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Volume 9
Reservoir System Analysis for Conservation

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13. ABSTRACT *(Maximum 200 words)*

This is Volume 9 of the 12 volume report prepared by the Hydrologic Engineering Center of the U.S. Army Corps of Engineers as a contribution to the International Hydrological Decade.

Procedures used in the analysis of reservoir systems for conservation are discussed. The main emphasis of the volume is upon the construction, formulation and operation of a simulation model used to evaluate the performance of a reservoir system design and operating plan. Topics include identification of the reservoir system, determination of study objectives and criteria, model formulation, model validation, organization and execution of simulation runs, analysis and evaluation of alternative systems. An example illustrating the hydrologic operation of a simplified simulation model is presented for a three reservoir system.

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FOREWORD

This volume is part of the 12-volume report entitled "Hydrologic Engineering Methods for Water Resources Development," prepared by The Hydrologic Engineering Center (HEC) as a part of the U.S. Army Corps of Engineers' participation in the International Hydrologic Decade.

Volume 9 discusses modeling of reservoir systems for conservation. Objectives and criteria for designing reservoir systems are discussed in general terms. The main emphasis of the volume is upon the construction, formulation and operation of a simulation model used to evaluate the performance of a reservoir system design and operating plan. An example illustrating the hydrologic operation of a simplified simulation model is presented for a three reservoir system. This volume is intended to serve as a primer for the engineer desiring to become familiar with the basic concepts and capabilities associated with existing simulation models.

The volume was written by Messrs. Leo R. Beard, William K. Johnson, Harold E. Kubik, Edward C. Morris and Arthur F. Pabst. Helpful review comments were made by John Peters and Bill S. Eichert. Final editing was performed by Edward C. Morris.

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Introduction

CHAPTER 1. INTRODUCTION

Section 1.1 Scope

This volume describes in detail methods and procedures used in the analysis of reservoir systems for conservation purposes such as water supply, low-flow augmentation, hydroelectric power, recreation and navigation. Reservoir systems analysis for flood control is covered in Hydrologic Engineering Methods for Water Resources Development, Volume 7, and specific methods for determining reservoir yield in Volume 8.

Chapter 1 of this volume describes the objectives, nature, and modeling of reservoir systems. The general procedure for conducting a systems analysis study using simulation models is described in Chapter 2. Attention is focused upon the identification of water resources needs, formulation and validation of a simulation model, and the evaluation of alternative system designs. Chapter 3 illustrates the general procedures discussed in Chapter 2 with an example basin study including sample calculations normally contained within a simulation model.

The emphasis in each chapter is upon procedures and methods which are generally applicable to reservoir systems studies. A Users Manual for a simulation model developed at the Hydrologic Engineering Center may be found in Appendix 1 of Volume 7. This should be useful to those who desire more specific information regarding the formulation and use of simulation models for analysis.

Section 1.2. Objectives of Water Resources Systems

The goal of water resource development is to enhance the general well-being of people. Water developments may contribute to man's well-being in many ways, such as provision of safe, potable water supplies for municipal use, provision of water for irrigation or for industrial use, reduction of flood hazards, provision of hydroelectric power, and creation of water-oriented aesthetics and recreation opportunities. Reservoir systems, the subject of this volume, fulfill many of the objectives of the total water resource system. During the design of a water reservoir system the engineers must be able to evaluate this well-being so that the performance of a particular configuration may be judged. Well-being must be transformed into specific measurable objectives.

Typical objectives of water resource development are national economic development, regional economic development, environmental quality, and social well-being. The attainment of national income is measured by the national impact on employment, goods and services, and personal income changes caused by a particular development. Regional income encompasses the same items, except the area of influence would be contained within a smaller geographic area. The environmental objective measures how well the quality of man's environment is enhanced or maintained. Social well-being assesses man's health, safety and culture. Once the objectives are postulated, the design of a water reservoir system will be based on how well the system fulfills the specified objectives.

Section 1.3. Nature of Reservoir Systems

Design of a reservoir system must consider that the natural water supply is variable and unpredictable, and requires speculation upon man's diverse and changing needs for water. The nature of a reservoir system will be shaped by the engineer's perceived notion of these considerations.

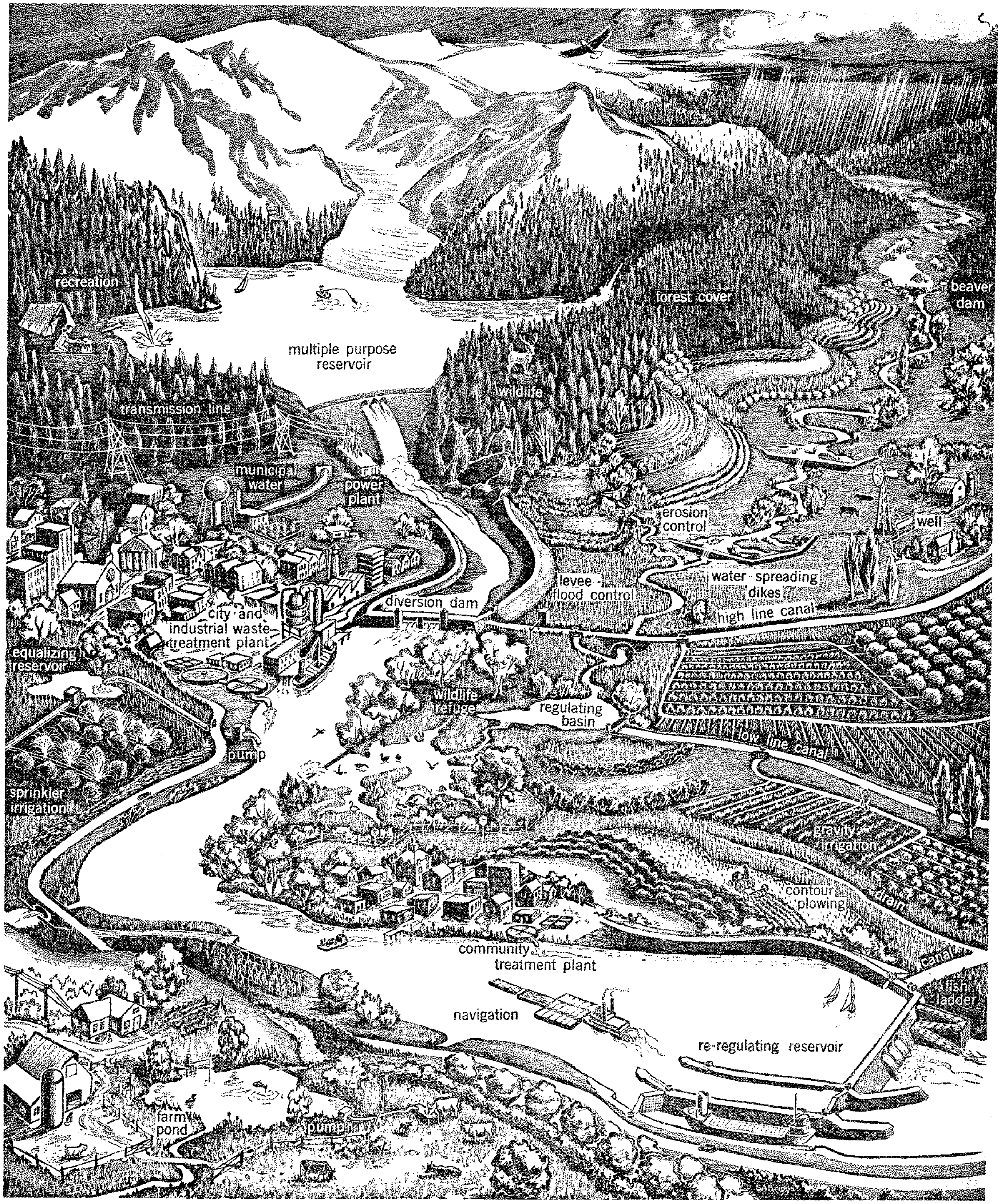
Water as it occurs naturally falls upon the earth at variable times and locations and in uncertain magnitudes and quality. Water demand is multipurpose, seasonally variable, and not altogether predictable. This variability in time, location, quantity and quality of both supply and demand is the reason reservoirs and related development works are necessary. Reservoirs smooth out this variability by reducing high flows and increasing low flows, and generally make water available when and at the location it is needed.

Figure 1.1 illustrates a variety of components that might be contained within a reservoir system. Reservoirs, diversion dams, power plants, levees, navigation locks, natural and man-made channels, pumping plants and wells may be included in a water resource system configuration.

Reservoir systems may be grouped into two general categories: conservation and flood control. Conservation encompasses water supply, low flow augmentation for water quality, recreation, navigation, irrigation, and hydroelectric power and any other purpose for which water is saved for later release. Flood control is simply the retention of water during flood events for the purpose of reducing downstream flooding. This volume will discuss the considerations involved in the design and regulation of a reservoir system comprised of conservation reservoirs. Separate analysis of flood control reservoirs which would be necessary in a water resources system analysis is discussed in Volume 7 "Reservoir Operation for Flood Control."

Section 1.4. Modeling Reservoir Systems

A system's complexity is an important factor when selecting methods and techniques for analysis. Complexity is influenced by the hydrologic system, size of the river basin, and variability of rainfall, runoff, and groundwater. The simplest system might be a single project reservoir serving a water supply need. A complex conservation system might consist of a number of reservoirs, power plants, and diversions serving water supply, hydroelectric power, water quality and irrigation.



A Multiple-Purpose River Basin Development

(From "A Water Policy for the American People," Report of the President's Water Resources Policy Commission, 1950.)

Figure 1.1

Systems analysis is often employed in the design and operation of reservoir systems because of the number and interdependence of its components. The engineer is faced with a multitude of possible designs, each having an infinite number of operating rules. System techniques gather information about the system's performance to be used either directly or indirectly to infer design alternatives and operating rules. An organized procedure is required to model the physical characteristics of the system, in such a manner as to accommodate its complexities and interdependencies, without losing mathematical tractability.

Three basic methods which have been employed in planning, design, and operation of water resource system studies are: simplified ones such as non-sequential analyses; optimization analyses; and simulation analyses. The latter two have been applied with the implementation of the computer in water resources studies.

Because the simpler procedures were developed for hand computations, the complexity of the system which can be analyzed and the period of hydrologic records which can be used are severely limited. Generally only one reservoir with one purpose can be evaluated using data for only a critical flow period. Simulation analyses by hand are still used where computer programs are not available or where selective checking of calculations is necessary.

Optimization models are analytical algorithms which use mathematical techniques such as linear, non-linear and dynamic programming to optimize one or more expressions of system performance according to a predetermined evaluating scheme. Generally, they are used to attempt to demonstrate the sensitivity of objectives, parameters, and operation policies. In order to represent a reservoir system in closed form, a great number of assumptions and approximations are required. Natural hydrologic variability, an important aspect in hydrologic operations of a water resources system, cannot be handled in a computationally feasible manner.

Typically, linearization of some or all of the system's governing equations and evaluating functions is needed to produce a converging closed form solution. As a result the optimization model no longer solves the real problem but an approximate representation with the hope that the resulting solution is relatively close to the true optimal set.

Experience in the actual design of water resources systems has indicated that the complexity of the problems involved and the irregularity of the mathematical functions involved are so great that the utility of optimization techniques have been restricted to the solution of small parts of the over-all problem or to highly simplified versions of the problem. A great deal of research for improving these and associated techniques is currently underway. It is hopeful that the future may bring sufficiently powerful mathematical techniques for providing more direct solutions to optimum design of reservoir systems.

A simulation model is a set of mathematical relationships that describe the spatial and temporal operation of the system. The model can take into account the stochastic nature of rainfall and streamflow, handle the non-linear governing equations and benefit functions, and keep track of the system's operation. Its purpose is to represent and operate the system in as much detail as possible, providing the necessary information to evaluate how well an alternative performs. The model is based on sound physical principles that preserve hydrologic parameters, and provide tremendous model flexibility. However, because of its complex nature, simulation models cannot be represented in analytical form. Thus optimization within the simulation framework is not usually attempted. The optimality of an alternative is a function of the engineer's ability to manipulate design variables and operating policies in an efficient manner. There is no guarantee that a globally optimal alternative will be found. However, the ability of the simulation to provide essential information on the performance of the system and the extent that objectives were satisfied far exceeds the capabilities of optimization models.

Simulation models alone have generally been used more often on reservoir systems analysis than have combinations of optimization and simulation studies. Essentially good engineering judgment may be used in lieu of optimization models to determine preliminary location, type, and sizing of components in reservoir systems configurations. Unless the engineer has sufficient experience and ingenuity in optimization studies to formulate the design and operation of a reservoir system into an optimization format, simulation alone remains the more practical approach. Emphasis in the remainder of this volume will be on a description of the general procedure and methodology used to simulate the operation of reservoir systems.

Chapter 2

General Procedure

CHAPTER 2. GENERAL PROCEDURE

Section 2.1. Introduction

The general procedure for conducting a reservoir system analysis for conservation purposes using simulation is first, to identify the system; second, to determine the study objectives, and specify the criteria used to measure objectives; third, to examine the availability of system data; fourth, to formulate a simulation model which is mathematically and quantitatively representative of the system's components, hydrology, and operating criteria; fifth, to validate the model; sixth, to organize and execute simulations; and finally, to analyze and evaluate the simulation results according to how well they achieve study objectives. This general procedure represents the basic stages that an engineer frequently goes through in making an analysis. The sequence assumes that the engineer has knowledge of past and future stages. For example, while examining and collecting data it is necessary to look back to the criteria established to assess objectives and to look ahead to how the criteria will be evaluated to insure that the necessary data are available. Thus the general procedure is iterative in nature and one should have in mind the entire procedure as he works at each stage.

In the sections which follow, each step in this general procedure will be elaborated upon to provide a basis of understanding for the illustrative example described in Chapter 3.

Section 2.2. System Identification

An important first step in any system analysis study is to identify the geographic, hydrologic and physical features of the system. What specifically are the geographic boundaries of the system? What are the topographic and climatic variations within a region?

Geographic boundaries are commonly identified to coincide with the hydrologic or watershed boundaries. However, in metropolitan areas watershed bounds are difficult to establish and governmental boundaries often dictate the system to be analyzed, thus necessitating a careful accounting of water transfers in and out of the system.

Logical subdivision or sub-basins should be identified in case it becomes desirable to analyze smaller configurations of the system in the analysis. This sometimes occurs when large systems exceed the simulation model's capacity. Also, there may be special interest in a subsystem because it differs hydrologically from the rest of the system or because it may be constructed first.

The physical features of the system should be inventoried in order to aid in the initial derivation of potential design alternatives, and to highlight prominent characteristics which may need to be incorporated into the formulation of the simulation model. Sources and configuration of population centers indicating areas of existing or future water demand, and features compatible with potential development measures should be identified.

Also, special areas should be noted such as stream reaches of low flow or poor quality, potential reservoir sites, and any locations where demands for water may exist. A complete and thorough identification of the system early in the study can be very helpful later when decisions are made regarding data and model formulation.

Section 2.3. Determination of Study Objectives and Criteria Used to Measure Objectives

Selection of study objectives is dependent upon the scope of the study and the perceived water resources needs in the area of interest. In planning studies the usual question is, what combination of facilities or system components can best meet the water needs? In design, information is usually needed to refine the size and location of specific facilities structured to satisfy planning needs. In operation studies, information is sought to improve the operation of the system; for example, to minimize reservoir fluctuations while maintaining a selected level of power production. Also, as one proceeds from planning to operation, the level of detail and need for accuracy increases. These differences should be reflected in the selection of objectives, specification of criteria, and formulation of the model for analysis.

As outlined in Section 1.2, study objectives are grouped into several broad generalizations which must be specified in more detail prior to the evaluation of design alternatives. Typically, criteria are defined to quantitatively measure how well design alternatives satisfy study objectives. For example, an economic criterion could be defined to assess how well a particular diversion, irrigation structure, or reservoir satisfies a national or regional income objective. Other indexes such as employment figures, gross national product, or other economic indicators could be used as a supplemental or alternative criteria. The inundated area containing important natural or mineral resources could be one of the criteria used to appraise the environmental objective. The degree of social well-being could be determined from the number of people displaced by the water resources project. By defining criteria to quantify objectives, the performance of the simulation model may be gaged more effectively and completely.

When feasible, it is desirable to limit the proliferation of criteria used to define a single objective in order to avoid difficulties in the comparison of design alternatives. This is especially true in a multi-objective framework. As an analysis progresses, it may be desirable to modify original criteria or even objectives themselves to reflect information gained during the early analysis. Reservoir systems analysis is a dynamic process and as alternative systems are analyzed and better understood, the feasibility of fulfilling certain objectives should be reassessed.

Section 2.4. Examination and Collection of System Data

It should be recognized early in the analysis that for the simulation to be representative of the real system, considerable data will be necessary. The availability of this data will directly influence both how well a particular model will be able to represent the real system and the type of model that should be selected for the analysis. Basically, three types of information are necessary: data characterizing the physical components; data describing the hydrology; and data delineating the operating criteria and system requirements. Specific examples of each type are discussed in Sections 2.5 through 2.8.

The initial step in data collection involves determining the time period and recording interval for which data are available, the form of the data and the availability of personnel and facilities to process these data. For example, hydrologic data, primarily streamflow records or records from which streamflow can be derived are a very important requirement of simulation. The first question asked is, are streamflow records available, and if so, at what locations in the system? What is the period of record? Is the data continuous, daily, weekly, or monthly? What form is the data in? Is it tabulated in a bound volume or punched on computer cards, or on magnetic tape? What processing will be necessary to put it in the form required for simulation? Similar questions should be asked about the other types of data. This examination of data will help identify problem areas where data are not available or inadequate, or where data will be difficult to obtain or process. Volume 2 discusses hydrologic data management in detail.

In analyses conducted for planning studies it is often necessary to make assumptions regarding the characteristics of system components and various operating criteria. When making these assumptions the analyst should utilize available information to the greatest extent possible. Existing reservoirs, power plants, diversions, navigation locks, etc., can be a valuable source of information about future facilities and their operating rules.

Section 2.5. Model Formulation

A simulation model is formulated by describing mathematically and quantitatively, the components, hydrology, operating criteria and requirements of the system. If the simulation is to be performed by hand, this means setting up the necessary tables; if by computer, it means adapting an existing computer program to the system or developing a new program. The most common approach is to adapt a generalized computer program which has already been tested and used to model similar systems. Modifications can often be easily made to the program to accommodate unique features of a system. Decisions as to which procedure to adopt will depend upon the objectives of the analysis, what the existing program can do, the data available, and the cost and time required and available.

The purpose of the simulation model is to determine how well a set of design variables contributes towards maximizing the objectives discussed in Section 2.3. During model formulation the engineer must specify the hydrology, system components, and system operating rules and requirements. Since the last two items consist of design variables, the engineer must assume initial design components and operating rules which will probably require modification to attain optimum results.

The ultimate goal of reservoir system analysis is to construct an optimal configuration which is operated in the most advantageous manner to meet study objectives. During the design process the engineer is faced with numerous potential configurations and an almost unlimited number of possible operation rules. Each particular design alternative has a multitude of operating policies associated with it; hence, a totally exhaustive test of all configurations and rules would be necessary to determine the truly optimal combination. Because small changes in operation may affect the system evaluating functions for long periods of time in very subtle ways, it is not sufficient to compare alternative configurations without considering the regulation policies of the system. Therefore the engineer should construct the model so that design variables such as reservoir sizing, storage allocations and operating rules may be easily and efficiently manipulated and evaluated. This will facilitate the evaluation of the sensitivity of the variables as the engineer proceeds through the analysis. The process of determining design variables through simulation is iterative in nature. Based on the results of initial runs, it may become apparent that certain design or operating rules must be changed to improve the model's performance in subsequent runs.

The hydrologic components of the model are discussed in Section 2.6. The system components which must be specified in the simulation model are examined in Section 2.7. Section 2.8 outlines comprehensive procedures for developing system rules and requirements.

Section 2.6. System Hydrology Formulation

Three main hydrologic quantities that are commonly needed in simulation studies are the reservoir inflows, local inflows at control points, and reservoir evaporation. Both reservoir inflows and local flows are expressed in terms of magnitude and time period and are collected for all the important points in the system. Streamflow data should be transformed to uniform basin development conditions and data should be estimated for any missing time periods, according to the procedures outlined in Section 3.01 of Volume 8. When the streamflow record lengths are short or information is required at ungaged sites, regional analysis or synthetic derivation of data may be employed as discussed in Chapter 4 of Volume 8.

Water surface evaporation is important for long interval routings such as weekly or monthly periods to maintain a proper hydrologic balance in reservoirs by accounting for losses. Evaporation is described in terms of magnitude, time of occurrence and location. Sections 3.02 and 3.05 of Volume 8 review how evaporation losses are estimated and incorporated into equation form. Depending on the geology of the area, other losses, such as leakage and seepage may be important. These miscellaneous losses are examined in Section 3.03 of Volume 8.

Section 2.7. System Component Formulation

The primary components of a reservoir system include the reservoirs, stream channels, and if applicable power plants and diversions. The physical characteristics of reservoirs are presented by storage capacity and corresponding elevation, surface area and spilling and/or outlet capacity. In multi-purpose reservoirs the storage allocated for each purpose is specified. The procedure for determining storage allocations is discussed in Section 2.8. In order to maintain the proper hydrologic balance among the components, the continuity equation must be applied. This equation requires the change in storage for a specified time period to be equal to the difference between inflow and outflow, including diversions and any losses due to evaporation.

Stream channels convey the natural runoff and reservoir releases to different locations in the system. Channel capacity and hydraulic efficiency are two important channel characteristics which influence conveyance. Channel capacity can be described at selected locations in the system by specifying maximum permissible flows to the simulation model. While hydraulic efficiency is important when routing flows from one location to another, it is often ignored in reservoir system studies for conservation. This is because the interest is in low flow conditions rather than flood flows, and routing intervals of weekly or monthly periods are used which are usually longer than the travel times in the basin. Thus, for analyses involving conservation purposes the channel network can be adequately described by identifying the capacity and location of important points in the system.

Simulation of hydroelectric power plant operation requires description of those characteristics which affect power generation. These would include installed plant capacity, effective head, plant factor, plant efficiency and various relationships between reservoir storage and power plant releases and efficiency.

Simple diversions may adequately be described by specifying location and rate of withdrawal. Complex diversions are sometimes based on the available reservoir storage, magnitude of reservoir inflow or some other hydrologic characteristic.

Many of the system characteristics such as stream channel parameters, and elevation-storage relationships will remain constant throughout the simulation process. Other characteristics such as storage capacities, storage allocation zones, power plant capacities, diversion placements and sizing are design variables which will be manipulated iteratively during the simulation process until project objectives are satisfied.

Section 2.8. System Operating Rules and Regulations

Section 2.8.1. General Considerations

Operating rules for water resource systems must be established to specify how water is managed throughout the system. These rules are specified to achieve system streamflow requirements and system demands in a manner that maximizes study objectives.

System demands may be expressed as minimum desired and minimum required flows to be met at selected locations in the system. The distinction between desired and required flows, discussed in Section 3.3, is useful when the reservoirs are unable to furnish desired flows, in which case releases would be made for only the high priority required flows. The specification of operating rules at a reservoir site include the volume of storage allocated to the pool by time period, identification of each downstream location for which the reservoir will operate, and the priority system upon which the reservoir will meet demands. In addition rules may be established that govern diversion schedules, maintain minimum flows, and balance storage among reservoirs.

Some or all of the operation policies may be designed to vary seasonally in response to the seasonal demands for water and the stochastic nature of supplies. Operating rules, often established on a monthly basis, prescribe how water is to be regulated during the subsequent month based upon the current state of the system. As discussed in Section 5.01 of Volume 8, other time intervals may be necessary depending upon the fluctuations of demand and supply and the purpose of the various system components.

Because simulation models do not generally contain optimization routines, several runs with different operating rules must be performed to provide a basis of comparison. Value functions discussed in Section 2.11 are used within the simulation model to measure how well a set of design variables performs. When testing operation rules these functions should relate benefits as they occur in the reservoir system components rather than potential future services. Hence hydroelectric power benefits would be computed directly from monthly power releases necessary to generate power and not to the current water storage, which is merely an indicator of available water for present or future use.

Benefit functions, which are designed to evaluate how well target demands are satisfied, generally indicate that shortages cause severe adverse consequences while surpluses may enhance benefits only moderately. It is common practice to define operating rules in terms of a minimum yield or target value. It must be realized, however, that if the water supply to all demand points was rigidly constrained when droughts occurred it would be impossible to satisfy all demands. Higher priorty demands could be specified at some of the points in order to ration scarce supplies, but the determination of which points and at which critical storage levels these decisions should be made must be tested.

Operation rules are often defined to include target system states, such as storage, above which one course of action is implemented and below which another course is taken. Reservoir storage is commonly divided into different zones as shown in Figure 2.1. Figure 2.2 illustrates seasonally varying storage boundaries which are target levels for the various modes of operation. Flood control rule curves, for example, may provide for releasing as much flood water as possible when the water rises above the target level A, and the maximum possible flood-control releases without causing flooding when water is at or below that level. Curve B defines the bottom of flood pool. The reservoir would be kept at or below curve B when possible in order to provide sufficient storage for flood control purposes. Note that for this particular reservoir the flood control storage varies with the time of the year. This reservoir was located in a region where the threat of floods is less during the summer. Thus, a portion of the flood control space could be used for conservation purposes during the summer months. Likewise, rule curves may call for curtailing lower priority services when storage falls below a target level or for declaring surplus power or water when storage rises above a target level. Curve C of Figure 2.2 shows the pool elevation below which only critical

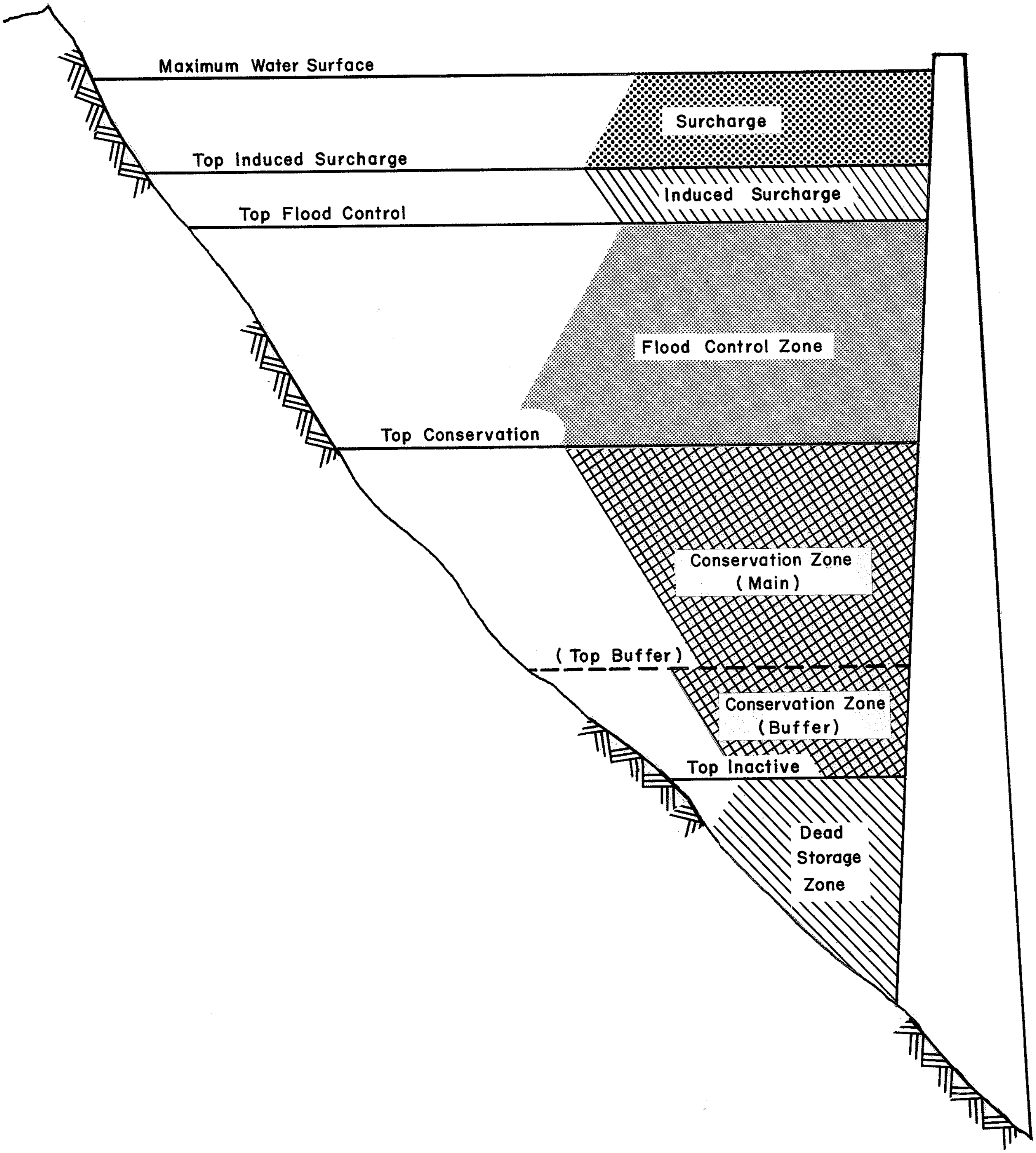


Figure 2.1 Reservoir Storage Allocation Zones

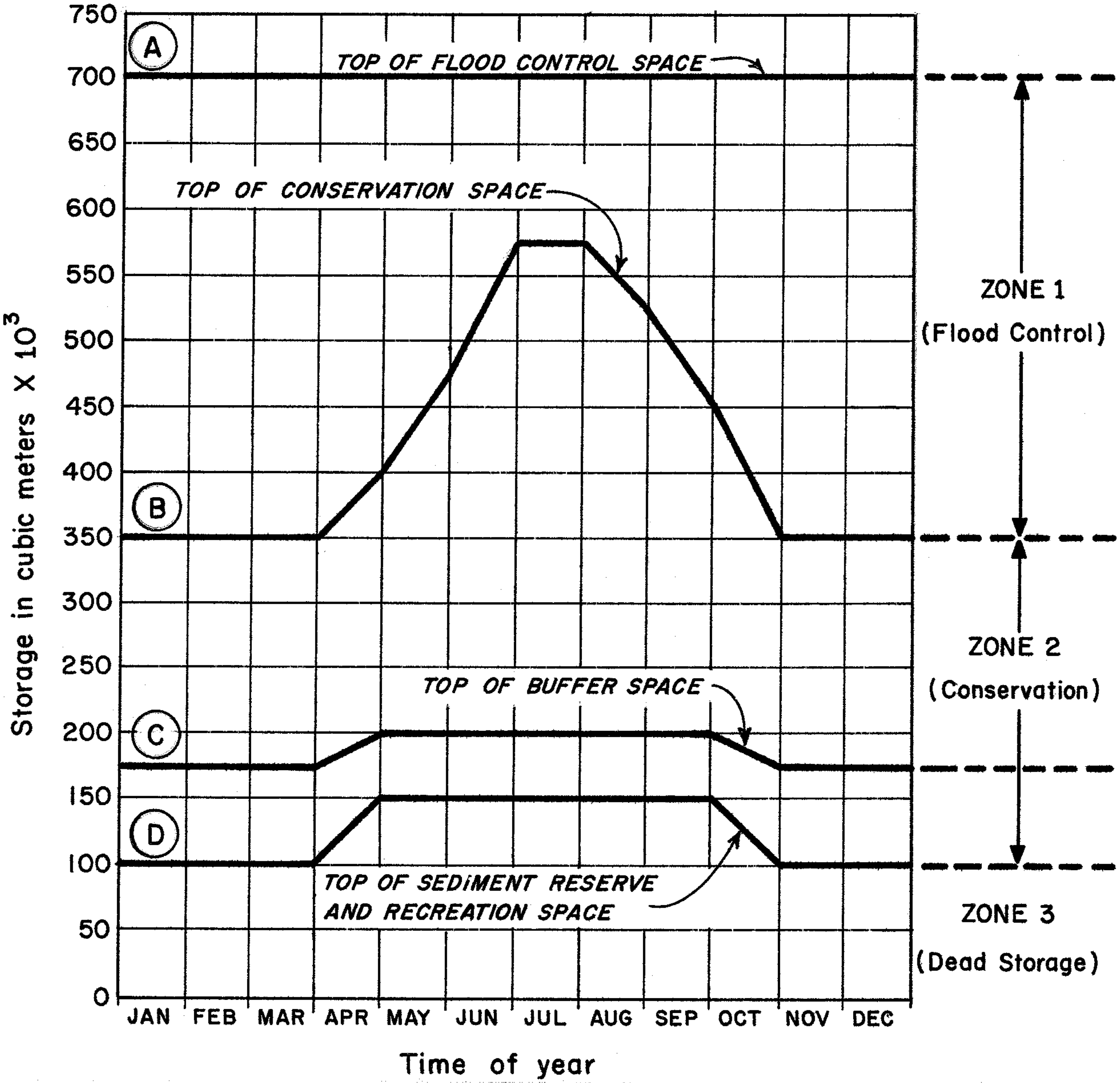


Figure 2.2 Example of Seasonally Varying Storage Boundaries for a Multipurpose Reservoir

services would be maintained. Again note that in the reservoir the level depends on the time of year, anticipating the occurrence of a wet period near the end of the year.

When a set of operation rule curves for one or a number of purposes must be developed, whether or not services conflict must be considered, and consequently the problem of deriving rule curves is greatly complicated. In addition, when a number of reservoirs serve the same purposes, it is usually necessary to develop system rule curves related to the total storage in all such reservoirs and then criteria for allocating the storage among the reservoirs.

It is essential that operation rules be formulated with information that will be available at the time when operation decisions are made. If forecasts are used in operation, the degree of reliability must be taken into account in deriving operating rules. In assessing the benefits that would be associated with a particular set of operating rules, forecast errors must be simulated in such a manner as to represent average anticipated accomplishment under those rules.

Likewise, all physical, legal and other constraints must be considered in formulating and evaluating operation rules. In system operation studies particularly, it must be recognized that owners of various system elements might not be induced to operate their element according to the system rules, and this is an additional uncertainty factor that must be considered in evaluating operation rules. Nevertheless, there are times when even the physical and legal constraints are subject to change, and this should not be forgotten when a particular constraint seriously affects system accomplishments. Finally, changes in operation occasioned by a rule curve should not be sudden and without warning. If necessary, a transition zone or provision for advance notice should be incorporated into the operation rules.

Section 2.8.2. Initial Estimates of Rule Curves

Rule curves are developed to provide guidance on what operational policy is to be employed at a reservoir or dam site. The operational decision is based upon the current state of the system and the time of year which accounts for the seasonal variation of reservoir inflows. A simple rule curve may base the next period's release solely on the current storage level and the current month. A more complex rule curve might consider storages at other reservoirs, specific downstream control points, and perhaps a forecast of future expected inflows into the reservoir. Generally, most curves are of the simpler variety because of their ease of application.

The derivation of initial rule curves requires an assumed reservoir storage capacity, and a release schedule designed to satisfy downstream water requirements. Because the adverse consequences of shortages are predominately those that occur during the single most severe drought of operation, a reverse routing procedure can be employed during the most critical period of record to derive an initial estimate of the rule curve. The most critical release schedule which provides only minimum required flow is specified by the rule curve in order to provide for acceptable shortages or desired contingency allowances during that critical period. If release schedules vary seasonally, releases used in this approximation should be the most adverse expected to accompany the critical sequence of hydrologic events.

The reverse operation study starts at the end of the critical period with the minimum permissible system storage. Releases and net evaporation are added and inflows subtracted from the end-of-interval storage to compute the storage in the previous interval. This process is repeated until the start of the critical period is reached, making sure that the minimum required flows are not violated in any interval. The storage in the reservoir during the reverse operation represents the minimum amount of water necessary to meet the minimum required flows during the critical period. If the maximum permissible storage is surpassed then the assumed demand schedule will need to be reduced, or a shortage of water will need to be accepted, or storage allocated to the conservation zone will need to be increased. If the minimum storage is violated then the critical period is actually shorter than assumed, at least in part of the system.

The initial estimate of the system rule curve is then the enveloping value of system storages, enveloping maximum storage values obtained each time of the year for conservation release schedules.

As an example, the initial derivation of a rule curve for conservation is shown in Figure 2.3. A critical storage was selected at the end of the critical period in October. Figure 2.4 shows the inflows during the critical period and the desired releases. Figure 2.5 displays evaporation estimates from the reservoir surface during the year and Figure 2.6 shows the relationship between pool elevation, surface area, and storage volume. The release and evaporation for September may be determined from Figures 2.4, 2.5, and 2.6 and added to the starting storage in October. Any inflow during September is then subtracted to yield the storage at the beginning of September. The reverse routing is continued in this fashion until the start of the critical period which occurs when the reservoir is full.

When determining rule curves among the various reservoirs in the system it must be recognized that critical conditions may not be attained at all projects in the system at the same time. In addition, when considering two reservoirs in series, the upstream reservoir release schedule will bias the development of a rule curve at the downstream one. For parallel reservoirs the best rule curve may require apportionment of releases from two or more reservoirs based upon available storage capacity or some other criteria. If the initial estimates of rule curves are

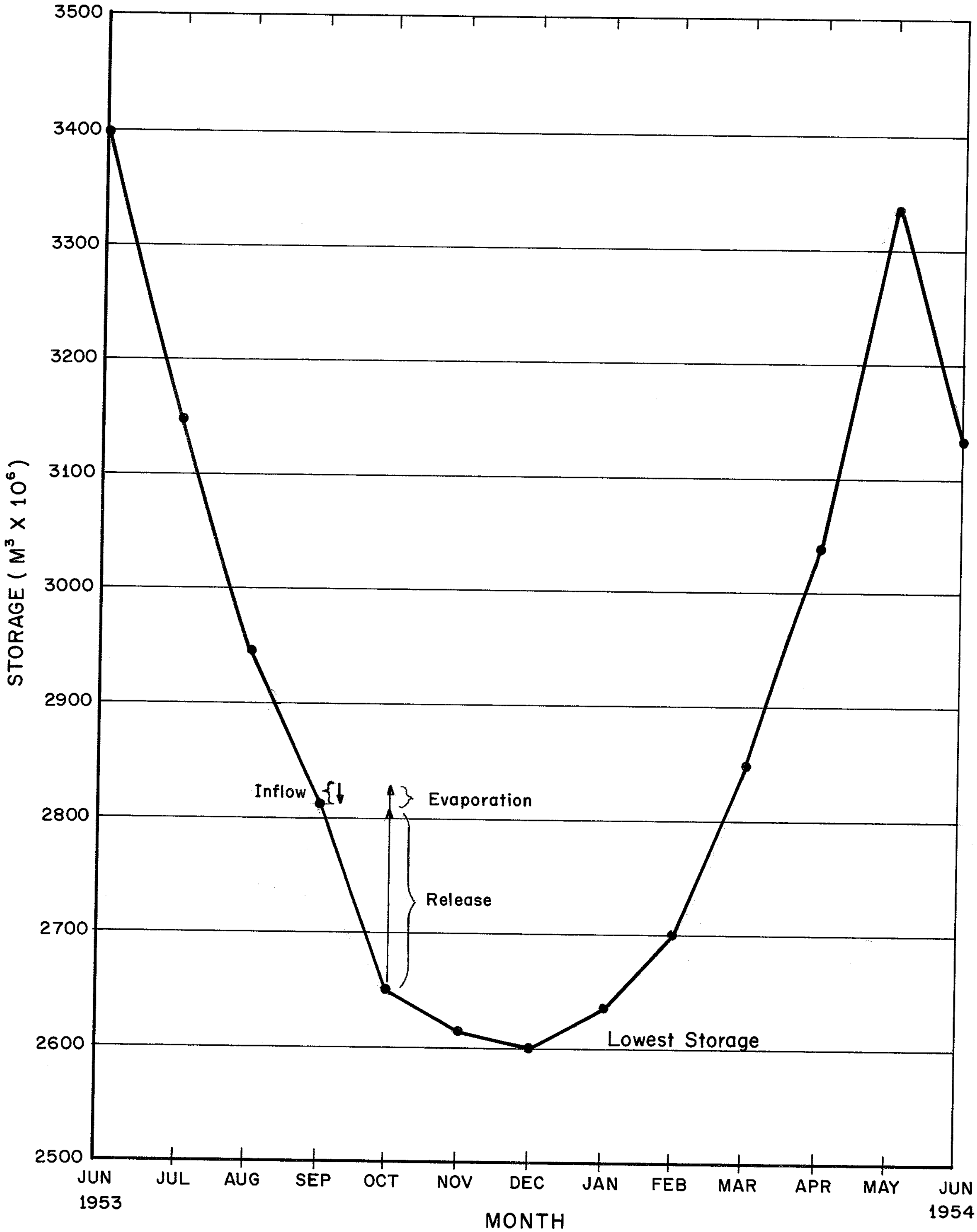


Figure 2. 3 Storage in Bull Shoals Reservoir, White River, Arkansas

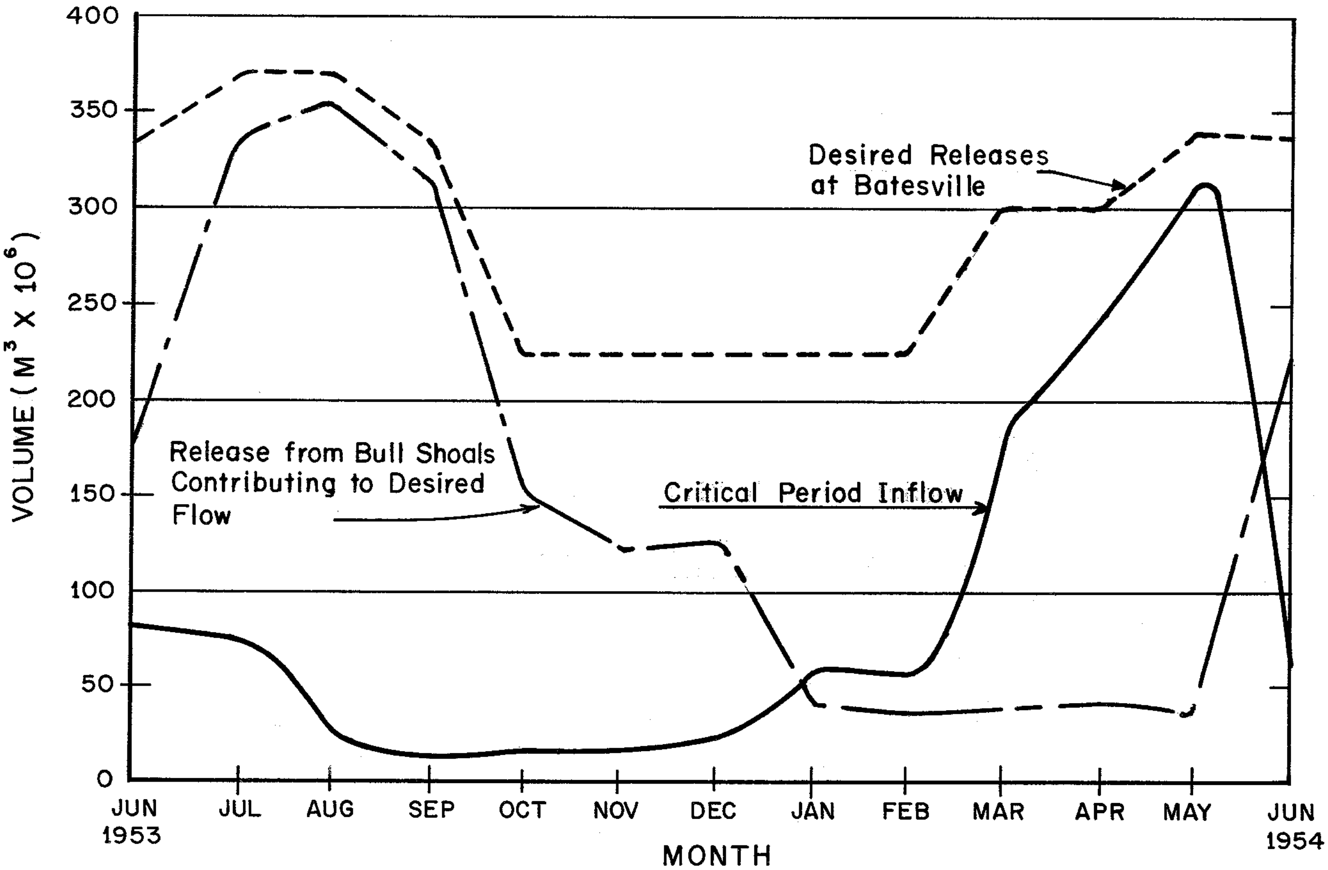


Figure 2.4 Monthly Critical Inflows and Desired Release Volumes for Bull Shoals Reservoir, White River, Arkansas

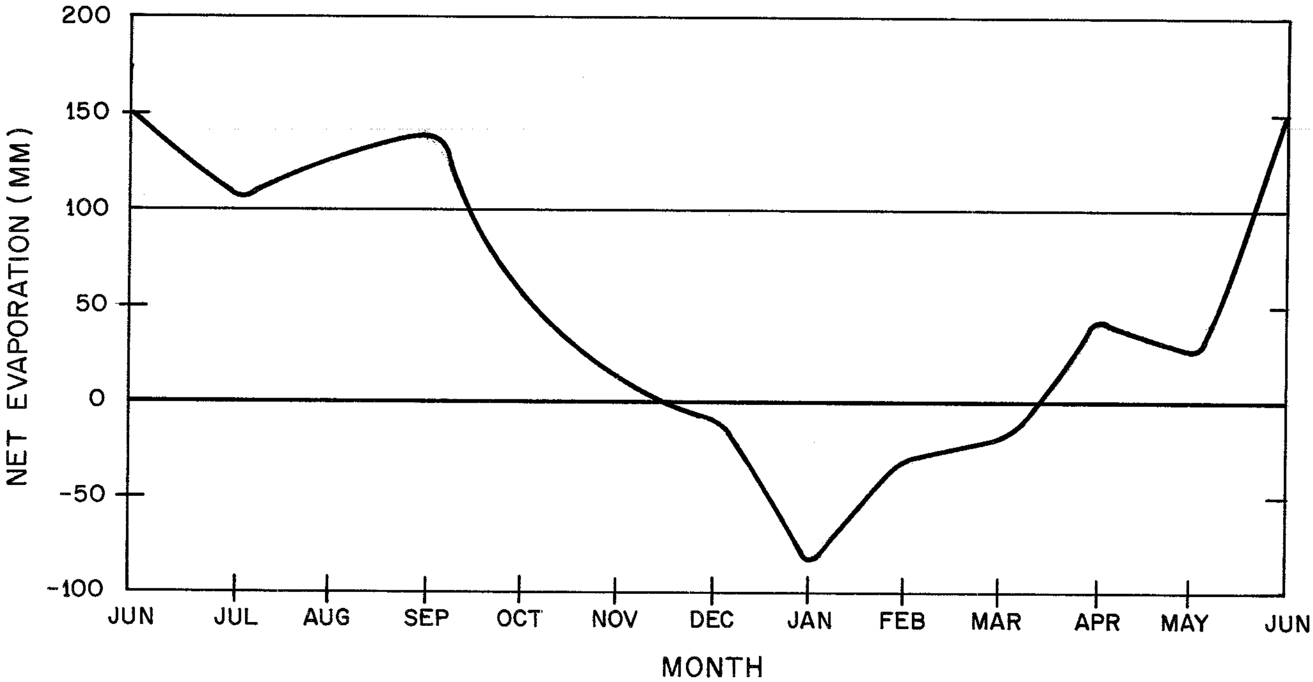


Figure 2.5 Average Net Evaporation for the White River Basin, Arkansas–Missouri

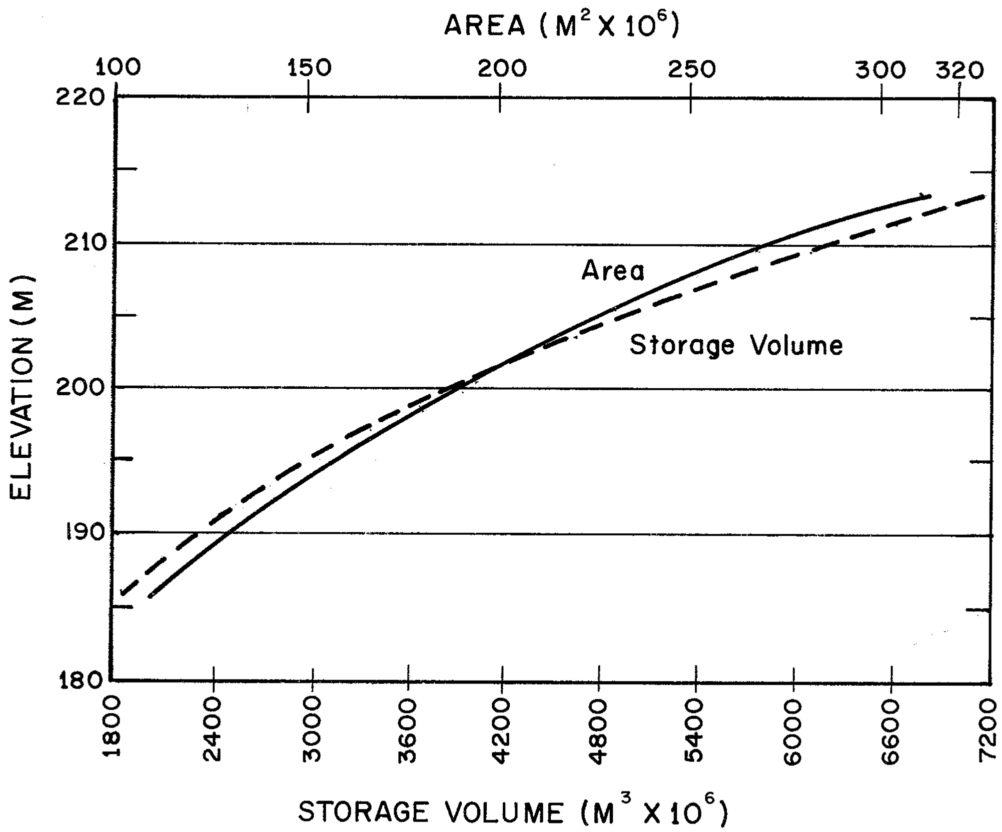


Figure 2.6 Area Capacity Curve for Bull Shoals Reservoir, on the White River, Arkansas.

derived independently, then substantial refinement will be necessary before adequate system reservoir release rules are achieved.

Section 2.8.3 Development of Rule Curves

Because of the complex interdependencies of system operating rules, it is usually necessary to simulate the system operation to determine a workable regulating scheme. As outlined in Section 2.8.2, initial curves may be estimated using the most critical historic sequence. These independent estimates must then be simulated within a hypothetical operation of the system to insure that system targets are satisfied, project objectives are maximized, and an equitable distribution of water within the system is maintained. Thus, an iterative procedure will be required that focuses careful attention to the establishing of operation rules that attain these goals.

When the balancing of reservoirs is an important consideration, the concept of index levels, which is outlined in Section 10.05 of IHD Volume 7, may be employed as an aid to making release decisions to keep the reservoir system in a balanced state. As explained in Section 2.8 and shown in Figure 2.1, reservoir storage allocation may be subdivided into flood control, conservation, and one or more buffer zones. The top of each of these zones has a corresponding reservoir level which is assigned an integer or index level. When the index levels at all the reservoirs coincide, the system is in balance. In reservoir operation, if the current index levels are unbalanced, the releases for the subsequent interval are adjusted insofar as possible to restore equilibrium. The index levels are also useful in the regulation of parallel reservoirs via the equivalent reservoir concept which is described in Section 10.05 of Volume 7.

When considering multipurpose reservoirs, inevitable conflicts will arise in the construction of rule curves for the various uses. Section

7.03 of Volume 8 outlines the development of a single purpose rule curve at one site for power operation, and Chapter 10 of Volume 7 provides some guidance on multiple reservoir operation for conservation and flood control purposes. However, the dilemma of handling an array of multipurpose reservoirs is usually overcome through an iterative process whereby separate rules for the various modes of operation are initially assumed, tested, then modified to improve the overall operation.

After an initial estimate of a set of rule curves for all system elements and purposes is available and the procedure for balancing reservoirs established, it is necessary to simulate the system operation based on those rule curves using the entire period of recorded hydrologic information, adjusted to present or future conditions. It is desirable to operate the system with one or more synthetic hydrologic sequences, to insure that there is no undue bias in the operation rules resulting from the unique set of observed flows. The development of synthetic hydrology is discussed in Chapter 5 of Volume 2, and advantages and limitations outlined in Section 4.03 of Volume 8.

A close examination of storages obtained in such a simulation study in relation to rule curve storages at each reservoir for each month of the year will help reveal whether any rule curve can be advantageously changed at any point during the year.

If, priority releases are curtailed because usable storage is depleted, it is necessary to examine preceding storages back to the time when the reservoir was full to determine whether and where operation rules permitted releases for secondary priority purposes. The rule curve would then be raised by the amount of priority shortage or additional water drawn from other reservoirs. If the curve is raised, the subsequent rule curve is above the reservoir storage by at least the amount of that shortage, up to the end of that particular drought period.

This procedure is, of course, only approximate, because evaporation and power requirements will differ with different storage patterns. Furthermore, the initial curves were based on the assumption that all usable storage can be used during that drought period and that no shortages are to be permitted. In the more general case, repeated simulation and evaluation, as described previously, will be required in order to arrive at a set of rule curves that make the best use of available resources. This usually means incurring shortages of acceptable magnitude and frequency in the interest of providing greater service with the same facilities or providing the specified service at minimum cost.

In developing storage balance curves or dividing rule curve storage among the various reservoirs, studies as described above must be made using the least possible releases from each reservoir needed to serve conservation demands at all points downstream. Reservoirs operating in parallel permit some latitude in relative releases, although they should be operated so as to maintain about the same degree of reserve in each reservoir relative to inflow flood potentials at the respective reservoirs. In actual operation, some exchange of rule curve storage among reservoirs can be permitted if simulation studies demonstrate this to be safe.

Section 2.9. Model Validation

Validation of the simulation model is necessary to insure that it functions precisely as it was intended. There are two frequently used ways to validate a model: simulation of the operation of the existing system with known responses; or simulation of a small hypothetical, test system that enables checking the operation by hand computations. The former approach is preferred because the existing system is the system of interest in the analysis. There may be functional relationships in the existing system that were overlooked or assumed in the model that have a significant influence on system response. For example, to minimize computer time a limited number of control points may have been selected. Comparing the model's results with the existing system may show that additional control points are necessary to accurately represent the system operation. Hand computations of small hypothetical systems have the advantage that they explicitly identify values at each step in the computation procedure. This is useful to acquire knowledge of how the model functions.

When adapting available generalized computer programs, it is important to validate the model. While it may be known that the program has been used successfully on numerous other systems, there is the possibility that the program still contains erroneous computation procedures which have not caused difficulties in previous use with other system configurations. Also, differences in computer hardware sometimes cause difficulties or the program's capability could be misunderstood. It is important therefore, even with working programs, to test the model's ability to simulate an existing or test system prior to its use.

Another reason for validating the model is to uncover any errors, inconsistencies, gaps or improper preparation of system data. Problems sometimes occur when data is transferred from a printed source to computer input. Making simulations of the existing system often uncovers these problems and allows an opportunity to correct them before the bulk of the study data is processed.

Section 2.10. Organization and Execution of Simulation Runs

Computer simulation models produce large quantities of information. The output from studies of large systems can overwhelm the analyst by exceeding both his ability to review and to store it. In order to overcome the problem, an efficient management strategy for organizing and executing the simulation runs should be developed. The objective of this strategy should be to obtain the desired information in as short a period of time and with as few runs as possible. In addition the engineer should design an efficient, clear output display during model formulation. Summary tables of important information are an effective contribution in this area.

The first step is to examine the number of possible systems to be analyzed and their components. Each system consists of an array of design variables that prescribe both its configuration and operating policy. It is necessary to construct a model that provides information about the design variables and those components that describe the performance of the system towards satisfying study objectives. This enables the engineer to make inferences about the impact of design variables on the system performance.

The second step is to set up a procedure for comparing the tradeoffs in performance of alternative systems. Initially a broad range of distinctly different systems may be compared. As a study progresses, inadequate systems are eliminated and the differences between alternatives usually become smaller. At this point considerable effort can be wasted by making too many runs just "to see what happens." The best way to overcome this tendency and thereby decrease the number of runs, is to learn what the critical aspects of the system are. This simply means understanding what is happening and why. For example, the operation of a complex system may be controlled at one or two locations that are not readily apparent. Regardless of what changes are made in other components, the system's response will remain essentially unchanged because it is being constrained by a few key requirements. Thus, an attempt should be made to have understanding replace a random selection process.

Management of simulation runs can also be improved by carefully developing a labeling and/or numbering scheme for the alternative systems and operating rules. For example, it is advantageous to number subsystems in a way that they can be identified as subsystems should it be desired to simulate them separately. Also, any numbering scheme could be used to identify such items as the addition or elimination of system components, the set of input data used, the date of simulation, any sequence number to indicate how many previous runs have been made, whether the run represents present or future conditions and any affiliation with participating agencies. This information will facilitate locating and storing runs during a study. A separate file of summary information may also be desirable for keeping a record of all systems run.

Section 2.11. Analysis and Evaluation of Alternative Systems

Section 2.11.1. General Evaluation Technique

As discussed in Section 2.3, study objectives and performance criteria are defined during the initial stages of water resource systems planning. Criteria is used to measure how well a system configuration fulfills study objectives. Usually criteria is related to system states such as monthly releases or reservoir levels. Value functions mentioned in Section 2.8.1 are often defined that relate states such as reservoir stages to dollar benefits or damages. Similar functions may be introduced that relate other purposes such as hydropower or recreation benefits to stages or discharge. When specific targets, outputs or constraints are imposed upon the system, other measures, such as shortage indexes may be employed. The combination of value functions and indexes are then interpreted to evaluate the over-all performance of a configuration.

Section 2.11.2 discusses criteria in detail. Evaluation functions are outlined in Section 2.11.3, and use of critical period analysis is presented in Section 2.11.4. Section 2.11.5 is devoted to special hydropower considerations.

Section 2.11.2. Evaluation Criteria

Section 1.2 lists national income, regional income, environmental quality, and social well-being as the four objectives commonly defined in water resources studies. As discussed in Section 2.3, objectives are broad generalizations that must be expressed in specific measurable quantities before meaningful comparisons of alternative plans can be made. Criteria are defined which serve as a measure of how well objectives are satisfied. When it is not desirable or possible to define criteria that measures the full range of possible system states, specific targets or needs are often established. Alternative system comparisons are then made by either counting the number of times a specified target level is not satisfied, or by specifying a special measure such as a shortage index.

Contributions towards the national or regional income objective have been measured in a variety of ways, but two methods are quite common. The simplest method is by least cost. The cost of each alternative system which meets the desired needs is compared and the alternative with the least cost is said to contribute the most to the economic objective by meeting the needs at least cost. A second method used extensively in water development in the United States is to measure how well various needs are met in economic terms, referred to as benefits, and to compare benefits with economic costs. The system with the maximum net benefit, determined by subtracting costs from benefits, is considered the one contributing the most to the economic objective. When benefits of a system can be measured or approximated in economic terms, the benefit-cost method is an effective way of evaluating the economic contributions of alternative systems.

Evaluation of an environmental objective to preserve, conserve or enhance the natural environment involves measuring the beneficial and adverse effects of reservoir systems towards this objective. Beneficial effects are those which contribute towards the objective, adverse are those that take away from the objective. Criteria for evaluation are usually a list of environmental features which the system affects. Examples include: the number of kilometres of stream added or lost; the area of land inundated; the number and kind of wildlife relocated; and the stock of fish available. By identifying the specific environmental features which are affected and assessing them in quantitative terms, if possible, the magnitude of their impact on the environmental objective is determined. Contributions of alternative systems to this objective can then be displayed and compared.

In a manner similar to that described for environmental objectives, the features related to health and safety are identified, listed, and, if possible, quantified. Some examples might include: the number of persons relocated; the number of communities disrupted; dependability of providing the minimum required water supply; and public health hazards prevented or caused. Alternative systems can then be evaluated according to their contributions to the various social objectives.

Several techniques are available for evaluating the consequences of failing to meet target demands. In situations where shortages occur, the fewest number, or shortest period or the smallest magnitude of shortage over the period of analysis could be used. A shortage occurs when the flow or condition is less than the desired or required one. For example, for needs such as water supply, low-flow augmentation, stream recreation, irrigation or navigation, the evaluating criteria used may be the number of time periods when the actual flow is less than the desired or required flow, or the number of successive time periods when a shortage occurs, or the magnitude of the shortage. The sequence of shortages is particularly important for irrigation needs. Reservoir recreation is more difficult to express in economic terms, therefore reservoir elevations during the recreation season may be used as a measure. A high and stable water elevation is desired, so the evaluation criterion is often the amount and frequency of drawdown from a desired elevation. Figure 2.7 shows typical

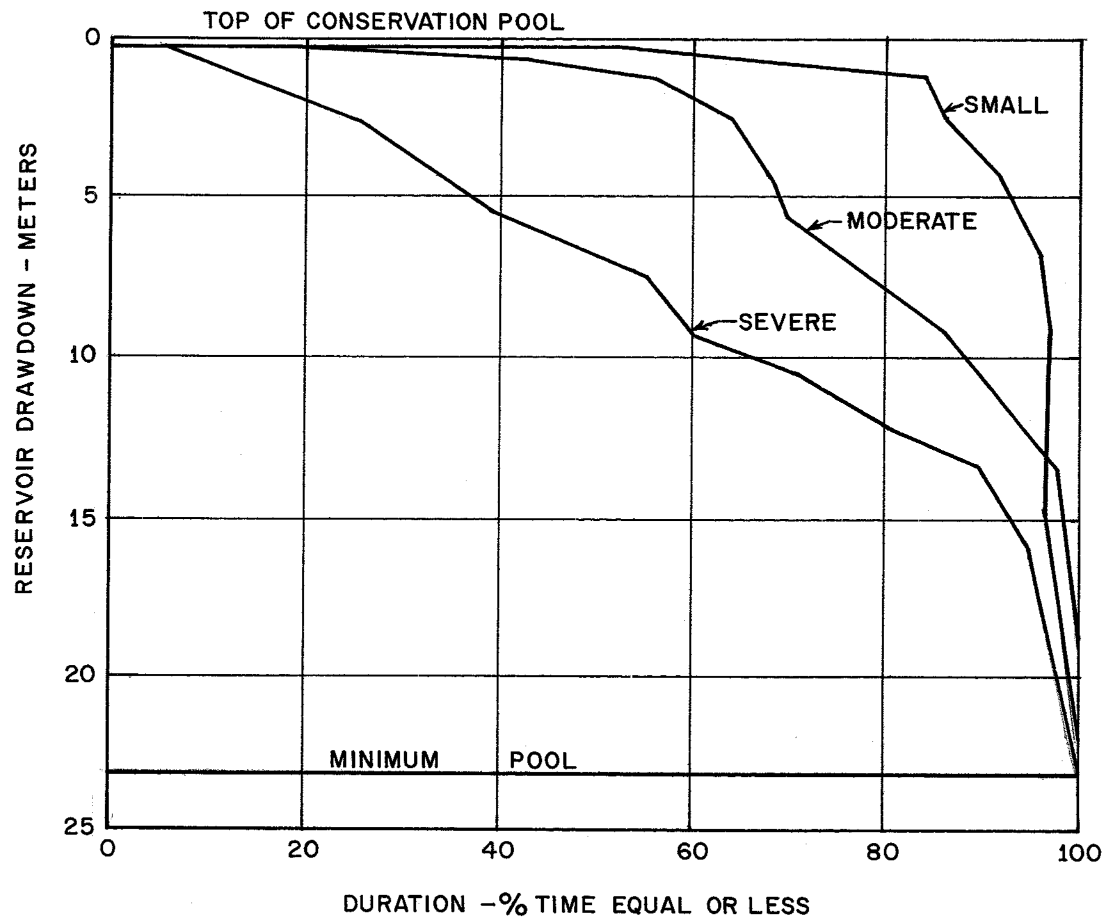


Figure 2.7 Reservoir Drawdown Duration Curves

small, moderate and severe drawdown conditions. The curves were developed simply by counting the number of time periods during the period of analysis the reservoir was within selected ranges.

Section 2.11.3. Evaluation Functions

Once a complete list of the essential evaluating criteria has been compiled, as outlined in Section 2.11.2, each criterion must be related to the hydrologic states that occur in the system. If a monthly time increment is used in simulation, these states would include monthly reservoir releases and stages, monthly diversion quantities and any other hydrologic quantities associated with the operation of the system. As a simulation run for a particular system configuration and operating procedure is performed, a set of system states is produced for each month of operation. By knowing the relationship between system states and the criteria which measure how well study objectives are fulfilled, the performance of each system can be quantified and subsequently compared.

Typically value functions or relationships are defined to relate the system states to criteria. In the simplest case, the dollar value of an irrigation diversion might be directly related to the quantity of water supplied. Figure 2.8 shows a typical curve demonstrating that water shortages have severe economic consequences and surpluses are of little

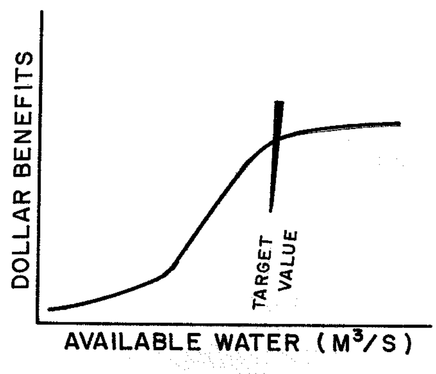


Figure 2.8 Irrigation Value Function

value. A more complex relationship could be developed for firm power production which depends on both the reservoir elevation and release. In this case power production could be specified as a criterion rather than attempting to relate power production to a specific dollar value. Figure 2.9 illustrates general shapes of several value functions for various purposes. Instead of denoting specific criteria on the vertical axis, general benefit values are displayed.

When target values are desired, evaluation is based on whether or not the particular value has been satisfied. One common measure is the annual shortage index which reflects both the number and magnitude of shortages. Section 5.07 of Volume 8 details the procedure for calculating this index.

With the exceptions discussed in the following section, the primary evaluation of a water resource system operation for conservation purposes can be accomplished by use of a monthly simulation study if the average annual values attached to each target service are modified by the average annual shortage costs. This applies to all conservation services, but special consideration must be given to hydropower services as discussed in Section 2.11.5. Shortages in recreation services can be related to a complex set of variables, as discussed briefly in Section 2.11.4.

Section 2.11.4. Critical Period Analysis

Because economic and social impacts increase greatly with the degree of shortage, one or two periods of extreme shortage will ordinarily dominate in the evaluation. It is frequently satisfactory to study only one critical period during an entire sequence in order to develop an initial design or operation scheme. The critical period is defined as the period between the time that the system has last wasted substantial amounts of water, until the time that the system again wastes substantial amounts of water, during which time the system reserves have been at a

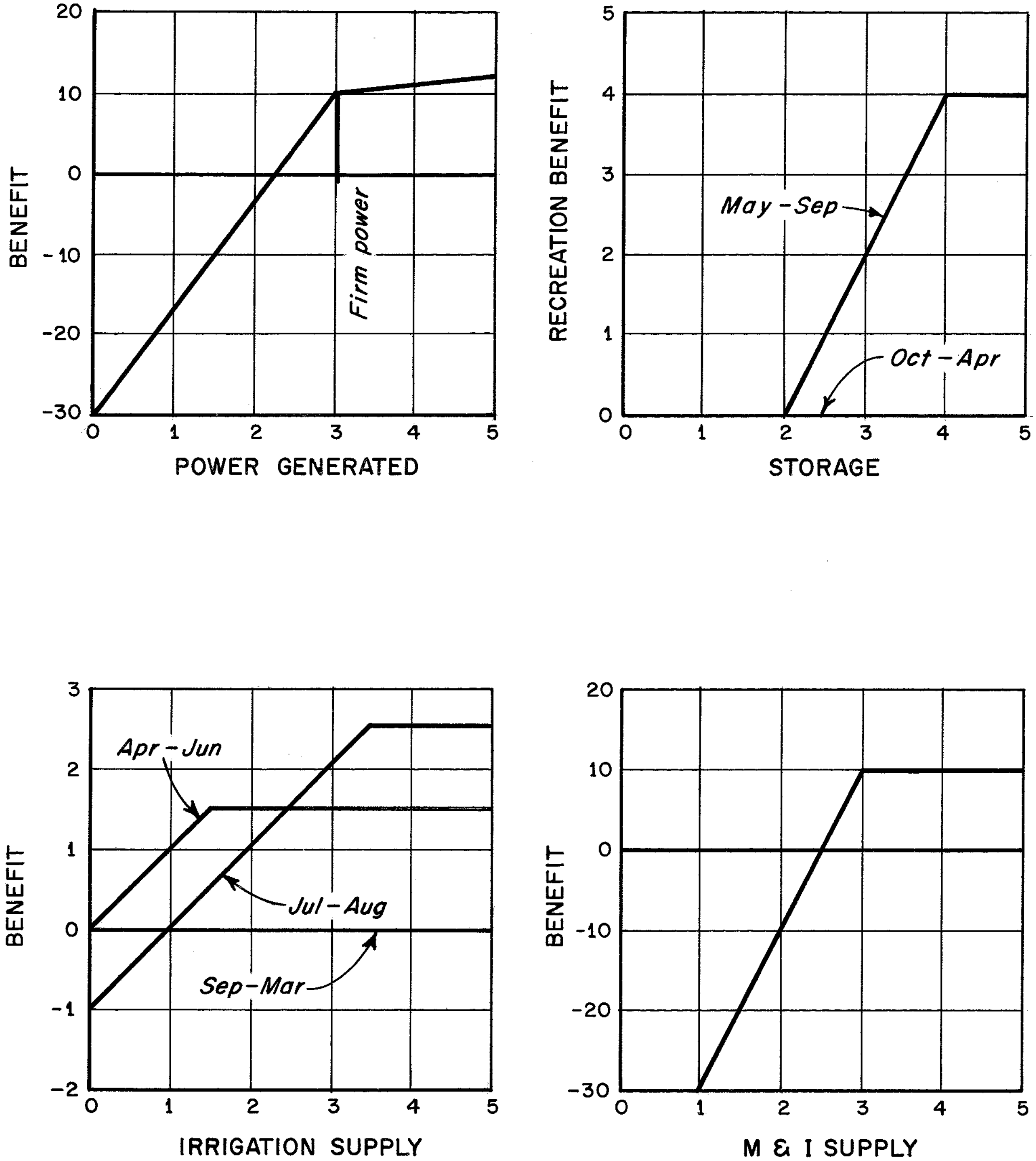


Figure 2.9 Typical Simplified Benefit Functions

minimum. The critical period does not necessarily start with all reservoirs full to the bottom of the flood-control reserve, because it is possible that some reservoirs start to draw down before others are completely full. Definition of the start of the critical period must be made with care.

When using computer techniques and the approximate nature and operation rules of the system are known, the critical period can be determined for each hydrologic sequence by making a monthly operation study. It can then be isolated for special optimization studies, thus saving a large amount of computation for each iteration. The operating rules and storage allocations are studied during the critical period to determine whether the design or operation scheme may be altered to improve the system's performance. The sensitivity of system shortages to changing sizes of system elements, changing operation rules or changing target services can be measured by examining relative supply and demand quantities during the critical period. When an optimum condition is obtained on this basis, the entire sequence or set of sequences of events must again be studied to determine whether critical periods have changed and whether events outside of the critical periods would greatly affect the optimization process.

The period of record is usually used as the sequence of events to simulate the system. However, synthetic sequences are also useful to prevent tailoring the design of the system to the worst drought of record. If the historic record is not extensive, the probability of experiencing a more critical drought or a less severe one over a longer duration is high. Incorporation of synthetic sequences in the testing procedure helps insure that the final design will perform well under a broader range of possible future flows. A review of these evaluations might indicate that a change in design or operation should be made.

After the critical period has started, continuous records must be kept of the accumulated demands, accumulated runoff and available storage above each critical supply point. If a shortage has not occurred by any particular time, the amount of water in storage can be divided by the supplies that have been furnished above each point to obtain an approximate ratio by which the supplies could have been increased without causing a shortage by that time. The minimum quantity thus obtained during the entire critical period could be used as the approximate amount by which reservoirs could be reduced in size.

If water shortages occur, the accumulated critical-period shortage at any time can be divided by the supplies that have been demanded above each point during the critical period up to that time in order to obtain an approximate ratio by which the demands must be reduced in order to prevent the shortage. The minimum demands thus obtained during the entire critical period could be used as the approximate amount by which the reservoirs should be increased in size in order to provide the target services.

The general logic discussed in the two preceding paragraphs can be used to establish by iteration the desired level of services and degree of shortages specified, or that would maximize a specified value function.

In addition, when evaluating and analyzing system output, particular attention should be given to continuity of services. In agriculture, for example, a severe shortage for a short time might be far worse than the same volume of shortage distributed over a longer period. In hydroelectric power generation, shortages must be made up by stand-by thermal or other generation facilities, and these are expensive to maintain in a ready condition. Hence continuity in power generation services is of great importance. In lake recreation, fluctuations of lake levels and lake temperatures can be detrimental even though the levels remain within acceptable limits for other supply purposes. Releases for navigation can sometimes be scheduled in order to maintain continuity even during drought periods.

Section 2.11.5. Special Hydropower Considerations

Power demands fluctuate rapidly, and in most systems hydropower is used to meet the most rapid demand fluctuations. In systems where thermal power predominates, hydropower might be used at a very high rate for only a few hours during the day. Consequently, power shortages can occur, even when sufficient water is available, if the combinations of head and turbine discharge capacity are insufficient at the right time to provide the peak power demand placed on the hydropower system.

It is possible in a monthly operation study to test the rate of energy generation capability against power load curves and to assure that the entire hydropower load curve can be satisfied. However, short interval operation study of power systems may be required to consider fluctuations in daily or even hourly power requirements.

For each month the hydropower load curve must be specified in tabular form. It is important to specify some relatively short durations in order to represent the very high peak demands that must be satisfied. For example the shortest duration may need to be 1 or 2 percent of a month. Other durations could be more widely spaced, such as 5 or 10 percent, 25 percent and 100 percent. Shortages in peaking-power quantities must be determined and evaluated for an adequate evaluation of hydropower benefits.

Chapter 3

Example Basin Study

CHAPTER 3. EXAMPLE BASIN STUDY

Section 3.1 Introduction

The intent of this chapter is to present an illustrative example describing the necessary steps to construct and perform a reservoir system operation study. As discussed in Chapter 2, the formulation of a simulation model may be divided into three steps: formulation of system hydrology; formulation of system components; and formulation of operating rules and regulations. An example from the White River basin consisting of a three reservoir system and one downstream control point, shown in Figure 3.1, is used to illustrate these concepts.

Section 3.2 Assembly of Hydrologic Data

The hydrologic data describe the amount of water that is available in the basin. As outlined in Section 2.6, hydrologic data composed of reservoir inflows, local inflows at control points, and reservoir evaporation are necessary in the formulation of a simulation model. The selection of the proper computational interval for these variables in the simulation is also an important consideration.

For conservation analysis relatively long time periods may be used. The time interval should be chosen so that seasonal variations in both available water and demand may be adequately represented. Too long of a computational interval gives poor definition of these time variable quantities. Too short of an interval unnecessarily increases the volume of data preparation and cost of analysis. The example in this chapter assumes that data is in a monthly time increment and represents average values during that computational interval.

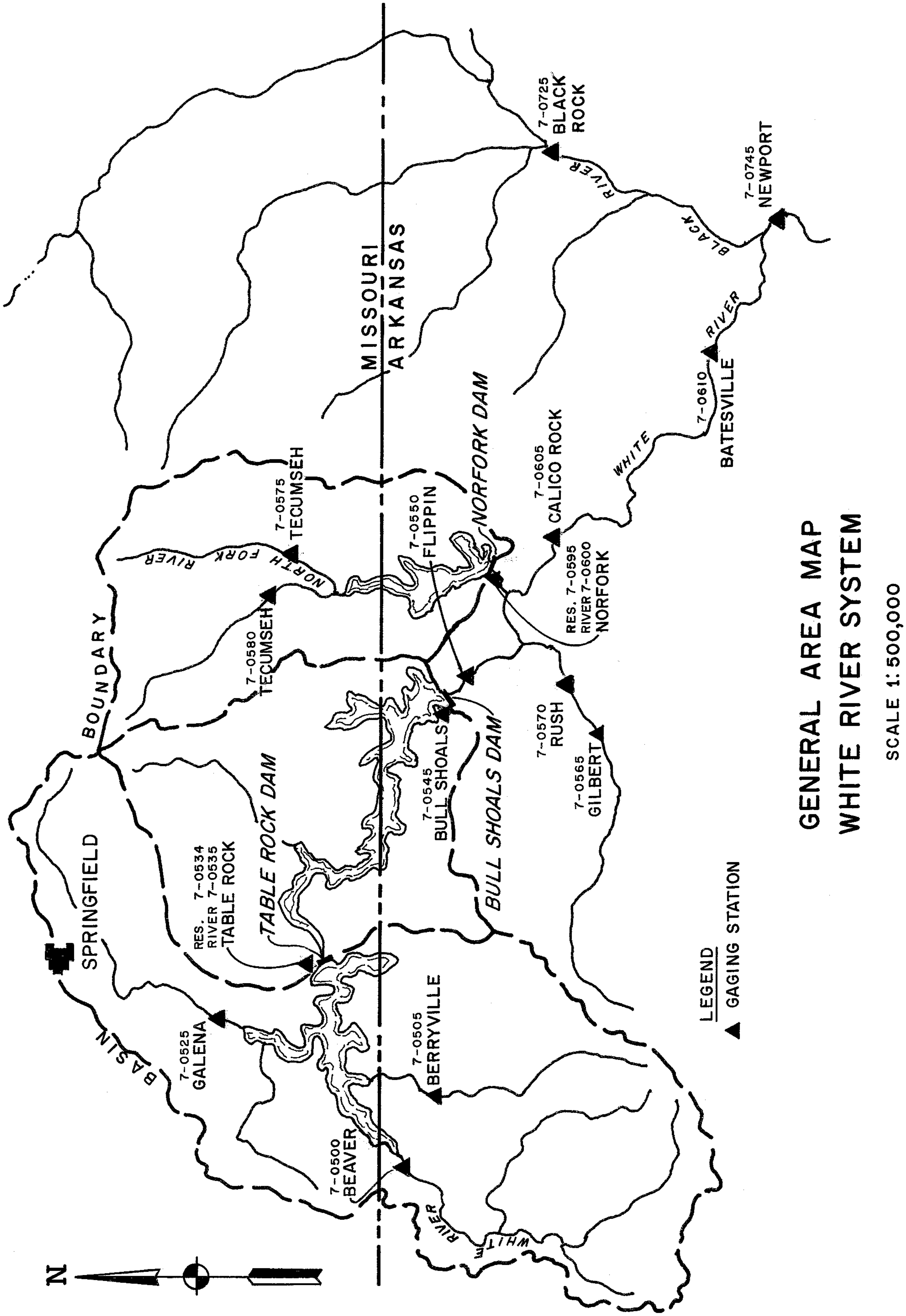

GENERAL AREA MAP
WHITE RIVER SYSTEM
SCALE 1:500,000
LEGEND
GAGING STATION
N
MISSOURI
ARKANSAS
BASIN
BOUNDARY
SPRINGFIELD
7-0500 BEAVER
7-0505 BERRYVILLE
7-0525 GALENA
RES. 7-0534 RIVER 7-0535 TABLE ROCK
TABLE ROCK DAM
7-0545 BULL SHOALS
BULL SHOALS DAM
7-0550 FLIPPIN
7-0565 GILBERT
7-0570 RUSH
7-0575 TECUMSEH
7-0580 TECUMSEH
NORTH FORK RIVER
RES. 7-0595 RIVER 7-0600 NORFORK
NORFORK DAM
7-0605 CALICO ROCK
7-0610 BATESVILLE
7-0725 BLACK ROCK
7-0745 NEWPORT
WHITE RIVER
BLACK RIVER

Figure 3.1

Section 3.2.1 Streamflow Data

Streamflow data should be assembled for all of the gaging stations in the basin that have records of sufficient length to be of use in the study. Figure 3.1 shows the gages in the vicinity of the three reservoirs and Table 3.1 illustrates typical monthly and annual streamflow information for one gage. During a study this data must be converted into monthly reservoir inflows and local inflows in the proper units as shown in Table 3.8, according to the procedures discussed in Section 2.6. The inflows for the 14-month period from March 1953 to April 1954 are shown in columns 3, 10, 19, and 28 of Table 3.8. The flow at the Table Rock and Norfork sites represents the inflow into the reservoirs. The flow into Bull Shoals and Batesville is local inflow, or that flow contributed by the drainage area between the upstream reservoir(s) and the location being considered. The period of record should ideally cover time spans when hydrologically severe conditions existed. The data in Table 3.8 was developed for mid 1953 through 1954 which was a severe drought period in the White River region.

Section 3.2.2 Project Loss Data

Reservoir evaporation is usually computed from meteorologic information or measured directly. When performing simulation studies average monthly values are often employed that remain constant from year to year. Figure 2.5 shows average monthly net evaporation for the White River basin. Evaporation is not considered in the example in this chapter in order to simplyfy the calculations.

Section 3.3 Assembly of System Demands and Flow Constraints

The system demands describe the distribution of water desired in the basin. These demands vary both in time and from location to location.

610. White River at Batesville, Ark.

Location.--Lat 35°45'37", long 91°38'28", in NE¼ sec.21, T.13 N., R.6 W., on left bank at downstream side of bridge on State Highway 11 at Batesville, 0.3 mile upstream from lock and dam 1, 0.6 mile downstream from Polk Bayou, and at mile 300.1.

Drainage area.--11,062 sq mi.

Records available.--July 1937 to September 1958. Gage-height records collected at lower lock gage since 1904 are contained in reports of U. S. Weather Bureau.

Gage.--Water-stage recorder and concrete dam. Datum of gage is 237.72 ft above mean sea level, datum of 1929. Prior to Jan. 28, 1939, staff gage on upper lock wall of dam 1, 0.3 mile downstream at same datum.

Average discharge.--21 years (1937-58), 12,360 cfs (8,948,000 acre-ft per year).

Extremes.--1937-58: Maximum discharge, 324,000 cfs Apr. 16, 1945 (gage height, 29.43 ft); minimum, 580 cfs Sept. 28, 1954; minimum daily, 592 cfs Sept. 28, 1954.
Maximum stage known, 31.1 cfs Feb. 1, 1916, at former site, observed by employee of Corps of Engineers (discharge, 382,000 cfs).

Remarks.--For regulation see Remarks for station at Calico Rock on preceding page.

Monthly and yearly mean discharge, in cubic feet per second, of White River at Batesville, Ark.

Water year	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	The year
1937	-	-	-	-	-	-	-	-	-	4,413	2,260	3,810	-
1938	3,165	2,935	7,846	12,960	51,540	20,890	27,720	30,450	15,390	3,571	2,471	1,567	14,860
1939	1,410	8,695	3,604	7,496	27,640	23,760	33,580	30,920	11,440	9,294	2,605	1,547	13,380
1940	1,556	2,246	2,315	2,555	2,987	5,579	23,960	9,910	3,755	3,125	3,017	2,054	5,237
1941	1,652	2,589	7,143	16,990	10,160	5,697	23,760	6,563	3,478	2,068	1,815	3,714	7,097
1942	19,410	28,150	13,560	9,229	17,090	13,420	25,090	19,390	9,235	3,025	4,023	3,428	13,700
1943	2,245	27,740	32,380	15,740	4,974	9,209	15,920	71,230	15,450	2,930	1,504	912	16,800
1944	1,369	1,730	1,640	2,652	9,656	25,640	18,650	11,210	7,706	1,893	1,953	1,760	7,138
1945	2,753	1,623	3,323	2,454	28,370	72,740	100,400	35,600	53,690	10,920	3,003	5,164	26,510
1946	11,270	5,258	4,355	24,800	31,930	17,880	11,960	41,950	16,450	4,501	3,604	2,201	14,600
1947	1,958	28,600	29,730	9,560	6,097	4,812	21,950	22,100	10,080	4,695	2,380	2,073	12,010
1948	2,015	2,677	3,922	9,790	10,350	26,150	10,430	8,950	13,430	9,193	8,503	3,324	9,072
1949	2,311	3,466	8,605	45,000	44,790	18,530	12,060	13,700	12,880	13,040	3,593	3,987	15,000
1950	7,790	4,267	6,513	42,820	32,520	16,080	18,460	49,080	17,860	11,570	14,000	15,950	19,700

Monthly and yearly mean discharge, in cubic feet per second

Water year	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	The year
1951	5,025	4,209	3,534	7,939	29,270	27,680	19,530	11,200	12,890	24,100	3,872	3,381	12,610
1952	3,432	16,370	22,220	15,020	8,901	23,810	32,750	12,700	4,036	3,250	2,774	2,224	12,290
1953	2,030	5,979	6,086	6,195	6,346	18,110	17,600	24,880	7,329	5,814	5,583	5,800	9,339
1954	6,113	5,944	4,034	6,600	6,388	6,168	7,794	9,525	5,026	4,291	4,685	1,480	5,671
1955	1,224	1,587	3,125	2,485	5,035	10,460	9,587	9,230	11,510	10,150	6,908	6,049	6,449
1956	4,373	4,564	4,625	4,603	14,460	6,420	7,076	7,809	6,543	6,413	5,950	5,697	6,508
1957	4,490	4,943	5,436	5,886	9,328	11,240	34,470	30,090	26,670	29,620	25,860	24,680	17,740
1958	13,760	11,760	7,952	9,758	7,912	17,740	21,040	22,260	17,190	13,300	14,420	8,331	13,820
1959													
1960													

Yearly discharge, in cubic feet per second

Year	W.S.P. no.	Water year ending Sept. 30					Calendar year	
		Momentary maximum		Minimum day	Mean	Runoff in acre-feet	Mean	Runoff in acre-feet
		Discharge	Date					
1937	857	-	-	-	-	-	-	-
1938	857, 877	260,000	Feb. 19, 1938	1,340	14,860	10,760,000	14,820	10,730,000
1939	877	165,000	Apr. 18, 1939	1,100	13,380	9,686,000	12,750	9,232,000
1940	897	93,600	Apr. 12, 1940	1,350	5,237	3,802,000	5,683	4,126,000
1941	927	114,000	Apr. 22, 1941	1,220	7,097	5,138,000	11,250	8,146,000
1942	957	122,000	Nov. 1, 1941	1,840	13,700	9,916,000	13,800	9,993,000
1943	977	281,000	May 12, 1943	780	16,800	12,160,000	11,980	8,672,000
1944	1007	54,800	Mar. 1, 1944	800	7,138	5,182,000	7,389	5,365,000
1945	1037, 1177	324,000	Apr. 16, 1945	956	26,510	19,190,000	27,620	19,990,000
1946	1057	106,000	Feb. 15, 1946	1,350	14,600	10,570,000	17,880	12,950,000
1947	1087	114,000	Dec. 13, 1946	1,100	12,010	8,696,000	7,693	5,570,000
1948	1117	73,900	June 20, 1948	1,130	9,072	6,586,000	9,558	6,938,000
1949	1147	236,000	Jan. 26, 1949	1,460	15,000	10,860,000	15,350	11,110,000
1950	1177	216,000	May 13, 14, 1950	2,810	19,700	14,260,000	-	-

Yearly discharge, in cubic feet per second

Year	WSP	Water year ending Sept. 30					Calendar year	
		Momentary maximum		Minimum day	Mean	Acre-feet	Mean	Acre-feet
		Discharge	Date					
1950	-	-	-	-	-	-	19,210	13,910,000
1951	1211	107,000	Feb. 21, 1951	1,960	12,610	9,132,000	15,060	10,910,000
1952	1241	77,700	Mar. 12, 1952	1,570	12,290	8,923,000	9,955	7,227,000
1953	1281	63,500	Mar. 18, 1953	1,180	9,339	6,761,000	9,508	6,884,000
1954	1341	47,900	May 2, 1954	592	5,671	4,106,000	4,821	3,490,000
1955	1391	58,500	Mar. 22, 1955	738	6,449	4,669,000	7,088	5,132,000
1956	1441	45,700	Feb. 18, 1956	2,210	6,508	4,724,000	6,617	4,804,000
1957	1511	124,000	Apr. 4, 1957	1,360	17,740	12,840,000	19,300	13,970,000
1958	1561	56,100	May 9, 1958	5,190	13,820	10,010,000	-	-
1959								
1960								

TABLE 3.1. Monthly and Annual Streamflow Values for the White River at Batesville, Arkansas (From USGS Water Supply Papers 1131 (1950), 1731 (1960)).

It is usually necessary to express these demands on a seasonal basis. These values should be available in the appropriate units, and should represent average values over the computational time interval. If the system demands change significantly with time, it may be necessary to make complete simulations for the period of record for each different demand schedule.

In the case of the White River example, a single monthly average demand schedule is assumed. Demands are specified below two of the reservoirs and at Batesville, the downstream control point, where a distinction is made between minimum required and minimum desired flows. In addition three diversions are specified as shown in Figure 3.2, a schematic diagram of the reservoir system.

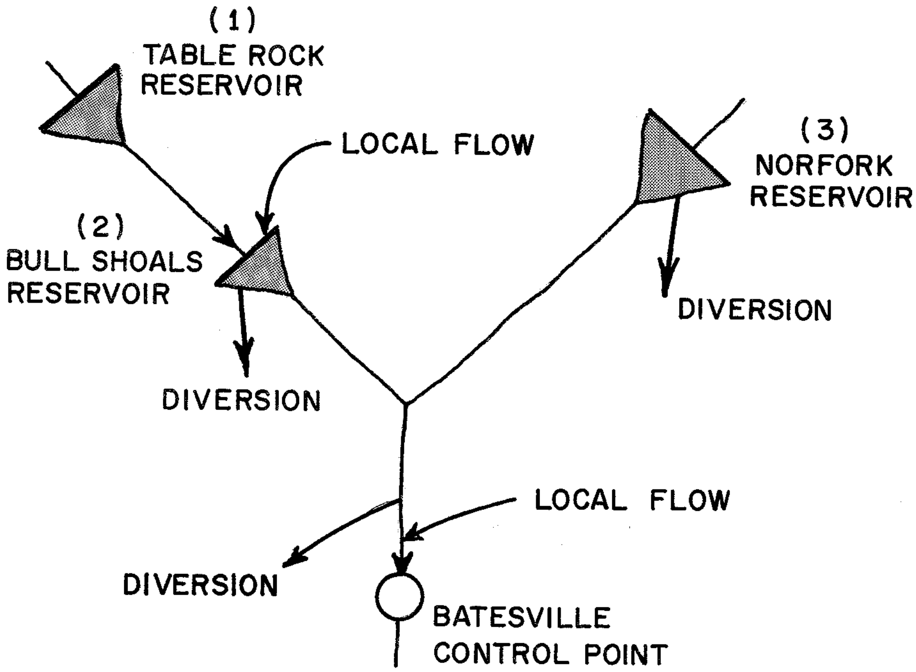


Figure 3.2 Schematic Diagram of Three Reservoir System

Section 3.3.1 Minimum Desired Flows

Minimum desired flows are streamflows that are desired at points in the river system. These flows will only be met when the reservoir level is between the top of the buffer pool and the top of the conservation pool. Typical uses of desired flows are for recreation, enhanced water quality, and fish and wildlife.

If sufficient streamflow and storage exist in the White River system, it is desired to maintain a flow higher than the minimum required flow of 85 M^3/S at Batesville during some of the months to reduce water quality problems. The monthly minimum desired flow is shown in Table 3.2.

Table 3.2 Minimum Desired Flow at Batesville (M^3/S)

Month	Desired Flow	Month	Desired Flow	Month	Desired Flow
Jan	85	May	127	Sep	127
Feb	85	Jun	127	Oct	85
Mar	113	Jul	142	Nov	85
Apr	113	Aug	142	Dec	85

When the storage in the reservoir system goes below a specified buffer storage, explained in Section 3.4.1, the higher desired minimum flow is reduced to the minimum required flow discussed in the following section.

Section 3.3.2 Minimum Required Flows

Minimum required flows are among the highest priority flows that the reservoir system should be able to provide at all times. These flows should be supplied as long as any releases can be made from the system.

These are the only releases that are made when a reservoir is below the top of the buffer pool. Typical uses of required flows are municipal and industrial water supply, power and under certain conditions perhaps water quality and navigation. The minimum required flows for this example are shown in Table 3.3.

Table 3.3 Minimum Required Flows

	Minimum Required Flows (M^3/S)
Table Rock	None
Bull Shoals	14
Norfork	11
Batesville	85

Section 3.3.3 Maximum Permissible Flows

Maximum permissible flows are those flow values that would cause flooding and/or subsequent damages if the values were exceeded. This is usually not an important consideration for conservation studies performed using monthly data. As shown in Table 3.8 the maximum permissible flows, which are normally non-damaging channel capacity, contained in Table 3.4, are not approached in the short period analyzed in this example. However, there are situations, especially at large-volume reservoirs, where the evacuation of the flood control storage may take one or more months, and the maximum permissible flow would be a constraint on the reservoir operation.

Table 3.4 Maximum Permissible Flows

	Maximum Permissible Flow (M^3/S)
Table Rock	None
Bull Shoals	380
Norfork	255
Batesville	765

Section 3.3.4 Diversion Demands

Diversion demands are extractions from a reservoir and/or a stream location. These demands have the same priority as the minimum required flows. Water diverted may or may not return to some other location in the system. The diversion could be a complete extraction by transferring water out of the stream system being studied. The diversion could be a transferral of water within the present system, such as flow from one subbasin into another subbasin. The diversion could also be an extraction of water for some consumptive use with a portion of the diversion being returned to some other point in the system. As an example, a municipal or industrial requirement would usually be treated as a diversion because between 50 to 80 percent of the demand is consumed by the entity requiring water and the remainder returns to the system. Irrigation requirements are also treated as diversion demands and the consumptive use usually ranges from 75 to 100 percent.

There are three diversion requirements in the example system. One diversion is at the Bull Shoals Reservoir, another diversion is at the Norfork Reservoir, and the third diversion is immediately upstream of the Batesville control point. The diversion requirements are shown in the Table 3.5. There are no return flows from the diversions.

Table 3.5 Diversion Requirements

Diversion in M^3/S

Month	Bull Shoals	Norfork	Batesville
June	8.5	6	8.5
July	18	7	18
August	28	9	28
September	21	7	21
October	7	2	7

*The diversion is zero for months not shown.

Section 3.3.5 Schedule of Energy Demands

Hydroelectric energy demands dictate the release of water at projects with power plants. Hydroelectric power projects that are used for peaking power must be simulated in a way to express the average quantity of water released during the routing time period. Hydroelectric power plants can sometimes be included in a large power system that has an energy requirement greater than the sum of the at-site energy requirements. The flow released for generation of hydroelectric energy is nonconsumptive and can be used for any of the downstream diversion or flow demands.

In order to keep calculations relatively simple, hydroelectric power production is not considered in this example. Section 6.03 of Volume 8 contains example computations from a reservoir routing with a schedule of hydroelectric demands.

Section 3.4 Assembly of Reservoir Data

There are several parameters for reservoirs that must be specified to represent the physical characteristics of the reservoirs and to describe the criteria under which the reservoirs operate. It is usually necessary to provide storage volume, surface area, outlet capacity, and elevation tables for each reservoir. The volume of storage to be allocated to each of the reservoir storage levels must also be specified.

Section 3.4.1 Reservoir Storage Levels

The volume of storage to be allocated to each of various reservoir storage levels must be specified in order to operate the reservoirs in a system (Figure 2.1). Typically, three storage zones should be defined:

(1) Buffer zone (between top of inactive pool and top of buffer)
(2) Main conservation zone (between top of buffer and top of conservation)
(3) Flood control zone (between top of conservation and top of flood control pool)

The level of the reservoir relative to these zones will determine what reservoir releases will be made. That is, if the reservoir is in the flood control zone, flood control releases should be made. If in the main conservation zone, minimum desired flows should govern. If in the buffer zone, only minimum required flows should be met.

Where a system exists such that more than one reservoir can meet a particular flow requirement, releases should be made so that all reservoirs are kept in a relative state of balance. One approach is to keep all reservoirs the same percent full within each zone. Thus, if two reservoirs could provide a minimum desired flow at a demand point and both were in the main conservation zone, releases would be divided between the two projects so that the same percent of the conservation storage would remain (e.g. 82%) at the end of the computation period. In more complex situations, it is often desirable to use a certain portion of the conservation storage in one reservoir before using any of the conservation storage of a smaller reservoir. In this case, two conservation zones may be defined for the larger reservoir such that releases can be made only from the higher conservation zone in the larger reservoir until it is depleted. Then proportional releases can be made from both reservoirs thereafter. In a similar way, buffer zones can be defined to meet minimum required flows on a priority basis among several reservoirs.

These storage levels may be fixed for the entire analysis, or they may be varied with the season of the year. In areas where precipitation is uniformly distributed throughout the year and demands are uniformly distributed as well, storage levels should be constant throughout the year. However, if a seasonal variation in supply and demand exists, it may be appropriate to vary the volume in each storage zone, as illustrated in Figure 2.2.

Table 3.6 shows the allocation zones for the three reservoirs in the White River system. For this example these allocations are assumed to remain constant throughout the year.

Table 3.6 Storage Allocations

	Accumulated Storage $M^3 \times 10^6$		
Reservoir (No.)	Top of Inactive	Top of Conservation	Top of Flood Control
Table Rock (1)	1980	3330	4210
Bull Shoals (2)	2590	3760	6670
Norfork (3)	970	1540	2440

The initial storage in each reservoir project must also be specified at the beginning of the computations. If the purpose of the study is to simulate the operation during a historic low-flow period, it is important to begin the computations either at a time when the reservoirs would have a full conservation pool or with a realistic beginning storage. Columns 8, 15 and 24 of Table 3.8 contain starting storages for the reservoir system simulation. These storages reach the top of the conservation pool during the second and third computational intervals.

Section 3.4.2 Storage, Surface Area, Outlet Capacity, Elevation Data

Specific data on reservoir geometry and outlet works capacity may be needed. If reservoir losses due to evaporation are significant, it will be necessary to know the reservoir surface area that corresponds to reservoir storage. In such cases a complete table of reservoir area versus storage would be needed covering the range of all storages from the bottom to the top of flood control pool. This would be obtained from survey data of the reservoir site.

In addition, if outlet capacity would constrain releases to be less than minimum desired or required flows for any storage levels, an outlet capacity versus storage table would be needed.

Where power is being generated, elevations are needed to calculate the head on the turbines as well as the power releases and the power generated. Thus, power applications would require an elevation versus storage table.

Because this example does not include the computations for evaporation or hydroelectric energy area and elevation data are not required in the computations.

Section 3.4.3 Operation Criteria

In addition to the physical characteristics describing the components, the rules for operating each reservoir in the system must be specified. For instance, it is necessary to specify which downstream demands and flow requirements the particular project is required to supply or assist in supplying. When the reservoir system contains many projects, a decision must be made as to which projects will make releases to supply a particular purpose. This specification can be accomplished by using storage level designations, described in Section 3.5.1, which keep the storage levels between projects in balance in so far as possible during the analysis.

The following operation criteria has been established for the White River system:

a. The buffer level, or that level below which only the minimum required flow will be provided, in Bull Shoals and Norfork reservoirs is one-fourth of the usable conservation storage. The Table Rock reservoir does not contain any buffer storage.

b. At the Bull Shoals reservoir, one-half of the conservation storage above the buffer level will be used before withdrawing water from Table Rock or Norfork reservoirs to meet the flow requirements at the Batesville control point.

c. When the storage specified in "b" is used, then the conservation storage in Norfork can be withdrawn to one-half the storage above the buffer level while the Table Rock reservoir storage can be emptied and the Bull Shoals reservoir can be drawn to the top of the buffer level.

d. The remaining storage above the buffer level in Norfork reservoir will be used to meet system requirements before using any of the buffer storage in the Bull Shoals reservoir.

These represent only target levels and may be violated if flows at a given control point can only be met be making releases from one specific reservoir. The total reservoir storage below each of the target levels is given in Table 3.7.

Table 3.7 Cumulative Storage ($M^3 \times 10^6$)

Storage Level	Top of Pool	Table Rock	Bull Shoals	Norfork
6	Flood Control	4210	6670	2440
5	Conservation	3330	3760	1540
4		3330	3320	1540
3		1980	2880	1325
2	Buffer	1980	2880	1110
1	Dead	1980	2590	970

A graphic representation of the levels that should be assigned to reflect the priority of releases between reservoirs is shown in Figure 3.3. This figure shows the relative size of the active storage in each of the reservoirs as well as the amount of storage allocated to each purpose. The highest storage zone shown (area A) represents the flood control storage. For each reservoir, level 6 has been assigned to top of flood control and level 5 to bottom of flood control. Because equal level numbers are assigned, flood space will be evacuated concurrently from all three reservoirs, attempting to keep them in balance. Once all reservoirs have been evacuated to level 5 (top of conservation pool), releases should be made to bring all reservoirs to level 4 as required to meet water demands in the basin. As shown in Figure 3.3 with the level numbers that were assigned, to bring all reservoirs to level 4 would only require releases to be made from the Bull Shoals facility (area B). Then releases can be made to meet demands by withdrawing flows from all three sites (area C) until they have been lowered to level 3. At that time,

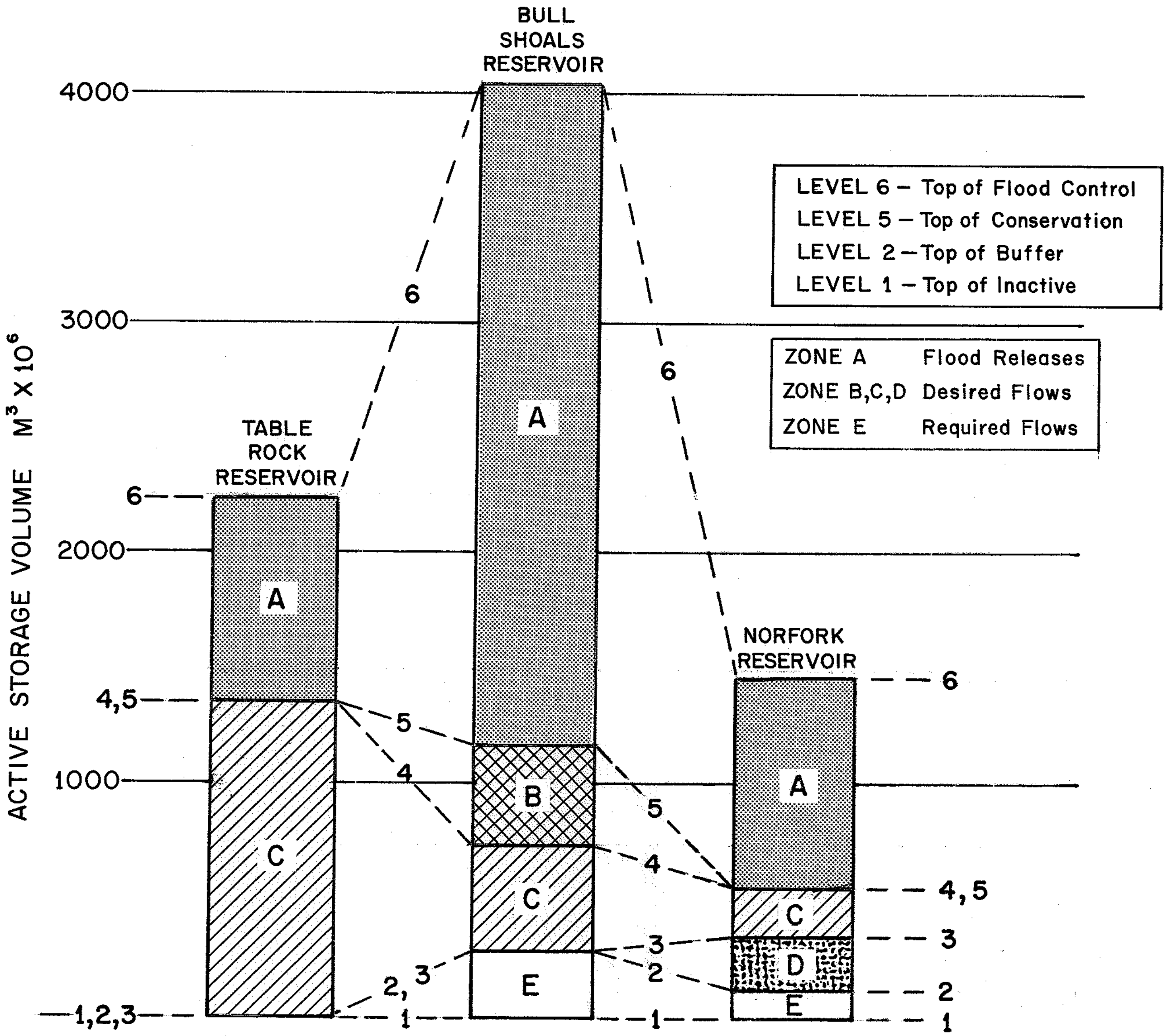


Figure 3.3 Active Reservoir Storage Allocation Zones and Index Levels

only releases from Norfork Reservoir would be made (area D). Finally, the buffer pools in Bull Shoals and Norfork (area E) would be utilized leaving all the active reservoir zones empty.

Section 3.5 Example Computations

Section 3.5.1 Analytic Solution for Releases with Balanced Storage Levels

In this section equations are developed to solve for regulated releases from both tandem and parallel reservoir pairs which maintain the storage balance within the system. Continuity and equivalent storage equations are combined with an assumed downstream demand to determine the releases.

Figure 3.4 shows how reservoir storage may be expressed in order to calculate balancing levels.

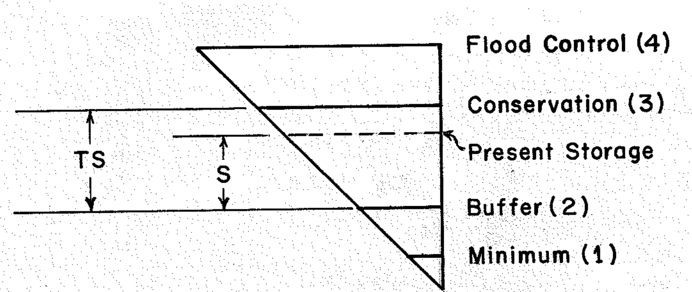


Figure 3.4 Example Reservoir Storage Designation

TS is the total storage within a particular zone. In this case TS would be the storage volume allocated to the conservation zone. S is the current amount of storage within the zone. As explained in Section

2.8.3 and shown in Figure 3.4, index numbers are assigned to the top level of each storage zone. The index number of a reservoir level can be computed by adding the index at the bottom of the zone to the ratio that represents the proportion of the zone that is currently filled.

$$\text{Index Level} = \text{Base Index} + \frac{S}{TS} \qquad (3\text{-}1)$$

Figure 3.5 illustrates typical inflows and outflows which might be found at a reservoir site.

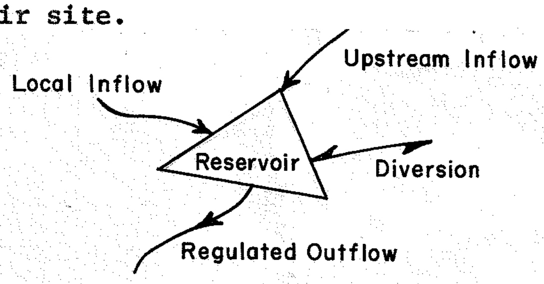


Figure 3.5 Typical Reservoir Inflows and Outflows

These flows may be combined into a continuity equation:

$$S = PS + QI + LF - QO \pm D \qquad (3\text{-}2)$$

where S = Storage within the zone at the end of the current period

QI = Upstream inflow for period, not including local flow into the reservoir.

LF = Local Inflow for period; generally used when QI is a regulated flow from an upstream reservoir.

QO = Outflow for period

D = Diversion for the period (-), or a return flow to stream (+)

Local flow terms are included in equation 3-2 in tandem reservoir analyses when the upstream inflow is regulated and far enough upstream that the intervening area contributes significant flow quantities. If there is no upstream reservoir then the local flow term is assumed to be zero, and all inflows included in the QI term. For the three reservoir system shown in Figure 3.2 the local flows into Table Rock and Norfork Reservoirs are assumed to be zero. Only the local flow term for the Bull Shoals Reservoir is defined with the upstream inflow QI set equal to the release from the Table Rock Reservoir.

Once a downstream demand is specified, Equations 3-1 and 3-2 may be combined to solve for the reservoir releases directly.

Parallel Reservoirs

When parallel reservoirs are being considered the downstream demand may be satisfied by any combination of QO_1 and QO_2 which adds up to the proper demand, QT, as shown in Figure 3.6.

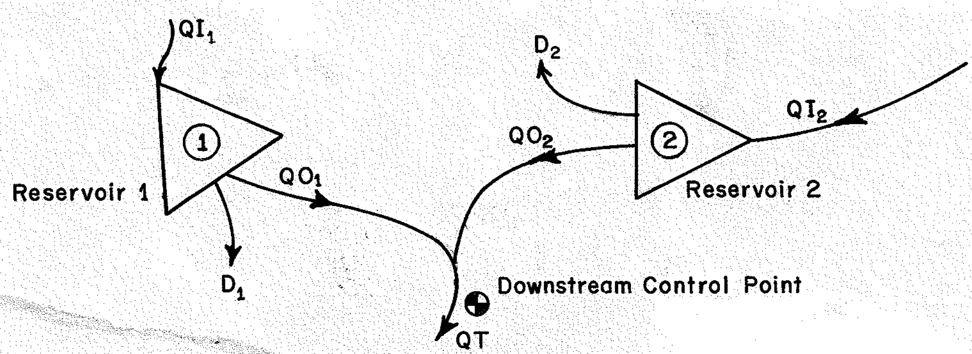


Figure 3.6 Parallel Reservoirs

QT is the amount of demand met by releases from the upstream reservoir(s). When local flows occur from the intervening area between the point(s) of reservoir release(s) and the downstream control point then the local flows must be subtracted from the demand at the control to determine QT. This occurs in the White River system where local flows above Batesville must be subtracted from the required flows at Batesville to determine the necessary amount of water to be released from Bull Shoals and Norfolk Reservoirs.

In equation form:

$$QT = QO_1 + QO_2 \tag{3-3}$$

for parallel reservoirs. In order to keep the two reservoirs balanced, the two index levels must be the same or:

$$\frac{S_1}{TS_1} = \frac{S_2}{TS_2} \tag{3-4}$$

Rearranging Equation 3-4 and substituting for the storages expressed in Equation 3-2 yields:

$$TS_2(PS_1+QI_1+LF_1-QO_1\pm D_1)=TS_1(PS_2+QI_2+LF_2-QO_2\pm D_2) \tag{3-5}$$

From Equation 3-3

$$QO_2 = QT - QO_1 \tag{3-6}$$

Then:

$$TS_2(PS_1+QI_1+LF_1-QO_1\pm D_1) = TS_1(PS_2+QI_2+LF_2-(QT-QO_1)\pm D_2)$$

$$QO_1(TS_2+TS_1) = TS_2(PS_1+QI_1+LF_1\pm D_1)-TS_1(PS_2+QI_2+LF_2-QT\pm D_2)$$

$$QO_1=\frac{TS_2(PS_1+QI_1+LF_1\pm D_1)-TS_1(PS_2+QI_2+LF_2-QT\pm D_2)}{TS_2+TS_1} \tag{3-7}$$

Since all the terms on the right hand side of Equation 3-7 are known, QO_1 can be calculated directly. QO_2, the release from reservoir 2, can then be determined using Equation 3-6.

Tandem or Series Reservoirs

In a series reservoir calculation the downstream demand must be satisfied from releases in reservoir 2, while releases in reservoir 1 are made to balance the system as shown in Figure 3.7.

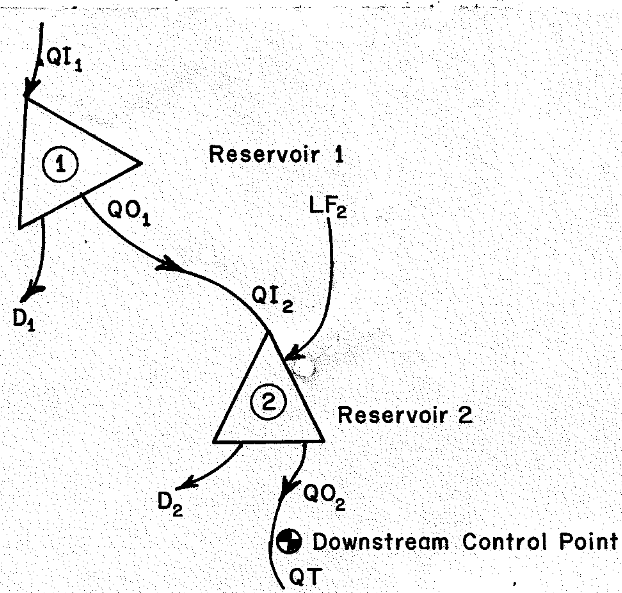


Figure 3.7 Tandem Reservoirs

For tandem reservoirs the downstream demand QT must be satisfied by the release from reservoir 2, and the release of reservoir 1 is equal to the upstream inflow to reservoir 2.

$$QT = QO_2 \tag{3-8}$$
$$QI_2 = QO_1 \tag{3-9}$$

Equations 3-4 and 3-2 hold for tandem reservoirs, hence Equation 3-5 may also be used. Equations 3-8 and 3-9 may be combined with Equation 3-5 to yield:

$$TS_2(PS_1+QI_1+LF_1-QO_1\pm D_1) = TS_1(PS_2+QO_1+LF_2-QT\pm D_2)$$

$$QO_1(TS_1+TS_2) = TS_2(PS_1+QI_1+LF_1\pm D_1)-TS_1(PS_2+LF_2-QT\pm D_2)$$

$$QO_1 = \frac{TS_2(PS_1+QI_1+LF_1\pm D_1)-TS_1(PS_2+LF_2-QT\pm D_2)}{TS_1+TS_2} \qquad (3\text{-}10)$$

Like Equation 3-7, all the terms on the right hand side of Equation 3-10 are known, and QO_1, may be solved directly.

In summary, when determining releases for parallel reservoirs Equations 3-6, and 3-7 are used. For tandem reservoirs Equations 3-8 and 3-10 are employed. When these equations are used in hand computations, care must be taken to avoid round-off errors, or the resultant releases will cause the reservoirs to be slightly out of balance. If this occurs small adjustments in the releases will bring the system back into balance.

If evaporation or hydroelectric demands are included in the reservoir analysis then the releases cannot be solved for directly. This is because the volume of water evaporated and the energy produced depend on the average reservoir area and depth respectively during the period, which are directly related to the reservoir's level. The average level requires knowledge of the ending reservoir level which depends upon the releases made. Hence an iterative procedure is required that: assumes an ending reservoir level; calculates the volume of evaporation lost and the volume of water necessary to satisfy the energy demand, determines reservoir releases; and computes a new ending reservoir level. This ending level is then used to recompute volumes of evaporation and water used for hydropower. The process is repeated until values of the ending reservoir levels do not differ significantly on successive calculations. Section 6.03 of Volume 8 contains a more complete discussion on the iterative procedure.

Section 3.5.2 Monthly Reservoir Operating Procedure

This section outlines the step by step procedure for determining monthly reservoir releases based upon the regulation criteria described in Section 3.4.3. The procedure outlined below operates in a sequential fashion, starting at the downstream control point at Batesville and working upstream. Table 3.8 illustrates the computations involved in a detailed sequential analysis. This table assumes streamflows that occurred from March 1953 to April 1954.

Step 1

This step determines whether water is stored or released during the month. The monthly system inflow is simply the sum of columns 3, 10, 19 and 28. The total amount of water desired in the system is found by adding columns 17, 26, 30 and 31. If the amount of water desired is greater than the monthly system inflow, water will be released. If the inflow is greater than the demand, water will be stored.

Step 2 (Zone 3-4, 5-6)

The next step is to determine releases for the three reservoirs. Figure 3-3 illustrates which reservoirs contribute to satisfy desired demands for the different storage allocation zones. When the current reservoir level is in zone 5-6 or zone 3-4, all three reservoirs contribute, and the procedure discussed in this step applies.

Using the equivalent reservoir concept discussed in Section 10.5 of Volume 7, Table Rock and Bull Shoals Reservoirs are assumed to be an equivalent reservoir which acts in parallel with the Norfork reservoir. The general equations for determining Norfork and Bull Shoals releases is developed from Equation 3-7 and shown below.

$$QO_2 = \frac{TS_3(PS_1+PS_2+QI_1+LF_2-D_2)-(TS_1+TS_2)(PS_3+QI_3-QT-D_3)}{TS_1+TS_2+TS_3} \tag{3-11}$$

$$QO_3 = QT-QO_2 \tag{3-12}$$

As previously mentioned in Section 3.5.1 the local flows into Table Rock and Norfork are assumed to be included in the inflow terms. The subscripts in the equation refer to the reservoir numbers shown in Figure 3-2. In this example QT is the water available above the diversion at Batesville (Col.29) which is found by adding the desired flow at Batesville (Col. 31) and the diversion above Batesville (Col.30) and subtracting the local flow at Batesville (Col. 28). For allocation zone 1-2 required flows at Batesville would be used instead of the desired flow in column 31. The calculated releases QO_2 and QO_3 must be checked to insure that they do not violate the minimum or maximum flow constraints contained in Tables 3.3 and 3.4. If this occurs the flow constraints take priority over the calculated releases.

Table 3.8
Reservoir System Analysis

RESERVOIR 1 TABLE ROCK

NO OUTFLOW CONSTRAINT

MONTH	CONVERSION FACTOR	INFLOW M^3/S	TOTAL RELEASE M^3/S	CHANGE IN STORAGE M^3/S	CHANGE IN STORAGE $M^3 \times 10^6$	STORAGE WITHIN ZONE $M^3 \times 10^6$	STORAGE TOTAL ACCUM. $M^3 \times 10^6$	LEVEL
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
Initial							3070	
3/53	2.6784	268	171	97	260	1350	3330	5.00
4/53	2.5920	230	230	0	0	1350	3330	5.00
5/53	2.6784	211	211	0	0	1350	3330	5.00
6/53	2.5920	20	20	0	0	1350	3330	5.00
7/53	2.6784	20.5	20.5	0	0	1350	3330	5.00
8/53	2.6784	6.5	119	-112.5	-301	1049	3029	3.78
9/53	2.5920	3.5	104.5	-101	-262	787	2767	3.58
10/53	2.6784	4	49	-45	-121	660	2646	3.49
11/53	2.5920	4	35	-31	-80	586	2566	3.43
12/53	2.6784	6	35.5	-29.5	-79	507	2487	3.38
1/54	2.6784	17	9	8	21	528	2508	3.39
2/54	2.4192	15	8	7	17	545	2525	3.40
3/54	2.6784	47	4	43	115	660	2640	3.49
4/54	2.5920	59	5	54	140	800	2780	3.59

Table 3.8
Reservoir System Analysis (continued)

RESERVOIR 2 BULL SHOALS — Q min = 14 M^3/S, Qmax= 380 M^3/S									
LOCAL INFLOW M^3/S	TOTAL RELEASE M^3/S	CHANGE IN STORAGE M^3/S	CHANGE IN STORAGE $M^3 \times 10^6$	STORAGE WITHIN ZONE $M^3 \times 10^6$	STORAGE TOTAL ACCUM. $M^3 \times 10^6$	LEVEL	DIVERSION M^3/S	RIVER RELEASE M^3/S
(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)
					3461			
134	193	112	299	440	3760	5.00	0	193
115	345	0	0	440	3760	5.00	0	345
106	317	0	0	440	3760	5.00	0	317
10	80	-50	-130	310	3630	4.70	8.5	71.5
10	145.5	-115	-308	2	3322	4.01	18	127.5
3	158.5	-37.5	-100	342	3222	3.78	28	130.5
2	139.5	-33	-86	256	3136	3.58	21	118.5
2	65.5	-14.5	-39	217	3097	3.49	7	58.5
2	47.5	-10.5	-27	190	3070	3.43	0	47.5
3	48	-9.5	-25	165	3045	3.38	0	48
8	14	3	8	173	3053	3.39	0	14
7.5	14	1.5	4	177	3057	3.40	0	14
24	14	14	37	214	3094	3.49	0	14
30	17	18	47	261	3141	3.59	0	17

Table 3.8
Reservoir System Analysis (continued)

RESERVOIR 3 NORFORK

$Q_{min} = 11\ M^3/S$ $Q_{max} = 255\ M^3/S$

INFLOW M^3/S	TOTAL RELEASE M^3/S	CHANGE IN STORAGE M^3/S	CHANGE IN STORAGE $M^3 \times 10^6$	STORAGE WITHIN ZONE $M^3 \times 10^6$	STORAGE TOTAL ACCUM. $M^3 \times 10^6$	LEVEL	DIVERSION M^3/S	RIVER RELEASE M^3/S
(19)	(20)	(21)	(22)	(23)	(24)	(25)	(26)	(27)
					1417			
100	54	46	123	215	1540	5.00	0	54
78	78	0	0	215	1540	5.00	0	78
66	66	0	0	215	1540	5.00	0	66
26	26	0	0	215	1540	5.00	6	20
18.5	18.5	0	0	215	1540	5.00	7	11.5
12	30	-18	-48	167	1492	3.78	9	21
11	27	-16	-42	125	1450	3.58	7	20
10	17.5	-7.5	-20	105	1430	3.49	2	15.5
11	15.5	-4.5	-12	93	1418	3.43	0	15.5
11	16	-5	-13	80	1405	3.37	0	16
19.5	12	7.5	20	100	1425	3.47	0	12
24	11	13	31	131	1456	3.61	0	11
34	41	-7	-19	112	1437	3.52	0	41
30.5	25	5.5	14	126	1451	3.59	0	25

Table 3.8
Reservoir System Analysis (continued)

BATESVILLE CONTROL POINT Qmin = 85 M^3/S Qmax = 765 M^3/S				
LOCAL INFLOW M^3/S	WATER AVAILABLE ABOVE DIVERSION M^3/S	DIVERSION M^3/S	MINIMUM DESIRED FLOW M^3/S	ACTUAL FLOW M^3/S
(28)	(29)	(30)	(31)	(32)
349	247	0	113	596
183	423	0	113	606
246	383	0	127	629
44	91.5	8.5	127	127
21	139	18	142	142
18.5	151.5	28	142	142
9.5	138.5	21	127	127
18	74	7	85	85
22	63	0	85	85
21	64	0	85	85
59	26	0	85	85
66	25	0	85	91
58	55	0	113	113
71	42	0	113	113

Table 3.8 Reservoir System Analysis (Cont.)

Columns are identified in numerical order from left to right.

Column(s)	Explanation
1	Date of routing period (month number/year).
2	Factor used to convert M^3/S to cubic metres. (M^3/S times Col. 2 equals $M^3 \times 10^6$).
3,10,19	Average uncontrolled inflow to reservoir in cubic metres per second, which is a hydrologic input.
4,11,20	Total release from the reservoir including any diversions, computed by considering all of the controlling constraints.
5,12,21	Change in storage in cubic metres per second from the previous time period, which is computed by subtracting the total release from the current period inflow.
6,13,22	Change in storage in cubic meters. Columns 5,12,21 are multiplied by the conversion factor in column 2.
7,14,23	End-of-period storage contained within the current allocation zone noted in columns 9,16,25.
8,15,24	Accumulated end-of-period storage computed by adding the change in storage (columns 6,13,22) to the previous period storage.
9,16,25	The relative storage level computed for the reservoir according to Equation 3.1.
17,26	Diversion demand supplied by reservoir. It might be useful to add a column for the diversion demand and a column for the difference between the demand and that supplied (diversion shortage).
18,27	Flow downstream, released from the reservoir which is computed by subtracting the diversion (columns 17,26) from total release (columns 11,20).
28	Uncontrolled inflow below the upstream reservoirs and above the downstream control point, which is a hydrologic input.

Table 3.8 Reservoir System Analysis (Cont.)

Column	Explanation
29	Accumulated upstream river releases (sum of columns 18 and 27).
30	Diversion at Batesville.
31	Desired flow at Batesville which is an input system constraint.
32	Actual flow at Batesville which is dependent upon the system flow constraints and availability of water.

Step 3 (Zone 3-4, 5-6)

Once the release from Bull Shoals is determined, Table Rock and Bull Shoals are assumed to act in tandem. The outflow from Table Rock necessary to bring the tandem reservoirs into balance with Norfork can be calculated as follows:

$$QO_1 = \frac{TS_2(PS_1+QI_1)-TS_1(PS_2+LF_2-D_2-QO_2)}{TS_1+TS_2} \qquad (3\text{-}13)$$

The outflows from Table Rock, Bull Shoals, and Norfork which contribute to the downstream demand at Batesville are shown in columns 4, 18 and 27, respectively.

Table 3-9 summarizes the equations necessary to calculate releases for each storage allocation zone. These equations were developed for reservoir levels that remain within the same storage zone during the computational interval. If it appears that the system will drop from one zone to the next, calculate the volume of storage remaining in the zone at the beginning of the computational interval. This volume ($M^3 x 10^6$) can be divided by the conversion factor in column 2 of Table 3-8 to determine the average monthly yield in M^3/S. The total desired demand (Col. 29) is known in M^3/S. The equations for the upper zone are used to determine releases made to satisfy that part of the demand equal to the remaining storage in the upper zone (M^3/S). The remaining demand is satisfied using the equation for the lower zone. The monthly releases for 8/53 in Table 3.8 are handled in this manner. A similar procedure can be used when reservoirs rise from one zone to another.

Step 2,3 (Zone 1-2)

If the reservoir is in zone 1-2, storage will be used from Bull Shoals and Norfork reservoirs. In order to maintain a level at the top of the inactive pool the releases from Table Rock are equal to the monthly inflows.

Table 3.9. Reservoir Release and Operating Criteria

Zone	Release Equations	General Operating Criteria
5-6	$QO_2 = .1919(PS_1+PS_2+QI_1+LF_2-D_2)$ $-.8081(PS_3+QI_3-QT-D_3)$ $QO_1 = .7678(PS_1+QI_1)-.2322(PS_2+LF_2-D_2-QO_2)$ $QO_3 = QT - QO_2$	Releases are made to meet diversions and desired demand, reduce reservoir storage to the top of conservation pool (level 5), and maintain balance among the reservoirs. QT, the target release values, is the sum of the water flow at Batesville and the diversion above it, less the local flow at Batesville. The flow at Batesville is based on maximum permissible discharge.
4-5	$QO_1 = QI_1$ $QO_3 = QI_3-D_3$ $QO_2 = QT-QO_3$	Releases are made to meet diversions and desired demands. Only storage at Bull Shoals is used. QT, calculated as above, is based on minimum desired flow at Batesville. If excess water is available, water is stored to the top of the conservation zone (5).
3-4	$QO_2 = .1072(PS_1+PS_2+QI_1+LF_2-D_2)$ $-.8928(PS_3+QI_3-QT-D_3)$ $QO_3 = QT-QO_2$ $QO_1 = .2458(PS_1+QI_1)-.7542(PS_2+LF_2-D_2-QO_2)$	Releases are made to meet diversions and desired demands. Storages are used from all three reservoirs and are kept in balance. QT, calculated as above, is based on minimum desired flows at Batesville.

Table 3.9 (Cont.)

Zone	Release Equations	General Operating Criteria
2-3	$QO_1 = QI_1$ $QO_2 = QO_1+LF_2-D_2$ $QO_3 = QT-Q_2$	Releases are made to meet diversions and desired demands. Only storage at Norfork is used. QT, calculated as above, is based on minimum <u>desired</u> flows at Batesville.
1-2	$QO_1 = QI_1$ $QO_2 = .3256(PS_2+QO_1+LF_2-D_2) -.6744(PS_3+QI_3-QT-D_3)$ $QO_3 = QT-QO_2$	Releases are made to meet diversions and required demands. Bull Shoals and Norfork Storages are used and kept in balance. QT, calculated as above, is based on minimum <u>required</u> flows.

Step 2,3 (Zone 2-3)

When the reservoir level is in zone 2-3 only the storage in Norfork reservoir is used to meet the Batesville demand. The outflow from Table Rock is equal to its inflow, and the same for Bull Shoals except for the inflow used to meet its diversion.

Step 2,3 (Zone 4-5)

When the reservoir is in zone 4-5 releases are made only from the Bull Shoals reservoir. Releases from Table Rock are equal to its inflows in order to maintain the top of the conservation pool. Norfork also maintains the level at the top of the conservation pool, using part of its inflows to satisfy its diversion, and releasing the remainder.

Step 4

Once the releases for all the reservoirs are calculated the ending storage volume and reservoir index levels are calculated.

For Norfork, the diversion (Col. 26) and river release (Col. 27) are added to determine the total release (Col. 20). If the inflow (Col. 19) is less than the release (Col. 20) then the difference must be released from storage. By multiplying this difference, which is in units of M^3/S, by the conversion factor in column 2, the reduction in volume ($M^3 x 10^6$) is calculated. This volume is then subtracted from the previous month's storage in column 24 to determine the ending storage.

Column 23 is simply the amount of storage within the allocation zone. The reservoir index level (Col. 25) is found using Equation 3-1. Table 3-10 contains the storage allocation within each zone for all the reservoirs.

Step 5

The ending storage in Table Rock is found by determining the difference between the release (Col. 4) and the inflow (Col. 3). Using the conversion factor discussed in the previous step the change in storage volume is calculated and the ending storage computed (Col. 8).

Table 3.10 Incremental Storages
Used in Calculation of Index Levels

Storage Allocation Within Zone (M^3 X 10^6)

Zone	Table Rock	Bull Shoals	Norfork
5-6	880	2910	900
4-5	0	440	0
3-4	1350	440	215
2-3	0	0	215
1-2	0	290	140
0-1	1980	2590	970

Step 6

The total release from Bull Shoals (Col. 11) is the sum of the diversion (Col. 17) and the release used to meet downstream requirements (Col. 18). The release from Table Rock (Col.4) is added to the local flow (Col. 10) to yield the water available to Bull Shoals. If the amount available is insufficient to meet total releases then water is released and columns 14, 15, 16 are determined as previously described.
and columns 14, 15, 16 are determined as previously described.

Step 7

As a final check, the sum of columns 18 and 27 should be equal to column 29. Column 29 plus column 28 less column 30 should be equal to column 32. Column 32, the water supplied to Batesville, should be within the flow constraints outlined in Tables 3.3 and 3.4. In addition, the reservoir index levels (Col. 9, 16, 25) should be the same as long as all reservoirs are contributing to downstream requirements and no minimum or maximum priority releases were implemented. An example where minimum flow requirements take precedence over balanced index levels occurs in period 1/54 which is discussed in Section 3.5.4.

Section 3.5.3 Computations for Period 6/53

The results of a step-by-step computation are shown in Table 3.8. The first three months (3/53, 4/53 and 5/53) are high flow months and the projects are constrained by flood control operations. This can be noted by looking at the storage level at each of the three reservoirs. In all cases, the reservoirs are at level 5.0 (see columns 9, 16 and 25 of Table 3.8) which, in this case, is the top of the conservation pool (Figure 3.3 or Table 3.7). The fourth month (June 1953) is the first month that the projects operate for conservation purposes. This month will be used to illustrate some of the computations necessary in a reservoir system simulation.

Step 1

The schedule of releases for the 6/53 period are based on supplying the desired demands. All three reservoirs are at the top of the conservation pool, or level 5 for the previous month (5/53). Therefore, it is necessary to determine if normal streamflow will be adequate or if releases from storage will be required to meet the scheduled demands for the period. Table 3.11 gives the demands for this period, and Table 3.12 displays the streamflow.

Table 3.11 Flow and Diversion Demands, June 1953

Demand, Type and Location	Column	Demand (M^3/S)
Diversion from Norfork Res.	26	6
Diversion from Bull Shoals Res.	17	8.5
Diversion above Batesville	30	8.5
Desired flow at Batesville	31	127
	Total	150

Table 3.12 Recorded System Streamflow, June 1953

Streamflow	Column	Flow (M^3/S)
Inflow to Norfork Reservoir	19	26
Inflow to Table Rock Reservoir	3	20
Local flow into Bull Shoals Res.	10	10
Local flow to Batesville	28	44
	Total	100

The flow demands exceed the system streamflow so some water will need to be released.

Step 2,3

Since all three reservoirs were exactly at the top of the conservation pool (5.00) during May, the releases will be made from Zone 4-5. Figure 3.3 and Table 3.10 show that there is no storage allocated between levels 4 and 5 in Table Rock and Norfork Reservoirs. Thus any storage released must come from Bull Shoals Reservoir.

Because releases are to be made from storage allocation zone 4-5, the desired flow at Batesville (Col. 31) of 127 M^3/S will be the target release. QT (Col. 29), the water that must be supplied above the Batesville diversion, is simply the sum of the diversion (Col. 30) and the desired flow (Col. 31) less the local flows at Batesville (Col. 28).

$QT = 127+8.5-44 = 91.5\ M^3/S$

Using the equations in Table 3.9 the outflows from the reservoirs are determined.

$QO_1 = QI_1 = 20\ M^3/S$

$QO_3 = QI_3-D_3 = 26-6 = 20\ M^3/S$

$QO_2 = QT-QO_3 = 91.5-20 = 71.5\ M^3/S$

Neither QO_2, the release from Bull Shoals, nor QO_3, the discharge from Norfork, violate the minimum permissible flows so the computations may proceed to Step 4.

Step 4

Because the total release from Norfork Reservoir (Col. 20), which is the sum of the river release QO_3 and diversion D_3, is equal to the inflow, no change in storage occurs. Thus the accumulated reservoir storage (Col. 24) remains at 1540×10^6 M^3, and the index level (Col. 25) stays at 5.00.

Step 5

The release from Table Rock also equals the inflow so no change in storage occurs. The index level (Col. 9) remains at 5.00 the top of the conservation pool with an accumulated storage of 3330×10^6 M^3 (Col. 8).

Step 6

The outflow from Table Rock of 20 M^3/S and the local flow at Bull Shoals of 10 M^3/S combine to provide 30 M^3/S of water. A river flow of 71.5 M^3/S and diversion of 8.5 M^3/S specify a total required release of 80 M^3/S. Thus the difference between available water and specified demands require that the additional 50 M^3/S be supplied from reservoir storage. 50 M^3/S is equivalent to 130×10^6 M^3 of storage.

$$50 \frac{M^3}{sec} \times \frac{3600 \text{ sec}}{1 \text{ hr}} \times \frac{24 \text{ hr}}{1 \text{ day}} \times \frac{30 \text{ days}}{1 \text{ month (June)}} = 130 \times 10^6 M^3$$

The ending accumulated storage (Col. 15) is found by subtracting the depletion of storage 130×10^6 M^3 from the previous accumulated storage of 3760×10^6 M^3 to yield 3630×10^6 M^3.

As shown in Table 3.10, Bull Shoals has a storage allocation (TS) of 440 $M^3 \times 10^6$. Since 130×10^6 M^3 has been depleted the remaining storage in zone 4-5 (Col. 14) is 310×10^6 M^3. The base index is 4 so the index level (Col. 16) is easily determined.

$$\text{Index Level} = \text{Base Index} + \frac{S}{TS} = 4 + \frac{310}{440} = 4 + 0.70 = 4.70$$

Step 7

To insure no computational errors occured, column 27 (20 M^3/S) plus column 18 (71.5 M^3/S) should equal column 29 (91.5 M^3/S), which it does. The actual flow at Batesville is found from the sum of column 28 (44 M^3/S) plus column 29 (91.5 M^3/S) less column 30 (8.5 M^3/S) yielding 127 M^3/S or the target desired flow. Note that the index levels at the reservoirs are not balanced. This is simply because Norfork and Table Rock reservoirs do not allocate storage to zone 4-5.

Section 3.5.4 Computations for Period 1/54

This period illustrates a case where it is not possible to balance the storage levels at all of the reservoirs to the same level number.

Step 1

Table 3.13 gives the demands and streamflows for this period.

Table 3.13 Monthly Demand and Streamflow, January 1954

Demand, Type and Location	Column	Demand (M^3/S)
Diversion from Norfork Reservoir	26	0
Diversion from Bull Shoals Res.	17	0
Diversion above Batesville Res.	30	0
Desired flow at Batesville	31	85
	Total	85

Streamflow	Column	Flow (M^3/S)
Inflow to Norfork Reservoir	19	19.5
Inflow to Table Rock Res.	3	17
Local flow into Bull Shoals Res.	10	8
Local flow to Batesville	28	59
	Total	103.5

The inflow exceeds the total demand; therefore, the system will accumulate storage.

Step 2

The previous month's index level for Table Rock and Bull Shoals is 3.38, and Norfork is 3.37. With only 18.50 M^3/S (103.5 -85) added to storage, the top of level 4 will not be exceeded, thus the equations in Table 3.9 for zone 3-4 can be used to determine the releases.

The desired flow at Batesville is 85 M^3/S. The desired water above the Batesville diversion (QT) is the sum of columns 30 and 31 less column 28.

$$QT = 85+0-59 = 26\ M^3/S$$

Using the equations in Table 3.9 the release from Bull Shoals and Norfork may be determined.

$$\begin{aligned} QO_2 &= .1072(PS_1+PS_2+QI_1+LF_2-D_2)-.8928(PS_3+QI_3-QT-D_3) \\ &= .1072(507+165+17+8-0)-.8928(80+19.5-26-0) \\ &= .1072(697)-.8928(73.5) = 74.72-65.62 = 9\ M^3/S \end{aligned}$$

$$QO_3 = QT-QO_2 = 26-9 = 17\ M^3/S$$

QO_2 and QO_3 are the discharges necessary to balance Norfork and the equivalent reservoir composed of Table Rock and Bull Shoals. However, the minimum permissible Bull Shoals release is 14 M^3/S, which takes priority over the release calculated to balance the reservoir. If QO_2 is 14 M^3/S, then QO_2 would be 26-14 or 12 M^3/S, which does not violate the minimum permissible flow at Norfork. Therefore:

$$QO_2 = 14\ M^3/S$$

$$QO_3 = 12\ M^3/S$$

Step 3

The release from Table Rock, calculated from the equations in Table 3.9, will balance Table Rock and Bull Shoals.

$$\begin{aligned} QO_1 &= .2458(PS_1+QI_1)-.7542(PS_2+LF_2-D_2-QO_2) \\ &= .2458(507+17)-.7542(165+8-0-14) \\ &= .2458(524)-.7542(159) = 128.80-119.92 = 9\ M^3/S \end{aligned}$$

Step 4

No diversion (Col. 26) occurs in January so the total release from Norfork (Col. 20) is equal to the river flow (Col. 27). The inflow of 19.5 M^3/S (Col. 19) exceeds the release of 12 M^3/S; therefore 7.5 M^3/S of the inflow will be stored. 7.5 M^3/S is equivalent to a volume of $20 \times 10^6 M^3$ for the month of January. The ending accumulated storage (Col. 24) is found by adding the previous storage of $1405 \times 10^6 M^3$ to $20 \times 10^6 M^3$ to yield $1425 \times 10^6 M^3$. The storage within zone 3-4 (Col. 23) is $100 \times 10^6 M^3$. The index level (Col. 25) is the sum of the base level 3 plus the percent zone 3-4 is filled or 100/215, yielding 3.47.

Step 5

The Table Rock inflow (Col. 3) of 17 M^3/S exceeds the release (Col. 4) of 9 M^3/S by 8 M^3/S. The surplus inflow equivalent to $21 \times 10^6 M^3$ increases the accumulated storage (Col. 8) from $2487 \times 10^6 M^3$ to $2508 \times 10^6 M^3$. Storage allocation zone 3-4 (Col. 7) contains $528 \times 10^6 M^3$ of water. The index level is 3+528/1350 or 3.39.

Step 6

The total release from Bull Shoals (Col. 11) of 14 M^3/S is equal to the river flow (Col. 18) because there is no diversion in January. The inflow from Table Rock (Col. 4) of 9 M^3/S plus the local flow (Col. 10) of 8 M^3/S provide a surplus of 3 M^3/S (17-14), which will increase the storage by $8 \times 10^6 M^3$. Adding this increase to the previous month's storage of $3045 \times 10^6 M^3$ yields $3053 \times 10^6 M^3$. Storage allocation zone 3-4 (Col. 14) contains $173 \times 10^6 M^3$ of water. The index level is 3+173/440 or 3.39.

Step 7

The river flow from Norfork (Col. 27) of 12 M^3/S and Bull Shoals (Col. 18) of 14 M^3/S make 26 M^3/S (Col. 29) of water available above Batesville. Adding the local inflow (Col. 28) of 59 M^3/S and noting that there is no diversion in January provides Batesville with a total flow of 85 M^3/S (Col. 32), which is precisely equal to its desired flow. The index level at Norfork (Col. 25) is different than the indices at Table Rock (Col. 9) and Bull Shoals (Col. 16) because the minimum permissible outflow at Bull Shoals had priority over the balancing discharge.

Chapter 4

Summary

CHAPTER 4. SUMMARY

It is rare that a complete water resource system is designed at the same time from the start to finish. The problem generally is to develop the best means of integrating one or more new units or new services into an existing system, subject to physical, legal and political constraints.

Because of the extreme complexity of the interactions among the various sources of water, unit operations, and demands for water; the modeling of a water resource system is usually extremely difficult, and the optimization or development of the best plan and best operation is substantially more difficult. For this reason, it is best to make a first approximation of a system and operation rules using the best judgment of experienced water resources planners and managers. Then the system operation can be simulated in the degree of detail desired, and evaluations of outputs can be made.

The simulation and evaluation processes discussed in Chapter 2 may be developed for general system descriptions and demand schedules or highly detailed ones. The general process starts first with the identification of the system as outlined in Section 2.2. Then the objectives are defined and criteria selected, according to the procedures discussed in Section 2.3, so that the scope of the simulation study may be determined and resultant alternative design configurations may be evaluated. Sections 2.4 through 2.10 outline the steps necessary to formulate a simulation model, and select initial design configurations and regulation schemes. Finally Section 2.11 discusses the analysis and evaluation of alternative design systems in search of the optimal one. In this case, the system design should be considered to include development of operation rules, and consequently requires a two level optimization process. For each plan considered, operation rules must be developed that are likely to be different from the rules for any other system configuration considered.

Chapter 3 illustrates the mechanics of developing a simulation model for a simple three reservoir system. Because only one design is considered and the operating policy is assumed, no optimization or alternative system evaluation is considered. However, the example does provide a fundamental background in how to assemble data, and to construct and operate a simulation model.

Although the simplified example in Chapter 3 was performed by hand computations, the complexity normally encountered in reservoir system analysis necessitates use of the computer. Two computer programs available are HEC 5C "Simulation of Flood Control and Conservation Systems" and HEC 3 "Reservoir System Analysis for Conservation." The description of the capabilities of HEC 3 is contained in Appendix 3 of IHD Volume 1, and the description of HEC 5C is in Appendix 1 of IHD Volume 7.

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Appendix 1

Conversion Constants

APPENDIX I

CONVERSION CONSTANTS

1 cubic meter	=	35.315 cubic feet
1 acre-foot	=	1233.482 cubic meters
1 gallon	=	3.785 liters
1 cubic foot	=	28.317 liters
1 Hectare	=	2.471 acres
1 Hectare	=	10,000 square meters
1 square meter	=	10.76391 square feet
1 acre	=	4046.86 square meters
1 square mile	=	2.59 square kilometers
1 Inch	=	2.54 centimeters
1 Meter	=	3.28084 feet
1 Kilometer	=	3280.84 feet
1 Mile	=	1.609 kilometers
24 hour-cfs	=	1.9835 acre-feet
28 day-cfs	=	55.538 acre-feet
30 day-cfs	=	59.504 acre-feet
31 day-cfs	=	61.488 acre-feet
30.475 day-cfs	=	60.446 acre-feet
1 week-cfs	=	13.8843 acre-feet
1 inch square mile	=	53.3333 acre-feet
1 cfs	=	724 acre-feet/year
1.55 cfs	=	1 mgd
1 cfs	=	448.83 US gallons per minute
.167 inches/week	=	cfs/1000 acres
.666 inches/28 day	=	cfs/1000 acres
.714 inches/30 day	=	cfs/1000 acres
.725 inches/30.47 days	=	cfs/1000 acres
.738 inches/31 days	=	cfs/1000 acres
8.688 inches/year	=	cfs/1000 acres

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