT and BAT (Figure 13). BAT will have an impact downstream of Vernal (Figure 13), also; however, effluent quality of Vernal is already within BPT value. The long range impacts of whether BPT or BAT are utilized indicate a probable BOD problem in 2000 if population increase due to energy development occurs. Vernal discharges into Ashley Creek which has little dilution capacity. Thus, it would be expected that violation of Utah BOD stream standard (5 mg/l) might occur by the year 2000. This shows the need to consider dilution and the possible effects of continued diversions and water depletions in the study area.

**Coliforms**

Significant variation in total coliforms occurred between the different sites (Figure 14). In the Duchesne, White, and Ashley Creek, coliforms are less than Utah stream standards and no significant impacts of the waste effluents could be seen except for BPT in the year 2000 as loading due to population increase caused a slight increase. In the Price, the coliform concentrations exceed standards upstream of the WWTP discharge and the effluent has little relative impact on the stream concentrations. Thus, more chlorination of effluent would have little impact. Upstream activities probably including significant but small feedlots and dairy operations plus the
Figure 11. Effects of treatment with alternate futures on dissolved oxygen concentrations in study area streams.
Figure 12. Effects of treatment with alternate futures on BOD in the Duchesne and White River.
Figure 13. Effects of treatment with alternate futures on BOD in Ashley Creek and the Price River.
Figure 14. Effects of treatment with alternate futures on total coliforms in study area streams.
operation of an animal by-products industry probably are responsible for the violation of the stream standards. NPDES permits were not issued for any wastewater sources in the immediate area except Carbon-Emery By-products into Drunkards Wash.

Suspended solids (SS)

As can be seen in Figure 15, little impact of treatment occurred with respect to SS. This occurs primarily because there is a naturally high level of SS in the study area streams and throughout the Colorado River Basin; the roily turbid waters are one of the aspects for which the river is noted. The White River has the highest natural sediment load (about 230 mg/l), the Price River has less than 75 mg/l SS, while the other streams have less than 30 mg/l SS. No stream standards exist for suspended solids, natural and/or disturbed vegetation areas have the major impacts on stream SS loads; thus, more detailed analysis of this problem needs to be performed than current data allows.

Conclusions

Application of the SSAM model to the Lower Reach of the Green River and its tributaries indicated that Utah State stream standards will be exceeded in very few cases. For high levels of development (i.e., the year 2000 estimates) BOD and coliforms may cause some localized problems. Problems associated with stream flow, land uses, and non-point sources appear to be of more concern than the point loads which receive prescribed levels of treatment.

The model (SSAM) was found to be a useful tool in this application. The model is relatively easy and inexpensive to apply and the resolution of the predicted values are commensurate with the data available for calibration and the objectives of the study.
Figure 15. Effects of treatment with alternate futures on suspended solids in the study area.
References


SOME ASPECTS OF GROUNDWATER SUPPLIES
IN THE UPPER COLORADO RIVER BASIN

by

Orson L. Anderson*

Introduction

If the energy resources of the Upper Colorado River Basin are to be exploited in sufficient amounts to relieve the energy bind of the nation, in particular that of the Southwest, then correspondingly large amounts of water in the Upper Colorado River system are required for the energy development, unless the technology of power development is changed.

In the Upper Colorado River Basin, the area containing energy resources (coal, oil shale, oil, natural gas, and uranium) is very sparsely populated (the larger towns are Rock Springs, Wyoming; Grand Junction, Colorado; Price, Utah; and Farmington, New Mexico) (Figure 1). Energy is exported from the Upper Basin into the populated load centers. In the sparely populated areas, energy projects are planned to be coincident with the location of the resources, but these are the very areas in which surface water is scarce. If mine-mouth powerplants are constructed, the water allocated to the state containing the energy resource must be used for the production of the energy, in spite of the fact that the energy often is being produced for consumption in another state. The Law of the River is such that it is extremely unlikely in the short term that institutional arrangements can be made to transfer a water right from a state using the energy to the state in which power is actually produced.

For these and other reasons, it is very important to quantify the amount of water that is in the Colorado River, so that the quantity

*University of California, Los Angeles.

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Figure 1. Upper Colorado River Basin and coal plant sites in Utah
(Sources: Reference 3, page 3; reference 8, Plate 1; New York Times, August 3, 1975, page 34.)
apportioned to each state can be estimated as accurately as possible. Any proposed power project must have relatively secure water rights in order for financing to be obtained, yet the state engineer who awards this water right to the project has to be sure that the amount of water awarded actually is available under conditions of variable water flow and in the context of other demands and rights.

**Upper Basin ground water for power production**

Not all coal-based powerplants in the Upper Basin involving California utilities are based on the use of surface water. In 1974 a power utility known as Intermountain Power Project (IPP) found a large flow of ground water near Caineville, Utah (see Figure 2 for the location). The ground water is in the Henry Mountain Basin. The following is quoted from the IPP booklet (4): *

It is estimated that the nominal 3,000 MW IPP project will require 50,000 acre-feet of water annually for cooling purposes.

To meet these water requirements, in the arid south-central region of Utah, ICPA undertook an aggressive and extensive program in 1971 to obtain an adequate water supply.

Applications were filed in the Caineville area for unappropriated surface and ground water to supply project requirements. The surface water would come from the Fremont River which flows through Caineville.

Favorable results have been obtained in connection with an ICPA test well drilled about 3-1/2 miles northwest of Caineville in Wayne County. During fall and winter 1973, the well was drilled to a depth of 760 feet, thus reaching 92 feet into the main Navajo Sandstone. Test pumping in

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*IIPP = Intermountain Power Project
LADWP = The Los Angeles Department of Water and Power
ICPA = The Intermountain Consumers Power Association
SCE = Southern California Edison*
Figure 2. Utah counties with new coal developments (Source: Reference 1, page 32).
this well in early February 1974 produced 7 cubic feet per second flow (approximately 1,000 acre-feet per year) and thereafter 3.1 cfs artesian flow. These results have prompted ICPA officials to describe this well as a 'major water find - possibly the largest in the State's history' [actually 7 cubic feet per second flowing for a year is about equal to 5,000 acre-feet].

A new IPP test well is currently being planned near the existing ICPA well in the Red Desert [west of Caineville]. This new well is planned to extend approximately 1,700 feet beneath the surface to the bottom of the Navajo Sandstone aquifer. The new well will provide additional data regarding the quantity and quality of the ground water and identify numerous characteristics of the aquifer.

Deep ground water in the Black Mesa Basin north of Flagstaff, Arizona, is presently used by a coal resources development concern. Water from this source is used to transport Black Mesa coal by slurry pipeline to the Mojave coal-fired plant in Nevada, which serves LADWP and SCE. This flow amounts to about 2,300 acre-feet per year.

A photograph of a clay model of the major structural basins of the Upper Colorado River Basin, was constructed from the Tectonic Map of North America (5). The surface of the model represents the elevation relative to sea level of the top of the Dakota sandstone, as taken from the topographic map. Of course, the earth's present surface does not conform to the top of Dakota, as it is deeply buried by younger rocks in many areas and has been eroded away in others. Also, the vertical relief is exaggerated in the model for clarity. The contours of the Dakota sandstone modelled in the photograph show a number of major structural basins in the Upper Colorado River Basin, Within each of these structural basins is a major coalfield, a major oil basin, or a major oil shale deposit. About 1,000 feet stratigraphically below the Dakota sandstone in the basins of southern Utah is the main aquifer, the Navajo sandstone, which was mentioned in the IPP report quoted above.
Figure 3 is a sketch of the structural basins taken from the day models. In Figure 3, major water recharge areas are also shown, for example, the Water Pocket fold and the San Rafael swell which are recharge areas of the Henry Mountain Basin.

The structural basins of the Colorado Plateau contain ground water in sedimentary rocks and fractured volcanic rocks. Discussion in this Bulletin is restricted to this type of ground water reservoir. There are other types of earth materials, however, such as unconsolidated materials (alluvium or gravel), which may serve as natural reservoirs for ground water. Water contained in alluvium within fault basins also is tapped in the Southwest, particularly in the basin and Range Province and while alluvial basins are important sources of water (especially in Arizona, Nevada, and western Utah), they are not available as a major water source within the Colorado Plateau Province which contains most of the Upper Colorado River Basin.

Ground water reservoirs can also be classified in terms of their size (whether they are local or regional in extent). Most aquifers in unconsolidated materials are localized because they are found in alluvial valleys, bounded by fault blocks. On the other hand, ground water reservoirs in bedrock, such as in the Navajo sandstone, can possibly extend tens and hundreds of miles.

Ground water divides, analogous to drainage basin divides on the land surface, separate ground waters of one basin from those of another. Often, ground water divides can be moved by an exceptional extraction of ground water from one side of the divide. Thus, the hydrologic highs will correspond only approximately to the saddle points of basins, demonstrated in Figure 3.

The important deep aquifers of the Colorado Plateau are the Navajo, Entrada, and Wingate sandstones. Of these, the Navajo sandstone forms the best aquifer because of its unusual microscopic structure. It is composed essentially of rounded grains of quartz, cemented by quartz or in some cases calcite (calcium carbonate). On a microscopic scale, Navajo sandstone is porous. The void space of Navajo
Figure 3. Structural basins and recharge areas in the Colorado Plateau Province.
sandstone is typically high, amounting to 15 to 30 percent (25 percent is the typical porosity). The voids are interconnected so that the sandstone is highly permeable (water flows easily through it).

Most of the basins in Figure 3 could conceivably contain large ground water reservoirs, because the sedimentary beds surrounding each basin dip inward toward the basin center. Thus, water entering the rocks at the basin margins presumably percolates slowly down the dip of the sedimentary formations toward the bottom of the structural basin. Aquifers in the center of basins have been filled slowly over geologic time.

In many cases, these aquifers lie at rather large depths (from 2,000 to 5,000 feet) below the surface. While little attempt has been made to prove by drilling that the basins are filled with water, there appears to be no geologic reason why the Navajo sandstone aquifers in the bottom of the basins should not be filled. The volume of Navajo sandstone in the lower part of the basins is so large that they could hold tremendous amounts of ground water. A basin 50 miles wide (assuming the aquifer to be 1,000 feet thick and to have a porosity of 25 percent) could contain several hundred million acre-feet of water. Goode (6) estimated that there is about 20 maf of deep ground water in the Navajo sandstone of the Kaiparowits Basin and that about half of that amount should be available for development from depths of less than 400 feet.

It is important to note that the potential of these ground water reserves will be affected by possible slow delivery rates or by adverse water quality. Basins with thick sections of Cretaceous or Tertiary rocks, such as the Piceance Creek and San Juan Basins, will probably have saline water.

The recharge areas (exposed Navajo sandstone with the appropriate dip) surrounding the basins are quite large (especially those surrounding the Henry Mountain and Kaiparowits Basins). The basin's annual recharge rate equals the aquifer's exposed area multiplied by
an appropriate fraction of the rainfall. Exposed volcanic rocks often change this computed recharge.

Some of the ground water in the Navajo sandstone in the Upper Colorado River Basin drains to and helps support the flow of the Colorado River and its tributaries. In these instances, large withdrawals of ground water would adversely affect the surface water flows. Consequently, there are legal constraints in the large-scale development of ground water (7). The Kaiparowits Basin and the Henry Mountains Basin appear to have minimal connection between ground water and the surface water supply of the Colorado River System.

**Summary**

Water stored in the deep Navajo sandstone aquifer lying beneath much of the Upper Colorado River Basin represents a potential source of water for tapping the substantial energy resources of the basin. There are, however, physical and legal problems associated with the development and utilization of this source of water. In many cases the Navajo sandstones lie at from 2,000 to 5,000 feet (610 to 1,525 meters) below the land surface. From a legal viewpoint it is important to determine whether the ground water is connected to the surface water system of the Colorado River drainage. If it turns out that the ground water is not connected with the surface water of the Colorado River, then a case can be made that this water is not subject to the Law of the River.

Refinement of geochemical methods of distinguishing between ground water in the Navajo sandstone and Colorado River surface water will be important for this purpose. Research applications in this area should be encouraged, especially those which disclose resident time of the ground water. Reynolds and Johnson (9) have analyzed the major element geochemistry of Lake Powell and its immediate tributaries, and their work could be the basis of the needed new study.
References


