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# Volume 8
# Reservoir Yield

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Augustine J. Fredrich

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US Army Corps of Engineers
Institute for Water Resources
Hydrologic Engineering Center
609 Second Street
Davis, CA 95616-4687

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This is Volume 8 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 are described which can be used to determine the relationship between reservoir storage capacity and reservoir yield for a single reservoir. Non-sequential and sequential methods for reservoir yield analysis are described in detail. Other topics include: guidelines for selection of technical procedure; data requirements; use of generalized and simulated data; establishment of study criteria; and, development and use of rule curves. Advantages and limitations of the various methods are discussed. A description of the generalized computer program entitled "Partial Duration - Independent Low Flow Events" is included as an appendix.

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Hydrologic Engineering Methods for Water Resources Development

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US Army Corps of Engineers
Institute for Water Resources
Hydrologic Engineering Center
609 Second Street
Davis, CA 95616

(530) 756-1104
(530) 756-8250 FAX
www.hec.usace.army.mil

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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 Hydrological Decade.

Volume 8 represents various methods and techniques which can be used to analyze the relationship between reservoir storage and yield. Although some information is presented on simplified techniques, most of the manual deals with the use of the sequential routing study, not only because it is the most widely known technique, but also because it produces the best results and is ideally suited for application with electronic computers. Throughout the volume, it has been assumed that the reader has little or no experience in the use of the techniques. Consequently, a great deal of attention is given to guidance in selection of data, development of criteria and assumptions and other elementary topics. While this may make përusal of the volume laborious for the more experienced reader, it is believed that such information is essential for intelligent use of the techniques described and is generally unavailable elsewhere.

The volume was written by Augustine J. Fredrich, with the exception of Chapter 7 which was based largely upon information furnished by Gerald E. Thomas and Jim Dalton when they were employed by the Little Rock District of the Corps of Engineers. Much of the material in Chapter 6 was based on studies by Bill S. Eichert and A. J. Fredrich previously reported in the Hydrologic Engineering Center's "Reservoir Storage-Yield Procedures" dated May 1967. Helpful suggestions were obtained from Leo R. Beard. Final editing and review was performed by Kenneth N. Brooks.

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Chapter 1

# Introduction

# INTRODUCTION

## Section 1.01 - Purpose

The purposes of this volume are: (1) to provide a descriptive summary of the technical procedures used in the hydrologic studies associated with the design of reservoirs for conservation purposes; (2) to furnish background information concerning the data requirements, advantages and limitations of the various procedures; (3) to describe in detail several of the important procedures that are most useful in areas of sparse data; and (4) to establish guidelines which will be helpful in selecting a procedure, conducting the studies, and evaluating results.

## Section 1.02 - Scope

The techniques described herein are, in general, limited to the technical procedures which are used to determine the relationship between reservoir storage capacity and reservoir yield (supply) for a single reservoir. Although the discussions and examples are limited to single reservoir analysis, many of the principles are generally applicable to multi-reservoir systems. Volume 9, entitled "Water Resource System Analysis," contains information regarding the application of the principles described herein to the analysis of a multi-reservoir system.

The procedures described herein may be used to determine storage requirements for water supply, water quality control, hydroelectric power, irrigation and other conservation purposes. This volume does not, however, discuss the determination of demands (water requirements) for the various purposes or the establishment of demand priorities. These aspects of water resources development are excluded because they are usually dependent upon political and economic objectives and physical conditions which vary considerably from country to country and even among regions within a country.

## Section 1.03 - Description of the Problem

The determination of storage-yield relationships for a reservoir project is one of the basic hydrologic analyses associated with the design of reservoirs. The problem of determining storage-yield relationships might be briefly described as the application of various theoretical and empirical methods to hydrologic data in order to determine the regulating effects of a reservoir project. The step-by-step procedures involved in storage-yield studies are illustrated in fig. 1.01.

Many of the methods described in other volumes of this report are necessary to provide data to evaluate the hydrologic aspects of reservoir planning, design, and operation. In many cases, the methods required to provide data for a reservoir analysis are more complex than the method for the reservoir study itself. However, since the usefulness and validity of the reservoir analysis are directly dependent upon the accuracy and soundness of basic data, complex methods can often be

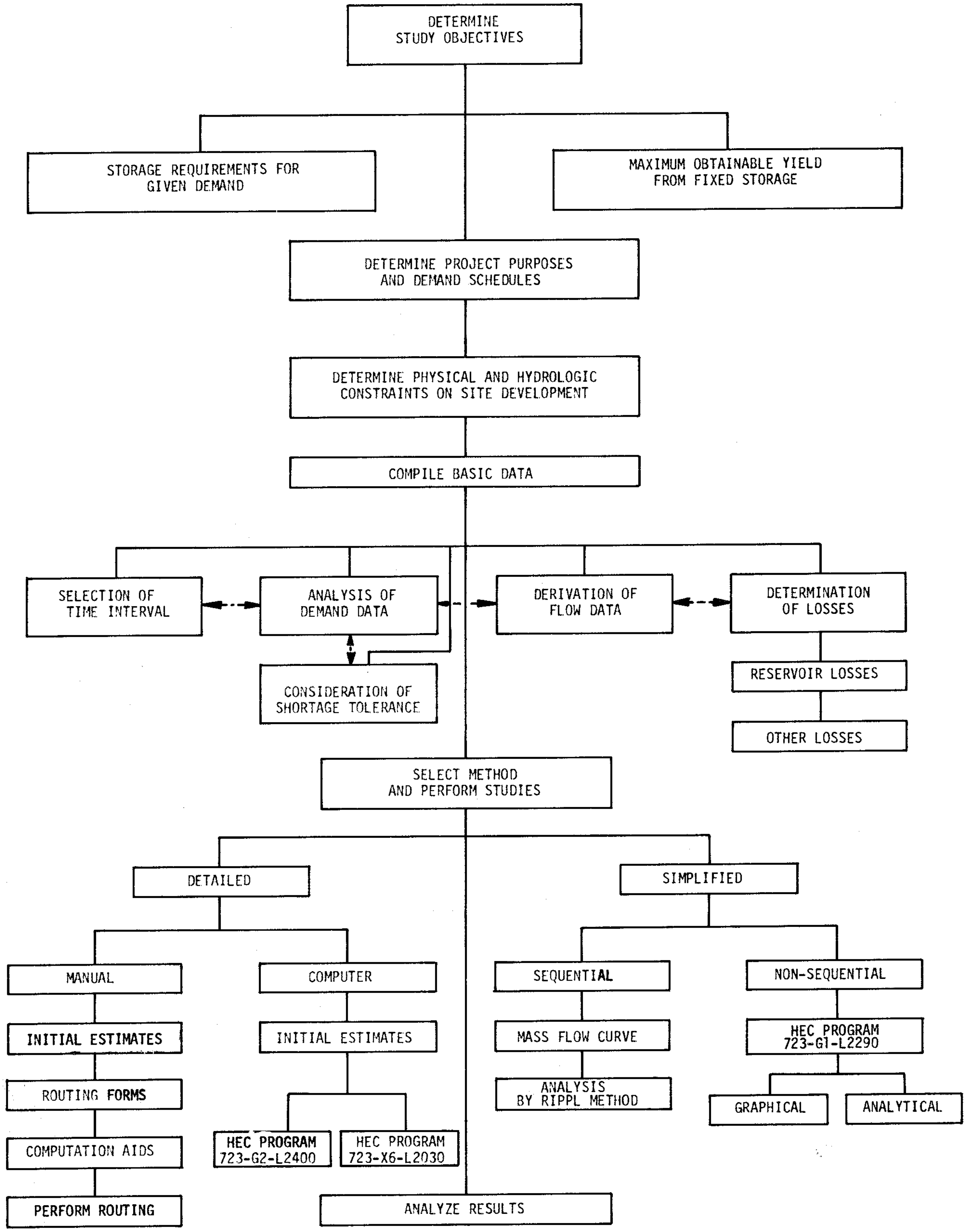



Fig. 1.01. Determination of reservoir storage requirements and firm yield

justified to develop the data.

This volume contains information on techniques to estimate and adjust basic data, a step-by-step description of sequential routing analysis for several types of studies, and a summary of methods to analyze the results of reservoir studies. Emphasis has been given to the sequential routing type of analysis because (1) it is adaptable to any study of a single reservoir or of multiple reservoir systems; (2) it gives results that are easily understood and explained by engineers familiar with basic hydrologic principles; (3) the accuracy of the study and the results can be closely controlled by the engineers performing and supervising the studies; and (4) it can be used with sparse basic data.

## Section 1.04 - Study Objectives

Before any meaningful storage-yield analysis can be made, it is necessary to establish and consider the objectives of the hydrologic study. The objectives of the study could range from a preliminary study to a detailed analysis for coordinating reservoir operation for several purposes. The objectives, together with the available data, will control the degree of accuracy of the study.

Basically, there are two ways of viewing the storage-yield relationship. The most common viewpoint requires the determination of the storage required at a given site to supply a given yield. This type of problem is usually encountered in the planning and early design phases of a water resources development study. Either simplified or detailed techniques may be used, however, due to the lack of precision generally associated with preliminary studies, the simplified techniques are most commonly used.

The second viewpoint requires the determination of yield from a given amount of storage. This often occurs in the final design phases or in re-evaluation of an existing project for a more comprehensive analysis. Since a higher degree of accuracy is desirable in such studies, detailed sequential routings are usually used.

Other objectives of a storage-yield analysis include the following: determination of complementary or competitive aspects of multiple project development; determination of complementary or competitve aspects of multiple purpose development in a single project; and analysis of alternative operation rules for a project or group of projects. Each objective dictates implicitly the method which should be used in the analysis.

Chapter 2

# Selection of Technical Procedure

# SELECTION OF TECHNICAL PROCEDURE

## Section 2.01 - Types of Procedures

The procedures used to determine the storage-yield relationship for a potential dam site may be divided into two general categories: (1) simplified analysis, and (2) detailed sequential analysis. The selection of the technical procedure may be governed by availability of data, study objectives, or budgetary considerations. In general, the simplified techniques are only satisfactory when the study objectives are limited to preliminary or feasibility studies. Detailed methods are usually required when the study objectives advance to the design phase.

The detailed sequential methods may be subdivided into simulation analyses and mathematical programming analyses. In simulation analysis, the most common detailed method, the physical system is simulated by performing a sequential reservoir routing (operation) study. In this type of study, one usually attempts to reproduce, with a high degree of fidelity, the temporal and spatial variation in streamflow and reservoir storage in a reservoir-river system. This is accomplished by a relatively complex type of bookkeeping which accounts for as many significant accretions and depletions as possible. In mathematical programming analysis, which has received increased emphasis in the past few years, the objective is to develop a mathematical model which can be used to analyze the physical system without necessarily reproducing exact sequential occurrences. The major limitation of this type of analysis is that the model is usually calibrated by use of historical data and, unless care is exercised by the user, past events may unduly bias the outcome.

A simplified method is often used to save time and money if demands for water are relatively simple or if approximate results are sufficient, as in the case of many preliminary studies. However, it should be emphasized that the objective of the simplified methods is to obtain a good estimate of the results which could be achieved by detailed sequential analysis. Simplified methods consist generally of mass curve and depth-duration analyses. Further discussion of these methods is contained in Chapter 6.

The electronic computer has changed the role of the simplified methods because of the relatively low cost of a detailed sequential routing utilizing computer programs such as The Hydrologic Engineering Center's programs "Reservoir Yield" (723-G2-L2400) and "HEC-3 Reservoir System Analysis" (723-X6-L2030). The increased feasibility of a detailed sequential routing study has not eliminated the need for simplified methods entirely, because these methods now become valuable tools to obtain good estimates of input data for the sequential routings by electronic computer.

In the past, detailed sequential routings have been used almost exclusively for development of operating plans for existing reservoirs and reservoir systems. Planning studies, on the other hand, have been based almost entirely on simplified methods. However, the advent of the comprehensive basin planning concept, the growing demand for more efficient utilization of water resources, and the increasing competition for water among various project purposes indicate a need for detailed sequential routings in planning studies. In a reservoir system, simplified methods will usually be inadequate for even preliminary analyses of individual projects because of the inability of the best simplified methods to account for a hydrologically integrated system operation. In order to provide the optimum benefit from water resources development, the planner and hydrologic engineer must work together in the planning stage to produce a plan of operation which will provide for intelligent management of a reservoir and efficient, economical utilization of all available water. The use of detailed sequential routings by electronic computer in hydrologic analyses for planning will be of major significance in achieving these objectives.

## Section 2.02 - Factors Affecting Selection of Method

Before initiating a storage-yield study, the study objectives and data availability should be examined in order to ascertain: (1) the method best suited for the study requirements; (2) the degree of accuracy required to produce results consistent with the study objectives; and (3) the basic data required to obtain the desired accuracy using the selected method. In preliminary studies, limitations in time and funds might dictate the data and method to be used and the accuracy obtainable. A technical study work plan is very useful in organization of study objectives, inventory of available data, and the selection of general procedures.

The availability of basic physical and hydrologic data will quite frequently be a controlling factor in determining which of the several technical methods should be used. Obviously, the detailed methods require more data which are often not available. Recent advances in the simulation and generalization of hydrologic and physical data, however, have fostered hopes that adequate data can be developed from a knowledge of the physical and climatological characteristics of a region. The simplified methods require less data, but the reliability of the results decreases rapidly as the length of hydrologic record decreases. Therefore, it is often desirable to simulate additional hydrologic data for use with simplified methods.

The relationship between study objectives and the type of technical procedure to be used is straight-forward; the more detailed analyses are needed when a high degree of accuracy is desired. Sometimes there is a need to determine differences between alternatives in, say, a screening study where a high degree of accuracy is not needed, but a detailed method must be used. For example, if simplified methods do not utilize data which could be expected to have major influences upon differences between alternatives, it would be necessary to utilize a more detailed method which accounts for variations in these data.

Chapter 3

# Analysis and Use of Available Data

## ANALYSIS AND USE OF AVAILABLE DATA

### Section 3.01 - Streamflow Data

Since the availability of physical and hydrologic data is a significant factor in the selection of an appropriate technical method for reservoir-yield studies, it is important to be cognizant of the nature, source, reliability, and adequacy of available data. If estimates are needed, the assumptions used should be documented, and the effect of errors in the estimates on the technical procedure should be considered.

Streamflow data are often not satisfactory for direct use in project studies. Usually streamflow information is required at locations other than gaging stations and for conditions of upstream development other than those under which flows occurred historically. At such locations where streamflow data are needed, estimates are usually made by correlation with stations having records for the desired time or by adjustment of such records on the basis of tributary area and other hydrologic factors. In deriving and applying the required relationships, it is desirable to use unregulated, "natural" flows in order that correlation procedures will apply. Where synthetic inflows are to be obtained, as with a stochastic generation process, natural flows should be used since general frequency functions characteristic of natural flows are employed in this process.

It is not always feasible to convert flows to natural conditions. Required data might not be available or the flow modification might be so complex as to require an unreasonable amount of computation. When feasible, conversion is made by adding historical storage changes (plus net evaporation) and upstream diversions (less return flows) to historical flows at the gaging stations for each time interval in turn. Under some conditions, it is necessary to account for differences in channel and overbank infiltration losses, distributary flow diversions, travel times and other factors. When it is not feasible to convert flows completely to natural conditions, they should be adjusted, if only approximately, to uniform conditions, such as present conditions. In such cases, every reasonable effort should be made to remove special influences, such as one major reservoir, that would cause unnatural variations of flow.

After recorded flows are converted to uniform conditions, flows for missing periods of record at each pertinent location should be estimated by correlation with recorded flows at other locations in the region. Usually only one other location is used and linear correlation of flow logarithms is used. It is more satisfactory, however, to use all other locations in the region that can contribute independent information on the missing data. Although this would require a large amount of computation, the computer program HEC-4 accomplishes this for monthly streamflow and is available in Volume 2 of this report. Flow estimates for locations where no record exists can be estimated satisfactorily on a cubic meter per second (cumecs) per square kilometer basis in some cases, particularly where a gage exists on the same stream. In most cases, however, it is necessary to correlate mean flow logarithms (and sometimes standard deviation of flow logarithms) with logarithm of drainage area size, logarithm of normal seasonal precipitation, and other basin characteristics. Correlation procedures and suggested basin characteristics are described in Volume 2.

After project flows for a specified condition of upstream development are obtained for all pertinent locations and periods, they must be converted to pre-project (non-project) conditions. Non-project conditions are those that would prevail during the lifetime of the proposed project if the project was not constructed. This conversion is made by subtracting projected upstream diversions and storage changes and by accounting for evaporation, return flows, differences in channel infiltration and timing where appreciable. Where non-project conditions will vary during the project lifetime, it is ordinarily necessary to convert to two or more sets of conditions, such as those at the start and end of the proposed project life. Separate operation studies would then be made for each condition. This conversion to future conditions can be made simultaneously with project operation studies, but a separate evaluation of non-project flows is usually required for economic evaluation of the project.

## Section 3.02 - Reservoir Losses

The non-project inflow to a proposed reservoir is determined by making necessary adjustments to recorded flow data as previously discussed.

This inflow represents the flow at the project site without the reservoir and includes runoff from the entire effective drainage area above the dam, including the reservoir area. Under non-project conditions, runoff from the area to be inundated by the reservoir is ordinarily only a fraction of the total precipitation which falls on that area. However, under project conditions infiltration losses over the reservoir area are usually minimal during a rainfall event; thus practically all of the precipitation falling on the reservoir area will appear as runoff. Therefore, the inflow will be greater under project conditions than under non-project conditions, if inflow is defined as total contribution to the reservoir before evaporation losses are considered. In order to determine the amount of water available for use at the reservoir, evaporation must be subtracted from project inflow. In operation studies, non-project inflow is ordinarily converted to available water in one operation, without computing project inflow as defined above. This is done in one of two ways: (1) by means of a constant annual loss each year with seasonal variation, or (2) with a different loss each period, expressed as a function of observed precipitation and evaporation. These two methods are described in the following paragraphs.

The constant annual loss procedure consists of estimating the evapotranspiration and infiltration losses over the reservoir area for conditions without the project, and the evaporation and infiltration losses over the reservoir area with the project. Non-project losses are usually estimated as the difference between average annual precipitation and average annual runoff at the location, distributed seasonally in accordance with precipitation and temperature variations. These are expressed in millimeters of depth. Under project conditions, infiltration losses are usually ignored, and losses are considered to consist of only direct evaporation from the lake area, expressed in millimeters for each period. The difference between these losses is the net loss due to the project. Fig. 3.01 illustrates the differences between non-project and project losses.

The second method used to account for the change in losses due to a reservoir project is based on historical records of long-term average monthly precipitation and evaporation data. This is accomplished by estimating the average runoff coefficient (that is, the ratio of runoff to rainfall) for the reservoir area under pre-project conditions and subtracting this from the runoff coefficient for the reservoir area under project conditions. The runoff coefficient for project conditions is usually 1.0, but a lower coefficient may be used if substantial infiltration or leakage from the reservoir is anticipated. The difference between pre- and post-project runoff coefficients is the net gain expressed as a ratio of precipitation falling on the reservoir (often estimated to be .7 for semi-arid regions). This increase in runoff is subtracted from gross reservoir evaporation (often estimated as .7 of pan evaporation) to obtain a net loss. Table 3.01 illustrates the use of this method.

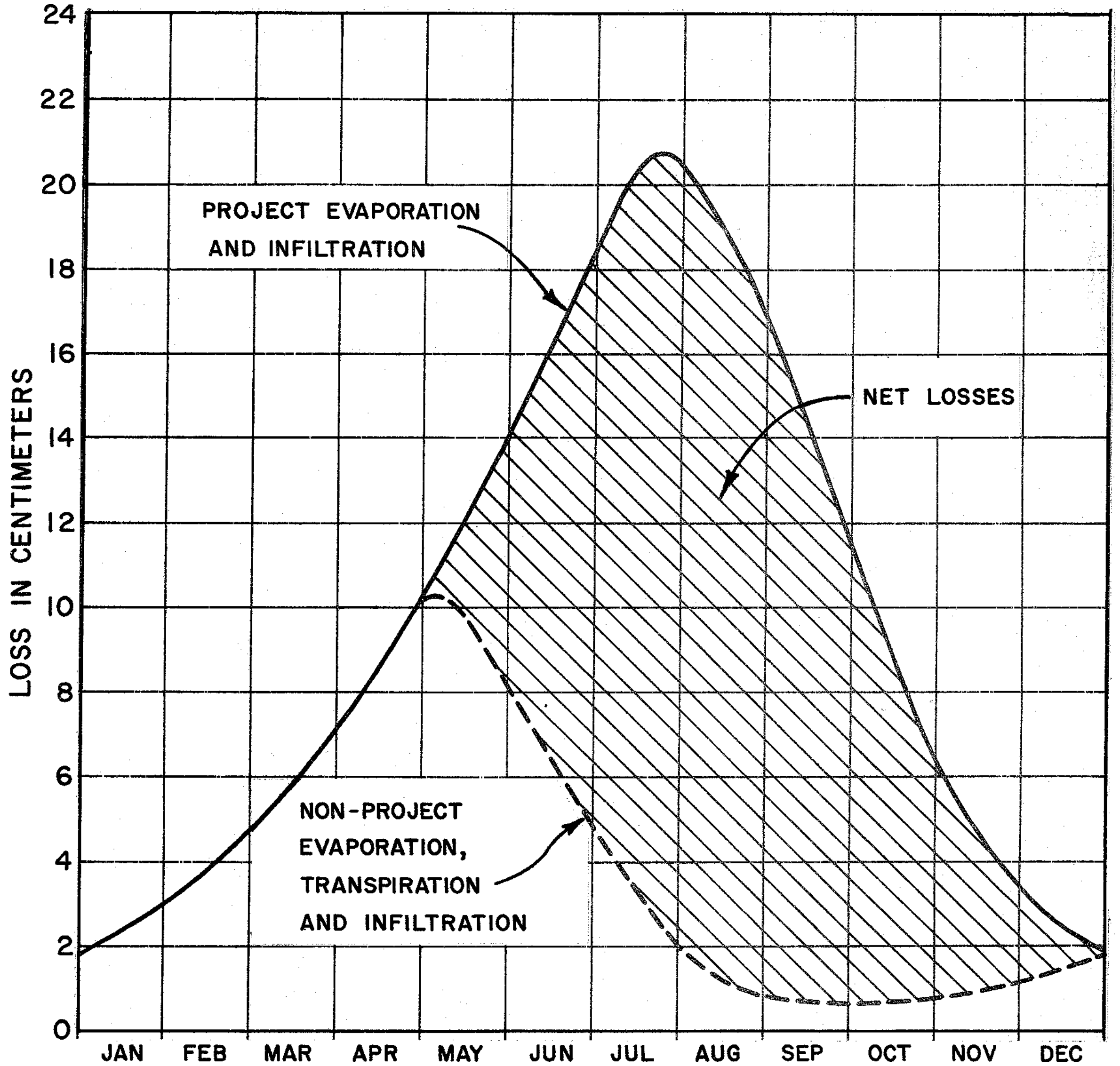



Fig. 3.01 Project and non-project reservoir losses

Table 3.01 Sample computation of average net reservoir loss

| | AVERAGE (1) PAN EVAPORATION (millimeters) | AVERAGE (2) GROSS RESERVOIR EVAPORATION (millimeters) | AVERAGE (1) PRECIPITATION (millimeters) | AVERAGE (3) POST-PROJECT INCREASE IN RUNOFF (millimeters) | AVERAGE (4) NET RESERVOIR LOSS (millimeters) |
|---|---|---|---|---|---|
| JANUARY | 43 | 30 | 53 | 32 | -2 |
| FEBRUARY | 63 | 44 | 61 | 37 | +7 |
| MARCH | 83 | 58 | 50 | 30 | +28 |
| APRIL | 150 | 105 | 83 | 50 | +55 |
| MAY | 136 | 95 | 122 | 73 | +22 |
| JUNE | 186 | 130 | 40 | 24 | +106 |
| JULY | 233 | 163 | 33 | 20 | +143 |
| AUGUST | 224 | 157 | 35 | 21 | +136 |
| SEPTEMBER | 173 | 121 | 29 | 17 | +104 |
| OCTOBER | 86 | 60 | 29 | 17 | +43 |
| NOVEMBER | 45 | 31 | 14 | 8 | +23 |
| DECEMBER | 33 | 23 | 46 | 28 | -5 |
| TOTAL | 1455 | 1017 | 595 | 357 | +660 |

NOTES:

(1) Long term monthly averages from climatological records.

(2) In this example, it has been assumed that the reservoir evaporation is 70% of the pan evaporation in all months. However, the percentage may vary from month to month.

(3) In this example, it has been assumed that under pre-project conditions 40% of the precipitation appeared as runoff in each month and therefore the post-project increase in runoff is 60% of the precipitation. In some cases, it might be necessary to have the percentage of increase in runoff vary from month to month in order to accurately represent the historical conditions.

(4) Average net reservoir loss is the average gross reservoir evaporation minus the average post-project increase in runoff. Positive values of average net reservoir loss indicate that post-project losses exceed gains and negative values indicate that post-project gains exceed losses.

## Section 3.03 - Other Losses

In final project studies it is often necessary to consider other types of project losses which may be of minor importance in preliminary studies. Often, these losses cannot be estimated until a project design has been adopted. The importance of these losses is dependent upon their relative magnitude. That is, losses of 5 $m^3$/sec (cumecs) might be considered unimportant on a stream which has a minimum average annual flow of 1,500 cumecs; but such losses would be significant on a stream with a minimum average annual flow of 25 cumecs. Various types of losses are discussed in the following paragraphs.

The term "losses" may not actually denote a physical loss of water from the system as a whole. Usually water which is unavailable for a specific project purpose is called a "loss" for that purpose although it may be used at some other point or for some other purpose. For example, water which leaves the reservoir through a pipeline for purposes of municipal water supply or fish hatchery requirements might be called a loss to power. Likewise, leakages through turbines, dam, conduits, and spillway gates are considered losses to power but they are ordinarily not losses to flow requirements at a downstream station. Furthermore, such losses that become available for use below the dam should be added to inflow at points downstream from the project.

Leakage at a dam or in a reservoir area is considered as losses to purposes which are dependent upon availability of water at the dam or in the reservoir itself; i.e., power generation, pipelines from the reservoir, and any purpose which utilizes pump intakes which are located at or above the dam. As a rule, leakage through, around or under a dam is relatively small and is difficult to quantify before a project is actually constructed. In some cases, the measured leakage at a similar type of dam or in a similar geologic area may be used as a basis for estimating losses at a proposed project. The amount of leakage is a function of the type and size of dam, the geologic conditions, and the pressure caused by water in the reservoir.

Leakage from conduits and spillway gates is a function of gate perimeter, type of seal, and head on the gate. The amount of leakage may again be measured at existing projects with various types of seals, and a leakage rate computed per foot of perimeter for a given head. This rate may then be used to compute estimated leakage for a proposed project by using the proposed size and number of gates and the proposed head on the gates.

If a proposed project is to include power, and if the area demand is such that the turbines will sometimes be idle, it is advisable to estimate leakage through the turbines when closed. This leakage is a function of the type of penstock gate, type of turbine wicket gate, number of turbines, and head on the turbine. The actual leakage through a turbine may be measured at the time of acceptance and during annual maintenance inspections, or the measurements of similar existing projects may be used to estimate leakage for a proposed project.

An estimate of the percent of time that a unit will be closed may be obtained from actual operation records for existing units in the same demand area. The measured or estimated leakage rate is then reduced by multiplying by the proportion of time the unit will be closed. For example, suppose that leakage through a turbine has been measured at 0.1 cumecs, and the operation records indicate that the unit is closed 60% of the time. The average leakage rate would be estimated at 0.1 x 0.6 or 0.06 cumecs.

The inclusion of a navigation lock at a dam requires that locking operations and leakages through the lock be considered. The leakage is dependent upon the lift or head, the type and size of lock, and the type of gates and seals. Again, estimates can be made from observed leakage at similar structures. Water required for locking operations should also be deducted from water available at the dam site. These demands can be computed by multiplying the volume of water required for a single locking operation times the number of operations anticipated in a given time period and converting the product to a flow rate over the given period.

The use of water for purposes related to operation of a project is often treated as a loss. Station use for sanitary and drinking purposes, cooling water for generators, and water for condensing operations have been estimated to be about 0.06 cumecs per turbine at some stations in the southwestern United States. Examination of operation records for comparable projects in a given study area may also be useful in estimating these losses. If house units are included in a project (to supply the project's power needs), data should be obtained from the designer in order to estimate water used by the house unit or units.

The competitive use of water should also be considered when evaluating reservoir losses. In initially estimating yield rates for various project purposes at a multiple-purpose project, competitive uses of water are often treated as losses. For example, consider a proposed project on a stream with an average minimum usable flow of 16 cumecs. The reservoir of this project is to supply 1.5 cumecs by pipeline for downstream water supply and 2.0 cumecs for a fish hatchery in addition to providing for hydroelectric power production. The minimum average flow available for power generation is thus, 16 - (1.5 + 2.0) = 12.5 cumecs. Care should be exercised in accounting for all such competitive uses when making preliminary yield estimates.

## Section 3.04 - Flow Demands

Seasonal variations in demands for water can substantially affect the storage-yield relationship. If it is known or anticipated that the demands for water from a reservoir will have significant seasonal variations, every effort should be made to obtain an estimate of the expected variation. Usually, these variations can best be estimated by consultation with the consumer or by examination of the records of similar projects in comparable regions. Further discussions of the variations inherent in demands for several purposes are contained in Chapter 5.

## Section 3.05 - Climatology

Climatological data are useful in estimating reservoir losses due to evaporation and in estimating runoff from snowmelt. Furthermore, in the absence of streamflow records, the importance of climatological data is greatly increased because of its usefulness in simulating and generalizing other hydrologic data. References 6 and 13 describe typical examples of the simulation of hydrologic data from climatological data. Additionally, climatological data may be used as a guide to estimate missing streamflow data.

In recent years, several computer models have been developed which attempt to simulate or synthesize streamflow data from climatological information and physical watershed characteristics. Among the most prominent of these are the Stanford Watershed Model and several subsequent modified versions of that model (ref. 16), and the SSARR Model used by the Corps of Engineers in the Pacific Northwest of the United States (ref. 21). In the hands of a knowledgable user, such models can be very helpful in developing streamflow information where only limited streamflow records exist, but where extended climatological data are available. The use of these models, however, requires a substantial amount of hydrologic engineering experience, a thorough understanding of the model itself, and access to a relatively large computer.

Chapter 4

# Use of Generalized and Simulated Data

USE OF GENERALIZED AND SIMULATED DATA

## Section 4.01 - Need for Generalized and Simulated Data

The use of generalized and simulated data has become more prevalent in the past few years because of the increased use of detailed methods which require large amounts of data. It is doubtful, however that even a simplified analysis could be made in many regions without the use of generalized data. In areas where very little data are available, the need for generalized and simulated data is usually greatest.

For purposes of discussion in the subsequent paragraphs, generalized data are defined as the physical or hydrologic data pertinent to a specific locale which have been developed by analysis of the data recorded at other locations in the same region. Generalization may be accomplished by simple inspection and judgment or by techniques such as simple or multiple linear regression. The latter techniques are described in Volume 2 entitled "Hydrologic Data Management". Usually, generalization is most useful with data which are influenced by factors characteristic of relatively large regions and which are known or can be estimated with a relatively high degree of accuracy. A major problem associated with generalization of data is that the use of an improper factor can cause large errors in the generalized data. For this reason, care should be exercised in selecting factors for a regional analysis to insure that they are, in fact, related to the data being generalized.

Simulated data are defined as the physical or hydrologic data pertinent to a specific locale which have been developed by use of the statistical characteristics of recorded or generalized data and an appropriate random component. Volume 2 also contains detailed discussions of most aspects of simulating hydrologic data. Such data are useful in studying long-term effects of reservoir projects and in examining critical flood and drought sequences. The value of simulated data, however, is highly dependent upon the recorded data and also whether or not such recorded data are representative with respect to long-term averages and variations.

## Section 4.02 - Types of Generalized and Simulated Data

In general, hydrologic data can be either generalized or simulated, while physical data can only be generalized. Combinations of generalized and simulated data are used in storage-yield studies, and recent studies describe techniques for simulating hydrologic data from statistical measures that are based upon regional analyses (references 6 and 13).

Hydrologic data are subject to random or quasi-random variations and are often simulated when historical records are short or non-existent. Evaporation, streamflow, precipitation, and water quality parameters such as temperature and total dissolved solids are examples of data which might be simulated. Generalized hydrologic data are usually related to either physical or climatological factors representative of a region. For example, evaporation for a particular reservoir site where no evaporation data exist might be generalized from analysis of evaporation data from other stations in the region.

Physical data are more frequently generalized than simulated. Seasonal fluctuations in demands, channel capacity, seepage rates, and area-capacity relationships are examples of physical data which might be generalized. Reference 11 describes a study in which generalized data were used for a screening study in order to minimize data preparation.

## Section 4.03 - Advantages and Limitations

The use of simulated and generalized data is essential in conducting analysis of the storage-yield relationship in areas of sparse data. Although there are dangers inherent in the use of such data, they can be minimized by exercising reason and good judgment concerning the nature of the hydrologic regime and the objectives of the study. The major advantage of using simulated data is that the methods of analysis can be improved. Other advantages include: (1) a better definition of differences between alternatives; (2) the capability of comparing various schemes of development or plans of operation with respect to sensitivity to variations in hydrologic data; (3) the capability to observe and evaluate hydrologic extremes which may otherwise be unanticipated; and (4) the opportunity to plan in advance, measures to minimize the effects of extremes which might occur in the future. However, the effect of using simulated or generalized data on study objectives must be evaluated with respect to the benefits obtained from this usage as well as the errors which might be induced.

The lack of observed, basic data is the most significant limitation in the application of simulated and generalized data to a reservoir design problem. However, there is promise that the importance of this limitation may be reduced by current investigations which could lead to a reduction in data requirements without an attendant increase in errors in the simulated or generalized data. A second limitation is the sensitivity of the techniques to errors in judgment or non-representative basic data. This may be overcome by limiting the use of such techniques to experienced and knowledgable personnel. Finally, there is a tendency to accept the simulated and generalized data without reservation and to consider such data as being equivalent to historical data. The acceptance and use of these data should be qualified by knowledge and understanding of the observed basic data.

Chapter 5

# Establishment of Study Criteria

## ESTABLISHMENT OF STUDY CRITERIA

### Section 5.01 - Selection of Time Interval

The selection of an appropriate time interval pertains primarily to the sequential type of analysis; however, similar reasoning must be applied to selection of flow periods for non-sequential analysis. Time intervals of one month are usually adequate for non-sequential and preliminary sequential analyses. For more detailed studies, shorter routing intervals (generally ranging from a minimum interval of one week to a maximum of one month) will ordinarily be required. Only in exceptional cases will routing intervals of less than one week be practical for entire periods of record. Considerable work is involved with shorter intervals and the effects of time translation, which are usually ignored in conservation routing studies, become important with intervals of less than one week. Shorter intervals are necessary and should be used during flood periods or during periods when daily power fluctuations occur.

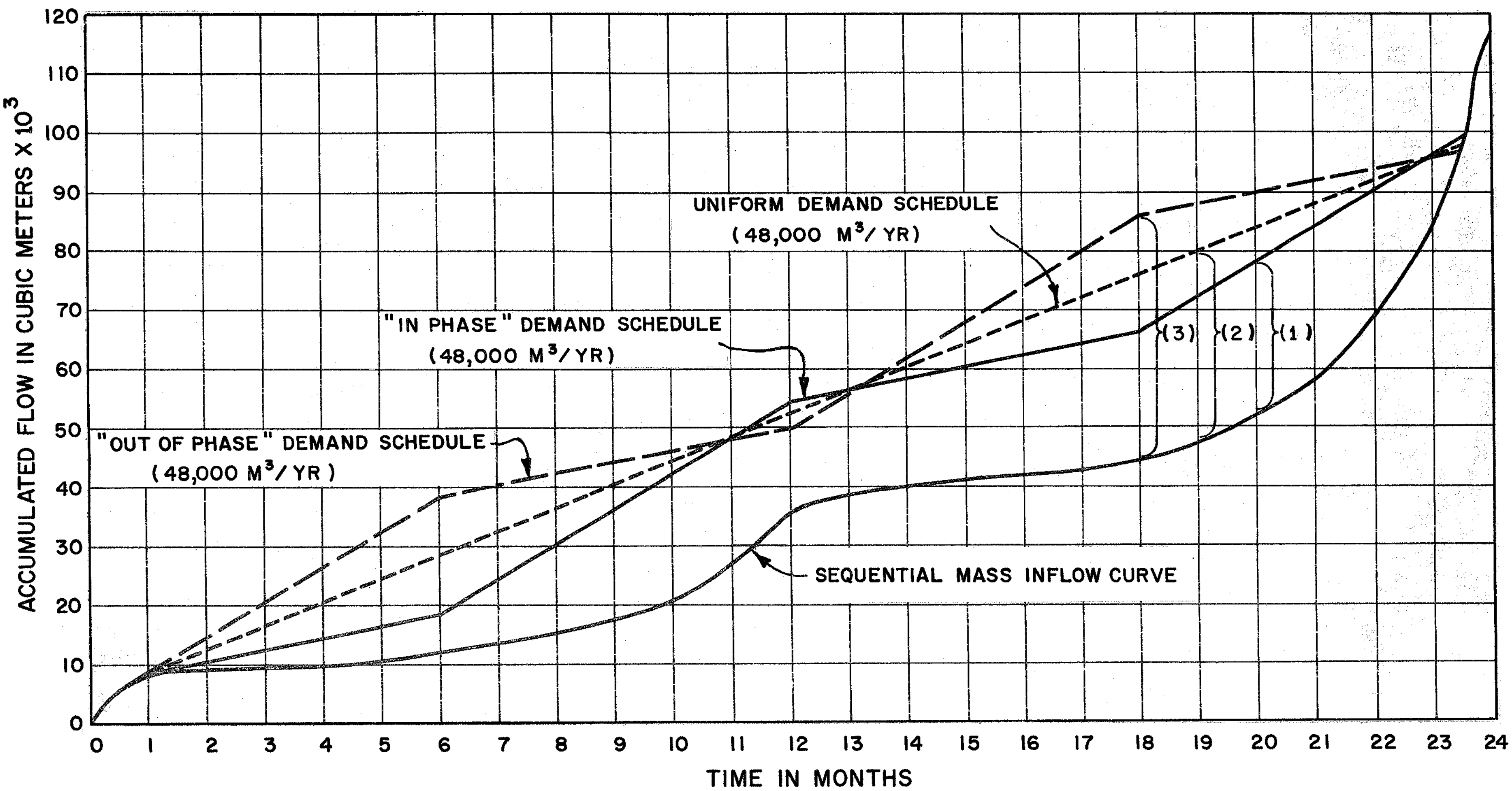
Ordinarily, when the use of short time intervals (one day or less) is necessary to obtain adequate definition of the conditions under study, the periods selected for analysis should exhibit critical combinations of hydrologic conditions and demand characteristics. For example, analysis of hourly power generation at a hydroelectric plant under peaking conditions might be studies for a one-week period where extremely low flows could be assumed to coincide with extremely high power demands. As a rule, studies involving short time intervals are supplementary to one or more studies of longer periods using longer time intervals. The results of the long-period study are often used to establish initial conditions such as initial reservoir storage for the selected periods of short-interval analysis.

In sequential conservation routing studies, the selection of a routing interval is dependent upon four major factors: (1) the demand schedule that will be utilized in determining the yield; (2) the accuracy required by the study objectives; (3) the data available for use in the study; and (4) the phase relationship between periods of high and low demands and high and low flow. If the water demand schedule is relatively uniform, it is ordinarily possible to estimate the amount of storage required for a specified yield by graphical analysis using the Rippl diagram or the non-sequential analysis discussed in Chapter 6. Demand schedules which show marked seasonal variations usually preclude the use of graphical techniques alone in determining storage requirements. This is especially true when the demand is a function which cannot be described in terms of a specific amount of water, as in the cases of hydroelectric power and water quality. In order to obtain accurate estimates of storage requirements when the demand schedule is variable, it is necessary to make sequential routing studies with routing intervals short enough to delineate important variations in the demand schedule. Graphical techniques may be utilized in obtaining a first estimate of storage requirements for the detailed sequential routing.

Errors may be introduced in storage determination by graphical techniques when an equivalent uniform demand schedule is assumed instead of using a known variable demand schedule. Such errors are illustrated in fig. 5.01. The variable demand schedule shown is relatively simple and thus does not accentuate the errors brought about by improper selection of routing interval. The uniform rate shown is equivalent to the selection of a one-year routing interval; the difference in storage requirements is the error that would result from selection of a one-year routing interval rather than a shorter interval that reflects the demand schedule variation.

As a general rule, if increased accuracy is desired, shorter routing intervals must be used. This is due to many factors, such as better definition of relationships between inflow and releases, and better estimates of average reservoir levels for evaporation and power-head calculations. Longer routing intervals tend to reduce the characteristic variations of streamflows, thus producing a "dampened" storage requirement. Ordinarily a monthly or seasonal routing interval is adequate for basic studies. However, when fluctuations in streamflows or demands have a significant effect on storage requirements, computations should be refined for critical portions of the studies, or shorter routing intervals should be used.

In general, the routing interval should not be shorter than the shortest period for which flow and demand data are available. Attempts to "manufacture" flow or demand data are usually time consuming and




(1) STORAGE REQUIRED TO YIELD 48,000 $m^3$/YR WHEN DEMAND SCHEDULE IS "IN-PHASE" WITH FLOW PATTERN = 26,500 $m^3$

(2) STORAGE REQUIRED TO YIELD 48,000 $m^3$/YR WHEN DEMAND SCHEDULE IS UNIFORM = 33,000 $m^3$

(3) STORAGE REQUIRED TO YIELD 48,000 $m^3$/YR WHEN DEMAND SCHEDULE IS "OUT-OF-PHASE" WITH FLOW PATTERN = 41,500 $m^3$

Fig. 5.01 Effects of demand sequence on storage requirements



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may create errors or give a false impression of accuracy unless reliable information is available for subdivision of basic data.

The selection of the flow interval for analysis by non-sequential methods is usually not as critical as for a sequential analysis. Since the non-sequential analysis is restricted to uniform demands, it does not produce results as accurate as those obtained by sequential methods; therefore, there is very little gain in accuracy with short intervals. Flow intervals of one month are usually suitable for non-sequential methods.

## Section 5.02 - Physical Constraints

Physical constraints which should be considered in storage-yield studies include: (1) maximum conservation storage available, (2) outlet capacities, and (3) channel capacities. If flood control is to be included as a project purpose, the maximum conservation storage feasible at a given site will be affected by the flood control analysis. Generally, the net storage available for conservation uses will be reduced if flood control is included as a project purpose.

## Section 5.03 - Priorities

In order to determine optimum yield in a multiple-purpose project, some type of priority system for the various purposes must be established. This is necessary when the competitive aspects of water use require a firm basis for an operating decision. Flood control usually has highest priority in a multiple-purpose project during actual operation; hence, during periods of flood control operations, conservation requirements might be reduced in order to provide the best flood-control operation. Although this volume is not concerned directly with flood-control operation or criteria, it is necessary to integrate flood control constraints with the conservation study to insure that operating conditions and reservoir levels for conservation purposes do not interfere with flood control operation. Priorities among the various conservation purposes vary with locale, water rights, and with the need for various types of water utilization. In multipurpose projects, every effort should be made to develop operation criteria which maximize the complementary uses for the various conservation purposes. Unless allocation of space to specific purposes is directed, specific allocations of space should be avoided in favor of operation rules based on total remaining storage.

## Section 5.04 - Storage Limitations

One of the reasons for making sequential conservation routing studies is to determine the effect of storage limitations on yield rates. Simplified yield methods cannot account for operational restrictions imposed by storage limitations in a multiple-purpose project. There are three primary storage zones, any or all of which may exist in a given reservoir project. The three zones may generally be described as: (1) flood control space, which is usually the uppermost storage space in the reservoir; (2) conservation storage, which is immediately below the flood control storage; and (3) reserve or dead storage, which is the lowest storage space in the reservoir. An additional space, called surcharge, exists between the top of the flood control space and the top of the dam and is reserved for storage of flood waters which the spillway is unable to pass. The boundaries between the storage zones and operational boundaries within the zones may be fixed throughout the year or they may vary from season to season as shown on fig. 5.02. The varying boundaries usually offer a more flexible operation plan which may result in higher yields for all purposes, although an additional element of chance is often introduced when the boundaries are allowed to vary. The purpose of detailed sequential routing studies is to produce an operating scheme and boundary arrangement which minimizes the chance of failure to satisfy any project purpose while optimizing the yield for each purpose. The three storage zones and the effect of varying their boundaries are discussed in the following paragraphs.

## Flood Control Storage

The inclusion of flood-control storage in a multiple-purpose project may adversely affect conservation purposes in two ways. First,

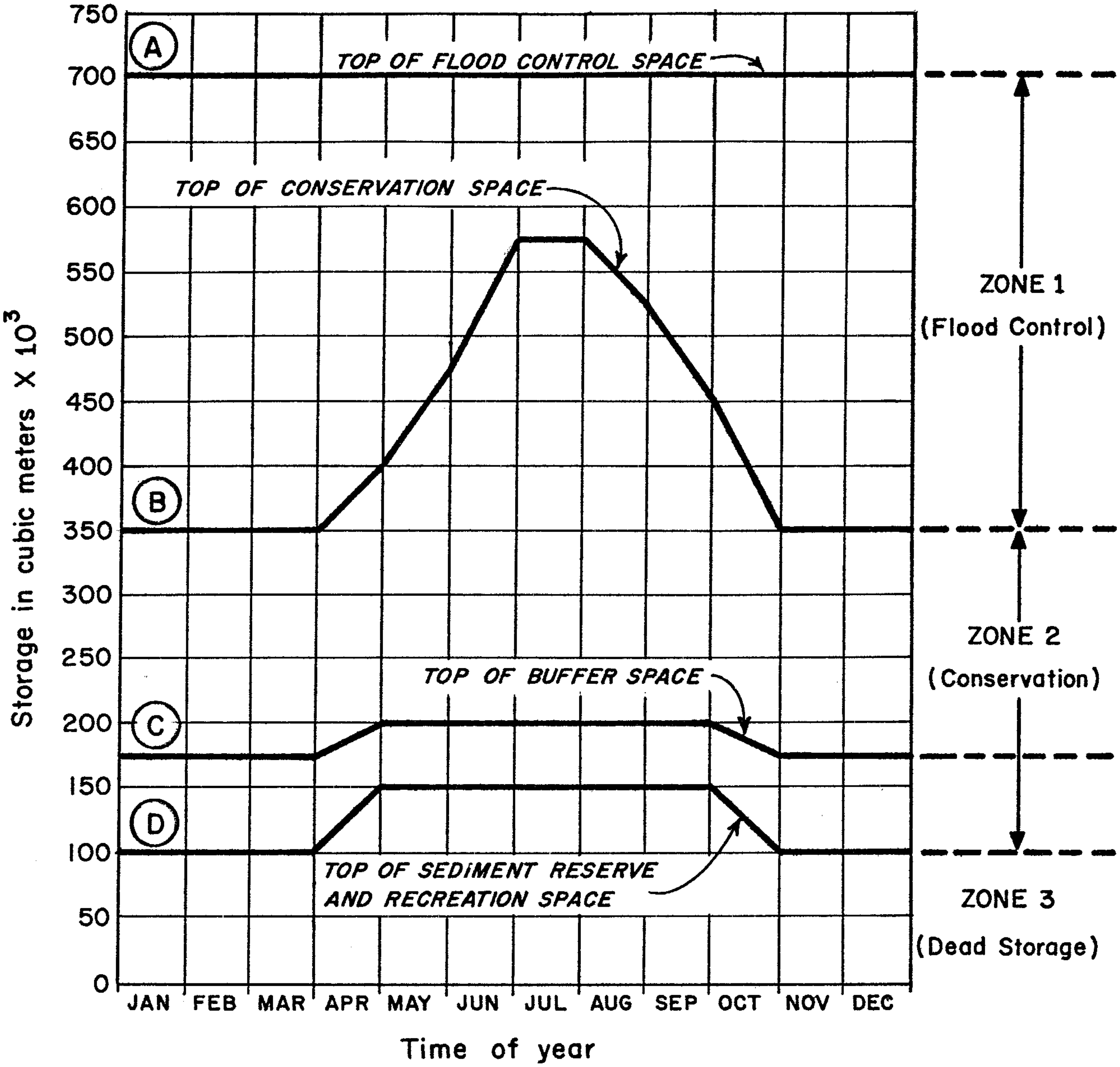


Fig. 5.02. Example of seasonally varying storage boundaries for a multipurpose reservoir

storage space which might otherwise be utilized for conservation purposes is reserved for flood-control usage, and second, flood control operation criteria may override conservation criteria with a resultant reduction or loss of conservation benefits. However, detailed planning and analysis of criteria for flood control and conservation operations can minimize such adverse effects.

Where competition between flood control and conservation requirements exists, but these requirements do not coincide time wise, the use of a seasonally varying boundary between flood control storage and conservation storage may be used to minimize the competition. The general procedure is to hold the top of conservation pool at a low level when conservation demands are not critical in order to reserve more storage space for flood control regulation. Then, as the likelihood of flood occurrence decreases, the top of conservation pool is raised to increase the storage available for conservation purposes. Operation criteria are then tested by detailed sequential routing for the period of recorded streamflow. Several alternative patterns and magnitudes of seasonal variations should be evaluated to determine the response of the storage-yield relationship and the flood control efficiency to the seasonal variation of the boundary. A properly designed seasonally varying storage boundary should not reduce the effectiveness of flood control storage to increase the conservation yield.

Flood control operation is generally simplified in conservation studies because the routing interval for such studies is frequently too long to adequately define the flood control operation. Nevertheless, flood control constraints should be observed insofar as possible. For example, channel capacities below the reservoir are considered for flood control release purposes, and storage above the top of flood control pool is not permitted.

## Conservation Storage

The conservation storage may be used to regulate minor floods in some multipurpose projects, as well as to supply water for conservation purposes. For this reason, a seasonally varying boundary between flood control storage and conservation storage is often advantageous to both flood control and conservation. In addition to seasonal variations in its upper boundary, the lower conservation storage boundary may also vary seasonally. If several conservation purposes of different priorities exist, there may be need for a buffer zone in the conservation storage. The seasonal variation in the boundary between conservation storage and buffer zone would be determined by the relationship between seasonal demands for the various purposes. Buffer storage may be required for one of two reasons. First, it may be used in the multipurpose projects to continue releases for a high priority purpose when normal conservation storage has been exhausted by supplying water for both high and low priority purposes; and, second, it may be used in a single-purpose project to continue releases at a reduced rate after normal conservation storage has been exhausted by supplying water at a higher rate. In either case, the boundary between the normal conservation storage and buffer storage is used to change the operation criteria. The proper location of this boundary and its seasonal variation are important factors in detailed sequential routing because of this change in operation criteria. The amount of buffer storage and consequently, the location of the seasonally varying boundary between the buffer zone and the remainder of the conservation storage must usually be determined by successive approximation in sequential routing studies. However, a simplified procedure which produces a satisfactory estimate in cases without seasonally varying boundaries is described in Chapter 6.

## Reserve or Dead Storage

Reserve or dead storage is the storage which is maintained in the reservoir for several purposes, such as providing for a recreation pool, head for power, a reserve for sedimentation, and sustenance of fish and wildlife. As a rule, the reservoir may not be drawn below the top of the reserve or dead storage. However, the top of the reserve may be allowed to vary seasonally in some instances. For example, if reserve storage is provided only to maintain a recreation pool, it might be permissible to withdraw water from the reserve storage for other conservation purposes during the season when there is little or no use of the reservoir for recreation. Should such a condition exist, it would be beneficial to examine the possible increase in yield for other purposes by means of detailed sequential routing.

## Section 5.05 - Effects of Conservation and Other Purposes

As previously indicated, the seasonal variation of demand schedules may assume an important role in the determination of required yield. The effect of seasonal variation is most pronounced when the varying demand is large with respect to other demands as is often the case when hydroelectric power or irrigation is a large demand item. The quantity of yield from a specified storage may be over-estimated by as much as 30% when a uniform yield rate is used in lieu of a known variable yield rate. Since detailed sequential routing is particularly adaptable to the use of variable demand schedules, every effort should be made to incorporate all known demand data into the criteria for routing. It should be emphasized that variable demand schedules often complicate the analysis of reservoir yield to the extent that it is impossible to accurately estimate the maximum yield or the optimum operation by approximate methods. Thus, successive trials using detailed sequential analysis must often be used to determine maximum yield. Computer programs such as those discussed in Chapter 6 are most useful for successive sequential routings. The project purposes which often require analysis of seasonal variations in demand are discussed in more detail in the following subparagraphs.

### Low-Flow Regulation

The operation of a reservoir for low flow regulation at a downstream control point is difficult to evaluate without a detailed sequential routing, because the operation is highly dependent upon the flows which occur between the reservoir and the control point (herein called the "intervening flow"). Since these flows can vary significantly, a yield based on long-period average intervening flows can be subject to considerable error. A detailed sequential routing, in which allowance is made for variations of intervening flows within the routing interval, produces a more reliable estimate of storage requirements for a specified yield, and reduces the chance of over-estimating a firm yield. Firm yield is the amount of water available for a specific use on a dependable basis during the life of a project. Ordinarily the yield and the corresponding operation of a reservoir for low-flow regulation is determined by detailed sequential routing of the critical period and several other periods of low flow. The entire period of recorded streamflow is usually not evaluated unless summary-type information is needed for functions such as power.

## Diversion and Return Flows

The analysis of yield for diversions is complicated by the fact that diversion requirements may vary from year to year as well as from season to season. Furthermore, the diversion requirements may be stated as a function of the natural flow and water rights rather than as a fixed amount. In addition, diversion amounts may often be reduced or eliminated when storage in the reservoir reaches a certain critical low value. When any one of these three items is important to a given reservoir analysis, the use of simplified methods must be abandoned in favor of the more detailed sequential techniques. These complications often require that a detailed sequential analysis for the entire period of flow record be made in order to determine accurately the yield and the reservoir operation criteria. Coordination of the operation criteria for other purposes with the diversion requirements may also be achieved with the detailed sequential analysis results.

Many of the considerations described in the previous paragraph also pertain to imports and exports of water among river basins. However, the import or export of water can also affect other purposes. In the case of hydroelectric power, there may also be imports and exports of energy which would affect the analysis. Usually imports and exports will vary annually as well as seasonally so that period-of-record sequential routings are necessary.

## Water Quality Control

Inclusion of water quality control and management as a project purpose almost always dictates that sequential routing studies be used to evaluate project performance. Practically every variable under consideration in a water quality study will vary seasonally. Among the variables which must be considered in a water quality study are: (a) the variation in quality of the inflow; (b) the subsequent change in quality of the reservoir waters due to inflow quality and evaporation; (c) the variation in quality of natural streamflow entering the stream between the reservoir and the control station; (d) the variation in effluents from treatment plants and storm drainage outflow between the reservoir and the control station; and (e) the variation in quality requirements at the control station. Accurate evaluation of project performance must consider all of these variations which are pertinent to water quality control.

There are several quality parameters which may require study, and each parameter introduces additional variations which should be evaluated. For example, if temperature is an important parameter, the level of the reservoir from which water is released should be considered in addition to the above variables. Likewise, if oxygen content is important, the effects of release through power units versus release through conduits must be evaluated.

### Hydroelectric Power Generation

If hydroelectric power is included as a project purpose, detailed sequential routings are necessary to develop operation criteria, to coordinate power production with other project purposes, and to determine the project's power potential. As a rule, simplified methods are usable for power projects only for preliminary or screening studies or when there is little likelihood of justifying power as a project purpose. Since power production is a function of both head and flow, a detailed sequential study is usually required when the conservation storage is relatively large and the head can be expected to fluctuate erratically over a wide range.

Determination of firm power or firm energy is usually based on sequential routings over the critical period. Various operational plans are used in an attempt to maximize power output while meeting necessary commitments for other project purposes. When the optimum output is achieved, a rule curve for operation can be developed. The rule curve is based upon the power output itself and upon the plan of operation followed to obtain the maximum output. Critical period analysis and rule curve development are described in Chapter 7. Additional sequential routings for the entire period of flow record are then made using the rule curve developed in the critical period studies. These routings are used to coordinate power production with flood control operation and to determine the average annual potential energy available from the project.

In areas where hydroelectric power is used primarily for peaking purposes, it is extremely important that storage requirements be defined as accurately as possible, since the available head during a period of peak demand is required to determine the peaking capability of the project. An error in storage requirements, on the other hand, can adversely affect the head with a resultant loss of peaking capability.

Tailwater elevations are also of considerable importance in power studies because of the effect of head on power output. Several factors which may adversely affect the tailwater elevation at a reservoir are:

(a) construction of a re-regulation reservoir below the project under consideration; (b) high pool elevations at a project immediately downstream from the project under consideration; and (c) backwater effects from another stream if the project is near the confluence of two streams. If any of these conditions exist, the resultant tailwater conditions should be carefully evaluated. For projects in which peaking operation is anticipated, an assumed "block-loading" tailwater should be used to determine reservoir releases for the sequential reservoir routing. The "block-loading" tailwater elevation is defined as the tailwater elevation resulting from sustained generation at or near the plant's rated capacity which represents the condition under which the project is expected to operate most of the time. Although in reality the peaking operation tailwater would fluctuate considerably, the use of "block-loading" tailwater elevation insures a conservative estimate of storage requirements and available head.

The advent of reversible pump-turbines has enhanced the feasibility of the pumped storage type of hydroelectric development. In a development of this type, reversible pump-turbines are included in the powerhouse along with conventional generating units, and an afterbay is constructed below the main dam to retain water for pumping during nongenerating periods. Sequential routing studies are required in analyses of this type because of the need to define: (a) storage requirements in the afterbay; (b) pumping requirements and characteristics; and (c) the extent to which such a plan should be developed. Many of the existing and proposed pumped storage projects in the United States, however, are single purpose projects whch do not have conventional units and often utilize off channel forebays.

## Section 5.06 - Shortage Tolerances

The shortage tolerances for various project purposes can greatly influence the amount of storage required to produce a "firm" yield. These tolerances differ considerably for different project purposes throughout the world. The relative importance of shortage tolerances for various purposes can be seen by reviewing design criteria for individual project purposes.

Hydroelectric power facilities are usually designed for the most adverse observed streamflow conditions. However, as discussed by Beard (reference 5), the most critical period observed in a historical streamflow record can be far different from conditions that might occur in the future. For a system analysis of hydroelectric power projects, shortages for individual reservoir projects can be tolerated as long as the overall system power production is not diminished. Hydroelectric power requirements can thus be generated from reservoirs that are not experiencing a severe drawdown, thereby taking advantage of the hydrologic diversity which exists in the system.

Municipal and industrial (M & I) shortages fall into the same general category as hydroelectric power since shortages are generally intolerable for purposes such as drinking water. However, some reduction in the quantity of water required for M&I purposes can be tolerated without serious economic effects by reducing some of the less important uses of water such as lawn watering. These reductions may be in the order of 10 percent. Most designs for M&I storage are based on supplying the firm yield during the most critical period of the flow record, with some reserve storage for use in the event of unprecedented droughts. Another approach to shortage tolerances for M&I is to design on a shortage probability basis, for example, providing the storage required to supply the full demand with a 1 percent or 2 percent chance of shortage in any given year.

Shortages for irrigation are acceptable under certain conditions. Some designers may accept one 50 percent annual shortage during the economic life of a proposed project, while others may accept 5 or 10 shortages of as much as 10 percent or 20 percent without adversely affecting the economics of the project.

Water quality shortage tolerances are usually expressed in terms of shortage probability. A 10 percent or a 20 percent probability of shortage during any year is often considered reasonable in determining the storage requirements for water quality purposes.

## Section 5.07 - Shortage Index

A general approach to shortage definition is to use some sort of a shortage index. The shortage index, as defined in reference 27, is equal to the sum of the squares of the annual shortages over a 100-year period, when each annual shortage is expressed as a ratio to the annual requirements, as shown below:

$$SI = \frac{100 \sum_{i = N}^{i = N} \left[\frac{S_A}{D_A}\right]^2}{N}$$

where: SI = Shortage Index

$S_A$ = Annual Shortage (annual demand volume minus annual volume supplied)

$D_A$ = Annual Demand Volume

N = Number of years in routing study

This shortage index reflects the observation that economic and social effects of shortages are about proportional to the square of the degree of shortage. For example, a shortage of 40 percent is assumed to be four times as severe as a shortage of 20 percent. Similarly, as illustrated in Table 5.01, shortages of 50 percent during 4 out of 100 years are assumed four times as severe as shortages of 10 percent during 25 out of 100 years.

Table 5.01 Illustration of shortage index

| Shortage Index | No. of Annual Shortages Per 100 Years | Annual Shortage ($S_A/D_A$) In % |
|---|---|---|
| 1.00 | 100 | 10 |
| 1.00 | 25 | 20 |
| 1.00 | 4 | 50 |
| .25 | 25 | 10 |
| .25 | 1 | 50 |

The shortage index has considerable merit over shortage frequency alone as a measure of severity, since shortage frequency considers neither magnitude nor duration. Reference 11 describes a screening study in which the shortage index was used in analysis of storage requirements. The shortage index can be multiplied by a constant to obtain a rough estimate of associated damages.

There is a definite need for additional criteria delineating shortage acceptability for various services under different conditions. These criteria should be based on social and economic costs of shortages in each individual project study, or certain standards could be established for the various services and conditions. Such criteria should account for degree of shortage as well as expected frequency of shortage.

Chapter 6

# Technical Procedures

TECHNICAL PROCEDURES

## Section 6.01 - Simplified Methods

Many simplified methods have been proposed and are being used for preliminary type analyses; however, these methods often sacrifice accuracy in the interest of saving time and funds. Although some of the more recent simplified methods rely upon the use of computers to produce more reliable results than usually obtained manually, it would probably be advantageous to devote that computer time to detailed sequential analysis when adequate data are available.

The use of simplified techniques which do not consider sequential variations in streamflow or demand is usually discouraged since relatively large errors can result. However, if it can be determined that a nonsequential analysis is appropriate, the procedures described herein will generally produce satisfactory results. These procedures can also be used to determine first estimates of storage requirements or yield rates for a more detailed analysis.

The most commonly used simplified sequential method is the sequential mass curve analysis, often referred to as the Rippl Method. This method produces a graphical estimate of the storage required to produce a given yield, assuming that the seasonal variations in demand are not significant enough to prohibit the use of a uniform draft (demand) rate.

The sequential mass curve is constructed by accumulating inflows to a reservoir site throughout the period of record and plotting these accumulated inflows versus the sequential time period as illustrated in fig. 6.01. The desired yield rate, in this example 38 million m$^3$/year, is represented by the slope of a straight line. Straight lines are then constructed parallel to the desired yield rate and tangent to the mass curve at each low point (line $\overline{ABC}$) and at the preceding high point that gives the highest tangent (line $\overline{DEF}$). The vertical distance between these two lines (line $\overline{BE}$) represents the storage required to provide the desired yield during the time period between the two tangent points (points D and B). The maximum vertical difference in the period is usually adopted as the required storage.

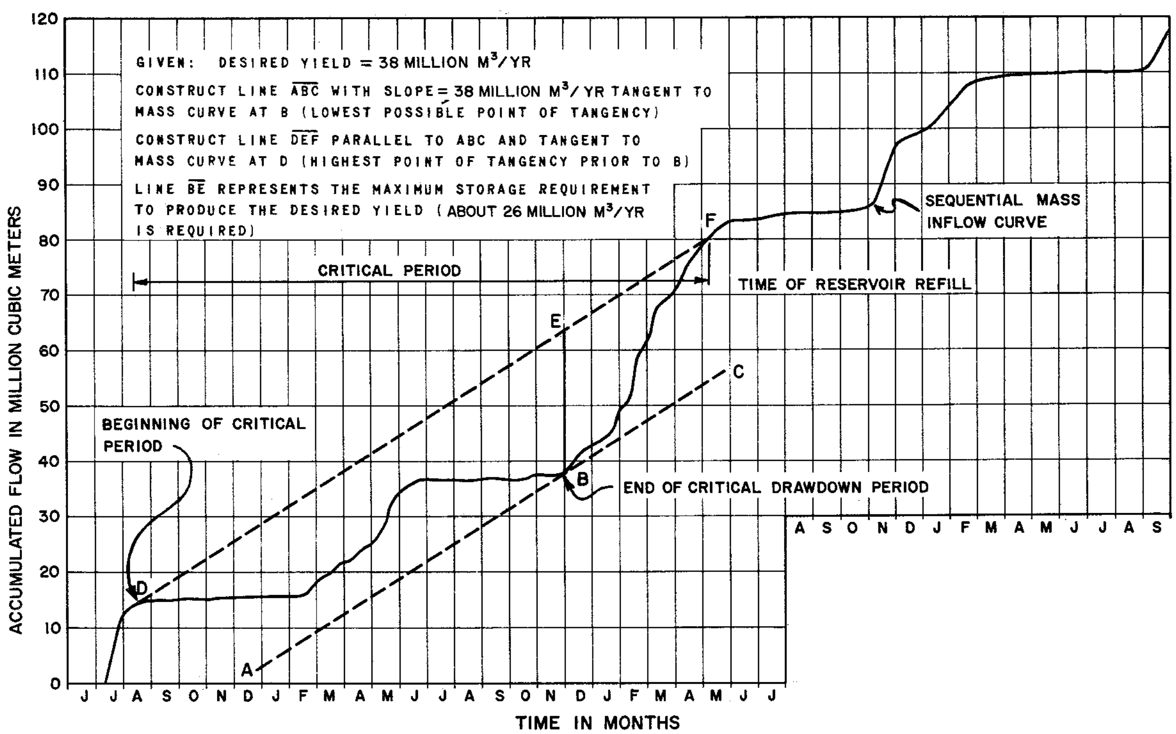


Fig. 6.01 Storage determination using sequential mass curve

## Section 6.02 - Non-Sequential Mass Curve Analysis

The Sequential Mass Curve method does not directly indicate the relative frequency of a shortage. However, by using non-sequential methods similar to those described in reference 20, a curve of yield vs. shortage frequency can be determined. The simplified method presented in this manual differs from the method presented in reference 20 in three ways:

1. The median plotting positions are used as outlined in reference 4. These correspond to sample size equal to the number of continuous years of record less the duration of drought considered.

2. Streamflow records are sometimes supplemented by the use of simulated streamflow.

3. The non-sequential mass curves are smoothed graphically to insure mutual consistency.

The historical flows, supplemented by simulated flows where needed, are used to determine frequency tables of independent low-flow events for several durations. The Hydrologic Engineering Center's program entitled "Partial-Duration-Independent Low Flow Events" (723-G1-L2290) has been developed for this purpose. The program is included as Appendix I of this volume. A series of low-flow events for a particular duration is selected by computing and arranging in order of magnitude, the independent minimum-flow rates for that duration for the period of record. The procedure utilized in the program can also be used for manual computation, but the time required for complete analysis is often prohibitive. Table 6.01

Table 6.01 - Sample computation of independent low-flow events, partial duration-6-month volumes

| YEAR | MONTH | MONTHLY FLOW VOLUME ($m^3x10^6$) | 6-MONTH VOLUME ($m^3x10^6$) | RANK |
|---|---|---|---|---|
| 1958 | Jan | 72.8 | | |
| | Feb | 138.1 | | |
| | Mar | 248.2 | | |
| | Apr | 164.3 | | |
| | May | 71.4 | | |
| | Jun | 30.8 | 725.6 | |
| | Jul | 16.6 | 669.4 | ** |
| | Aug | 15.5 | 546.8 | ** |
| | Sep | 10.8 | 309.4 | ** |
| | Oct | 9.2 | 154.3 | ** |
| | Nov | 6.6 | 89.5 | ** |
| | Dec | 11.2 | 69.9 | 3 |
| 1959 | Jan | 158.1 | 211.4 | ** |
| | Feb | 177.4 | 373.3 | ** |
| | Mar | 156.3 | 518.8 | ** |
| | Apr | 173.3 | 682.9 | ** |
| | May | 108.8 | 785.1 | ** |
| | Jun | 47.0 | 820.9 | |
| | Jul | 22.9 | 685.7 | |
| | Aug | 15.3 | 523.6 | |
| | Sep | 12.3 | 379.6 | |
| | Oct | 10.0 | 216.3 | |
| | Nov | 9.7 | 117.2 | |
| | Dec | 16.5 | 86.7 | |
| 1960 | Jan | 138.2 | 202.0 | |
| | Feb | 204.9 | 391.6 | |
| | Mar | 199.6 | 578.9 | |
| | Apr | 195.9 | 764.8 | |
| | May | 144.8 | 899.9 | |
| | Jun | 56.5 | 939.9 | |
| | Jul | 22.4 | 824.1 | |
| | Aug | 13.7 | 632.9 | |
| | Sep | 11.0 | 444.3 | |
| | Oct | 9.4 | 257.8 | |
| | Nov | 6.5 | 119.5 | |
| | Dec | 11.0 | 74.0 | |
| 1961 | Jan | 100.1 | 151.7 | |
| | Feb | 238.9 | 376.9 | |
| | Mar | 199.5 | 565.4 | |
| | Apr | 195.4 | 751.4 | |
| | May | 129.2 | 874.1 | |
| | Jun | 45.9 | 909.0 | |
| | Jul | 25.0 | 833.9 | |
| | Aug | 15.3 | 610.3 | |
| | Sep | 14.1 | 424.9 | |
| | Oct | 11.9 | 241.4 | |
| | Nov | 9.5 | 121.7 | |
| | Dec | 14.3 | 90.1 | |
| 1962 | Jan | 117.6 | 182.7 | |
| | Feb | 207.5 | 374.9 | |
| | Mar | 230.2 | 591.0 | |
| | Apr | 176.7 | 755.8 | |
| | May | 123.5 | 869.8 | |
| | Jun | 52.6 | 908.1 | |
| | Jul | 19.1 | 809.6 | ** |
| | Aug | 14.9 | 617.0 | ** |
| | Sep | 10.9 | 397.7 | ** |
| | Oct | 8.4 | 229.4 | ** |
| | Nov | 6.6 | 112.5 | ** |
| | Dec | 9.8 | 69.7 | 2 |
| 1963 | Jan | 130.6 | 181.2 | ** |
| | Feb | 166.6 | 332.9 | ** |
| | Mar | 235.1 | 557.1 | ** |
| | Apr | 194.0 | 742.7 | ** |
| | May | 92.3 | 828.4 | ** |
| | Jun | 41.2 | 859.8 | |
| | Jul | 16.1 | 745.3 | |
| | Aug | 12.0 | 590.7 | |
| | Sep | 12.3 | 367.9 | |
| | Oct | 11.9 | 185.8 | |
| | Nov | 13.4 | 106.9 | |
| | Dec | 23.5 | 89.2 | |
| 1964 | Jan | 166.0 | 239.1 | |
| | Feb | 159.8 | 386.9 | |
| | Mar | 178.5 | 553.1 | |
| | Apr | 193.6 | 734.8 | |
| | May | 91.0 | 812.4 | |
| | Jun | 34.8 | 823.7 | |
| | Jul | 16.3 | 674.0 | ** |
| | Aug | 10.0 | 524.2 | ** |
| | Sep | 9.6 | 355.3 | ** |
| | Oct | 8.2 | 169.9 | ** |
| | Nov | 8.9 | 87.8 | ** |
| | Dec | 14.1 | 67.1 | 1 |
| 1965 | Jan | 90.5 | 141.3 | ** |
| | Feb | 269.9 | 401.2 | ** |
| | Mar | 199.6 | 591.2 | ** |
| | Apr | 187.0 | 770.0 | ** |
| | May | 107.6 | 868.7 | ** |
| | Jun | 36.5 | 891.1 | |
| | Jul | 17.3 | 817.9 | ** |
| | Aug | 10.7 | 558.7 | ** |
| | Sep | 9.5 | 368.6 | ** |
| | Oct | 8.4 | 190.0 | ** |
| | Nov | 8.0 | 90.4 | ** |
| | Dec | 18.2 | 72.1 | 4 |

**Not considered after selection of the numbered volume in order to assure independence of subsequent selections.



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shows the tabulation and ranking procedure for manual analysis of a set of 6-month duration low-flow volumes.

In this case, the four lowest, independent 6-month volumes are given their respective rankings, and plotting positions are then determined. The plotting position (in percent) is calculated for the lowest volume (described in reference 4) as follows:

$$P = 1 - (.5)^{1/N} \tag{6-1}$$

where:

P = plotting position of lowest 6-month volume

N = number of years.

Although 8 years of monthly volumes are tabulated, the effective record for 6-month volume analysis is 7 years, 7 months (e.g., 8 years minus 5 months equals 7.58 years). Therefore, P is calculated as:

$$P = 1 - (.5)^{1/7.58}$$
$$= 1 - (.9128)$$
$$= .0872 \text{ or } 8.72 \text{ percent.}$$

Plotting positions (PP) for volumes other than the lowest are then calculated by:

$$PP = .0872 + (\Delta R)\left(\frac{\Delta P}{\Delta I}\right) \tag{6-2}$$

where:

$\Delta R$ = Rank number -1

$$\Delta P = (.5)^{1/N} - P$$
$$= .9128 - .0872$$
$$= .8256$$

$\Delta I = 7.58 - 1 = 6.58$

The number of events to be ranked is limited to the smaller of the following two limits:

(1) Limit 1 = N/n,

where: N = number of months in record

n = number of months in duration

(in this example = 96/6 = 16).

(2) Limit 2 = R,

where R is the rank of the last event with a plotting position less than 50% (in this example, R = 4, as shown below).

The 6-month volumes and their respective rankings are tabulated as follows:

| Rank | 6-Month Volume ($m^3X10^6$) | Plotting Position (percent) |
|---|---|---|
| 1 | 67.1 | 8.72 |
| 2 | 69.1 | 21.27 |
| 3 | 69.9 | 33.82 |
| 4 | 72.1 | 46.37 |

After the frequency tables of independent low-flow events are computed for various durations, low-flow frequency curves are obtained by plotting the average flow in cumecs on log probability paper. Curves are then carefully drawn for various durations. An example of such low-flow frequency curves for 35 years of data is illustrated in fig. 6.02.

Minimum runoff-duration curves (fig. 6.03) for various frequencies are obtained by plotting points from the low-flow frequency curves on logarithmic paper. The flow rates in cumecs, illustrated in fig. 6.02 are converted to

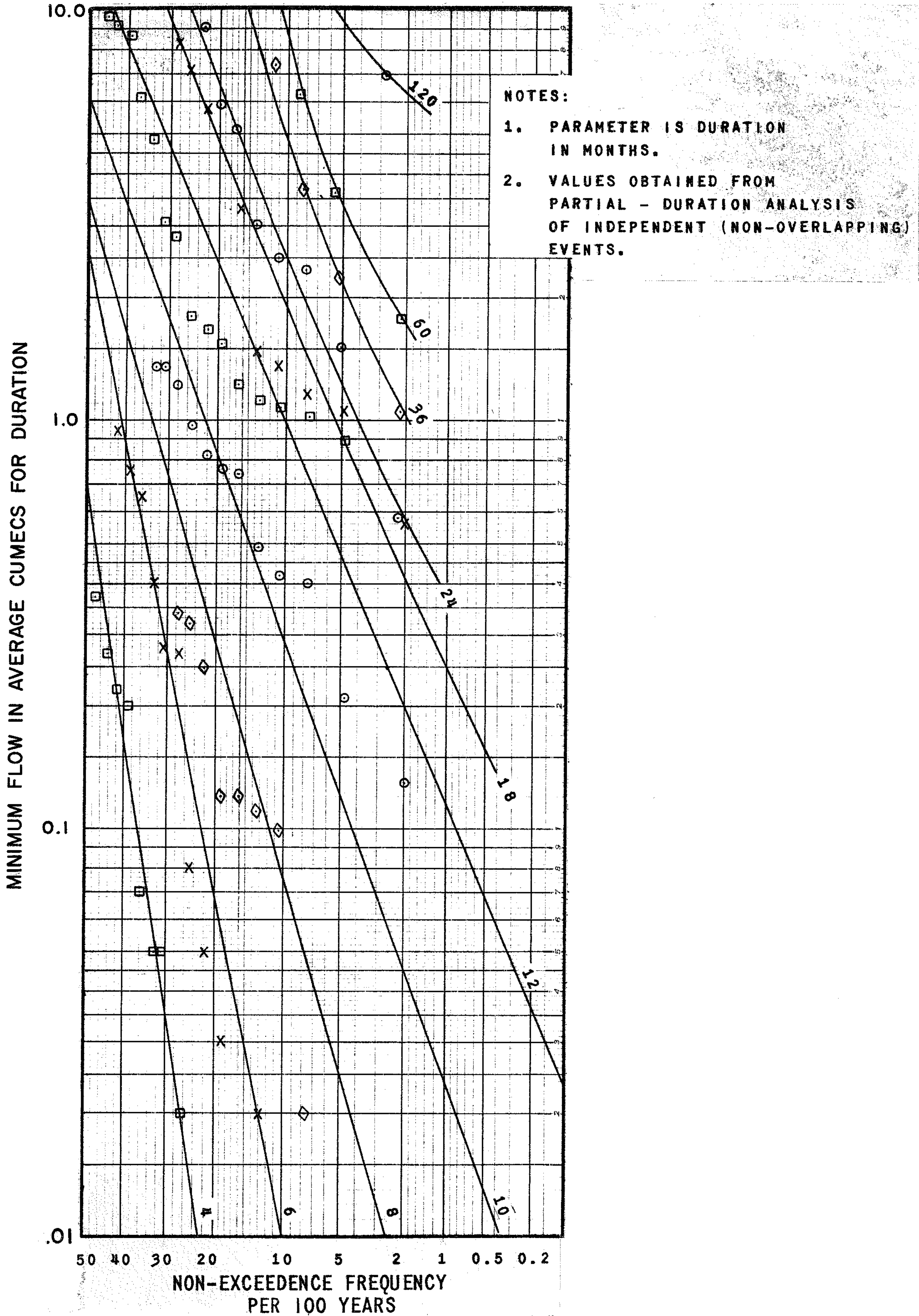


Fig. 6.02 Low-flow frequency curves based on 35 years of historical flows

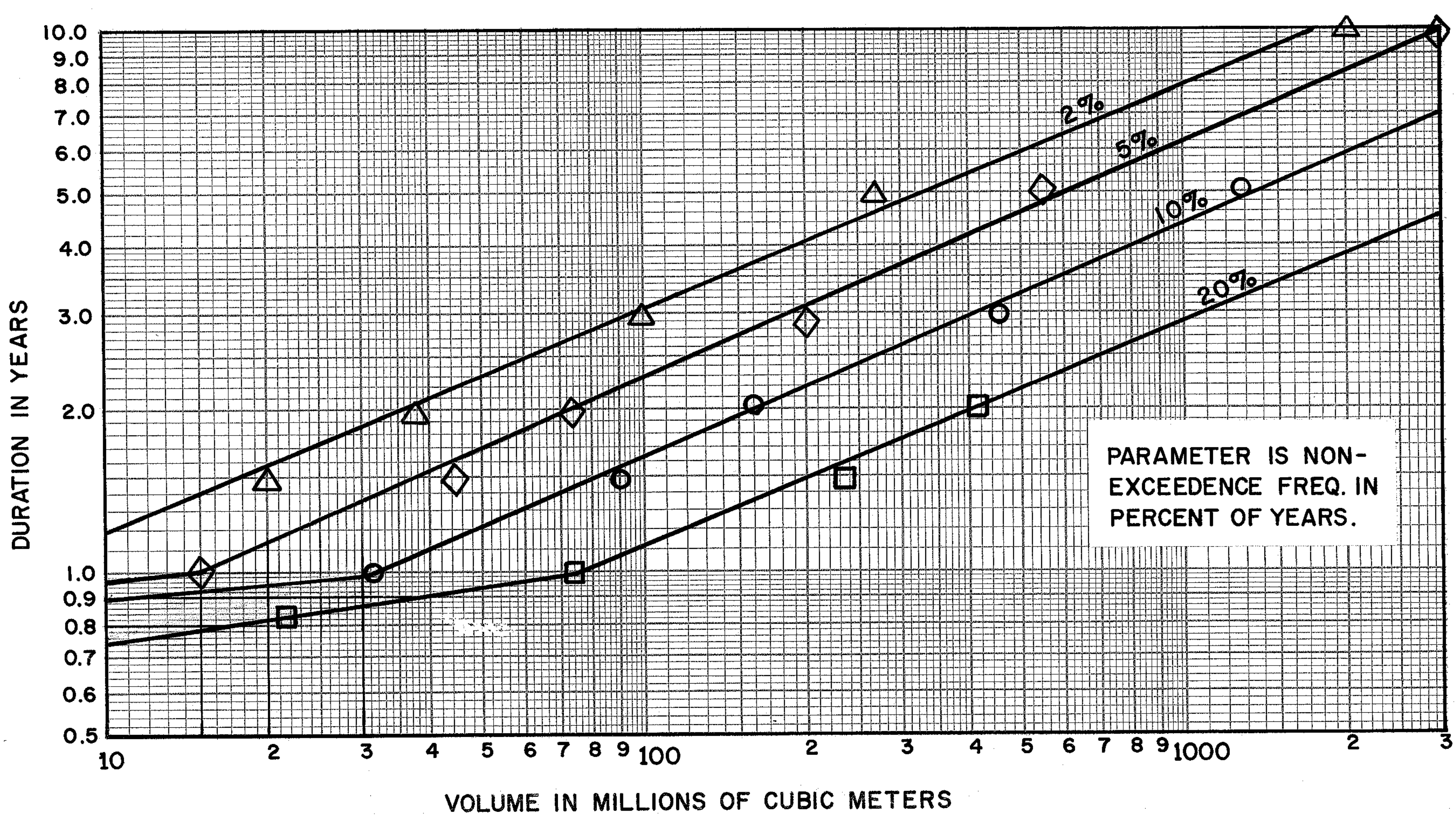


Fig. 6.03 Minimum runoff-duration curves based on fig. 6.02

volumes for fig. 6.03 (millions of cubic meters in this example). The logarithmic scales simply permit more accurate interpolation between durations represented by the frequency curves. The nonsequential mass curve (fig.6.04) is developed by selecting the desired volume-duration curve (fig. 6.03) and plotting this curve on arithmetic grid. The desired yield is then used to determine the storage requirement for the reservoir. The storage requirement is determined by drawing a straight line, with slope equivalent to the required gross yield, and by plotting this line tangent to the mass curve. The absolute value of the negative vertical intercept represents the storage requirement. The application of this procedure is severely limited in the case of seasonal variations in runoff and yield requirements, because the nonsequential mass curve does not reflect the seasonal variation in streamflows, and the tangent line does not reflect seasonal variations in demand. Hence, storage requirements determined by this method can be erroneous.

The low-flow frequency curves can be constructed with much less adjustment and with more reliability when simulated streamflows are used. The effects of using simulated flows are illustrated in the comparison between fig. 6.05, in which 500 years of simulated flow data were used to construct the low-flow frequency curves, and fig. 6.02 in which 35 years of historical data were used. Care must be exercised in the interpretation of fig. 6.02 and 6.05 since the abscissa in both figures is "non-exceedence frequency per 100-years," or the number of events within 100-years that have a flow equal to or less than the indicated flow.

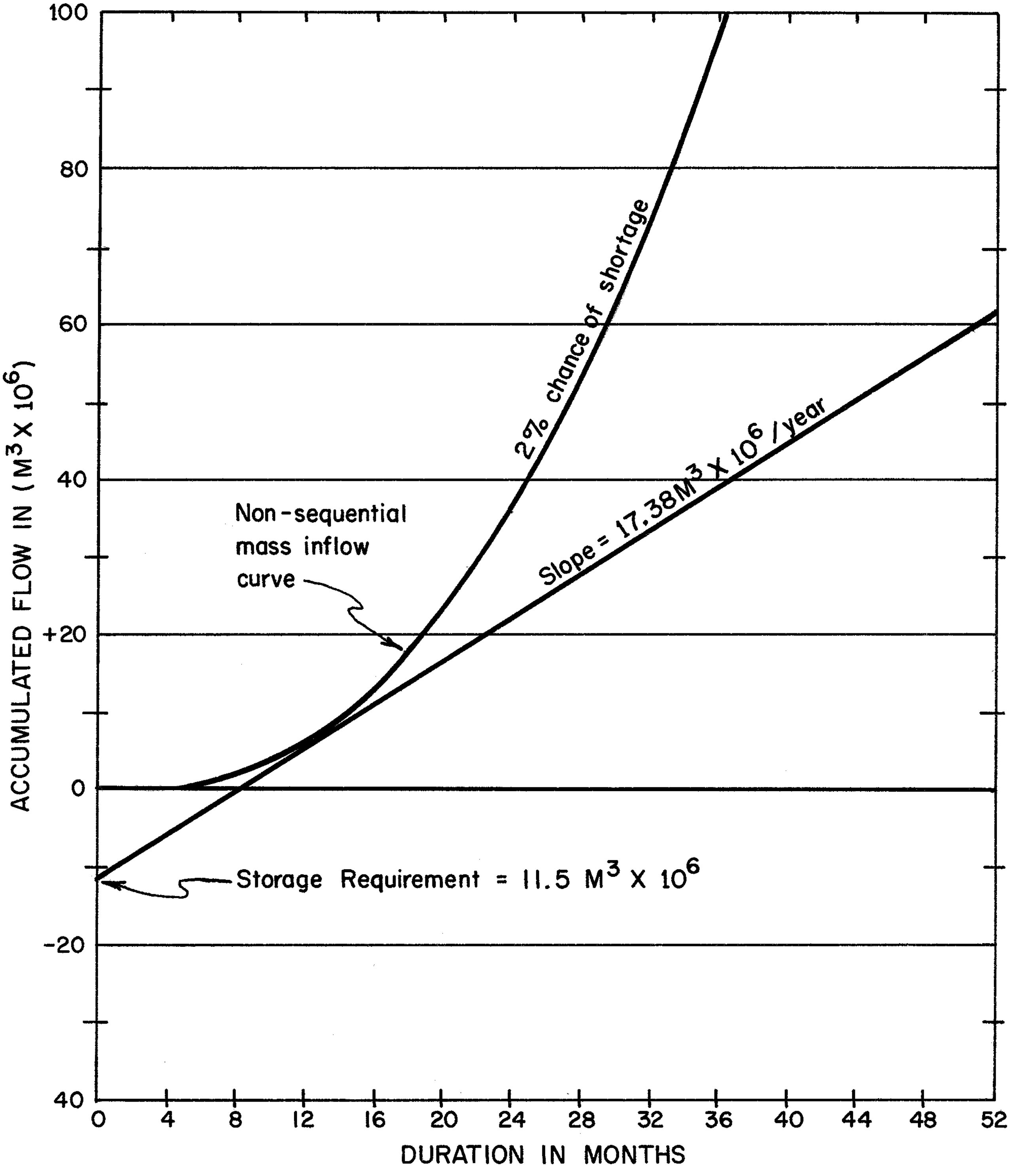


Fig. 6.04 A non-sequential mass curve developed from fig. 6.03

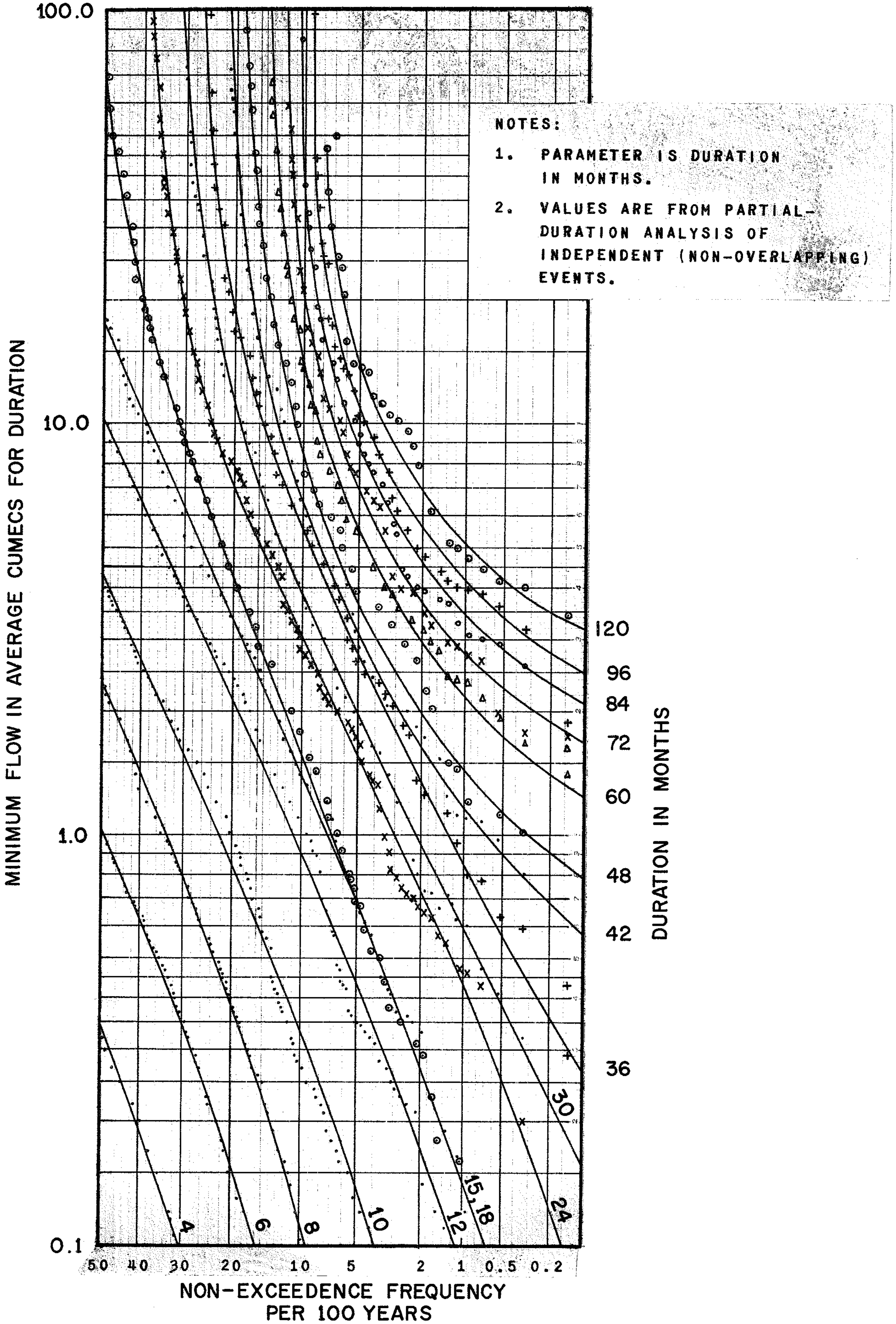


Fig. 6.05 Low-flow frequency curves based on 500 years of simulated flows

Thus, when low-flow durations in excess of one year are evaluated, the above terminology cannot be used interchangeably with probability. For instance, during a 100-year period, the maximum number of independent events of 120 months (10-years) duration is 10. Therefore, the 120-month duration curve cannot cross the value of 10 on the "non-exceedence frequency per 100-year" scale.

Since computation of reservoir yield by this simplified method for various reservoir storages and various frequencies may be required, a computer program has been developed using a procedure described in reference 14. Several storage-yield curves from this program for various non-exceedence frequencies are illustrated in fig. 6.06.

## Section 6.03 - Detailed Sequential Analysis

Sequential analysis is currently the most accepted method of determining reservoir storage requirements in the United States. Many simplified methods have been proposed and are being used for preliminary type analysis; however, these methods are giving way to the more sophisticated computer approaches. In many instances, the computer provides more accurate answers at a cost equal to or below the cost of preliminary estimates.

A computer program entitled "Reservoir Yield" (723-G2-L2400) has been developed by The Hydrologic Engineering Center (reference 24). This program performs multipurpose reservoir routings for a single reservoir op-

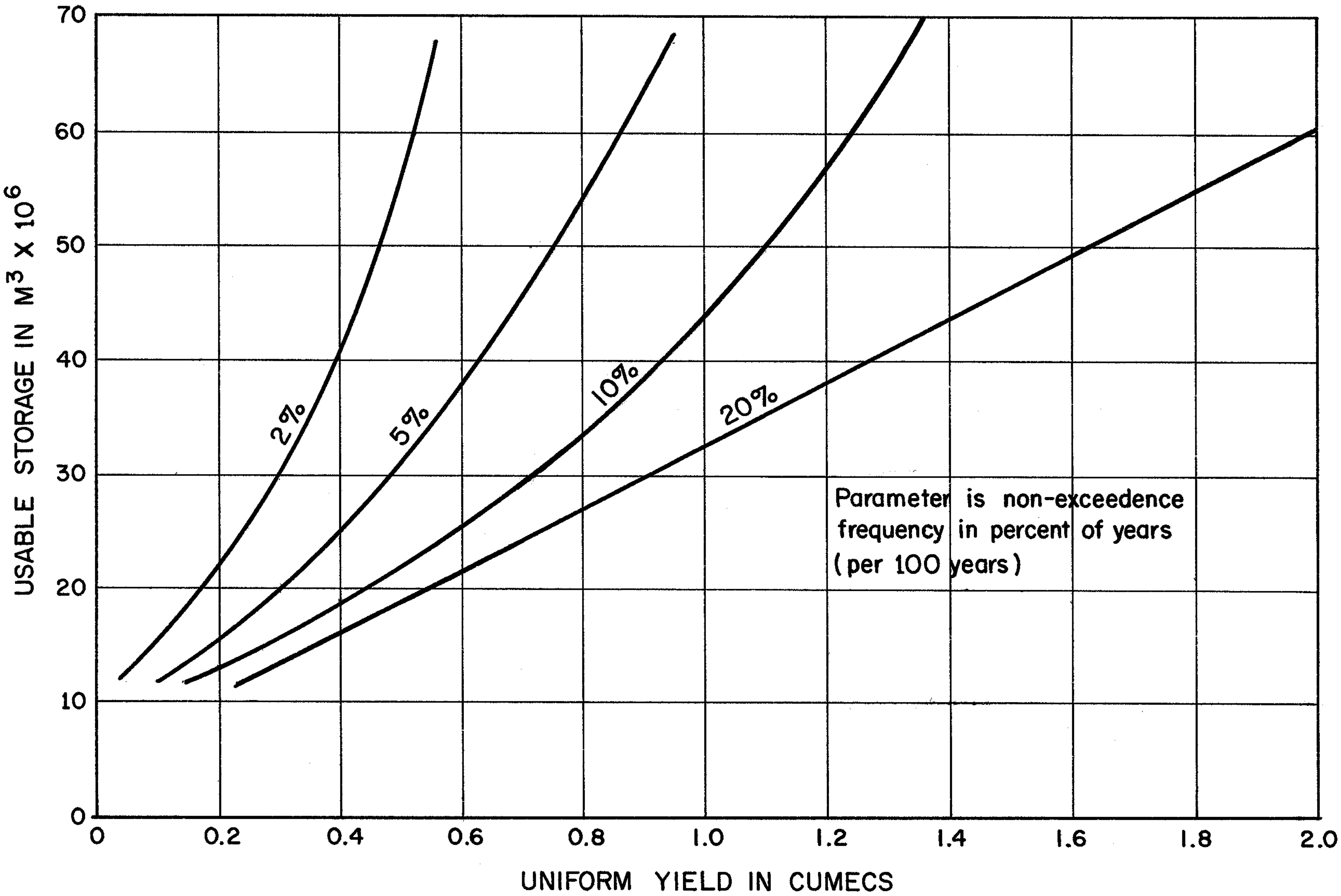


Fig. 6.06 Storage - yield curves, results of computer program described in reference 13

erating for services at the reservoir and at one downstream control point. The releases from the reservoir are determined by the individual requirements for many purposes. Reservoir releases may be controlled at the dam site by hydroelectric power requirements, downstream control for flow, diversion, water rights or quality. Additional releases may be made to a pipeline directly from the reservoir. The downstream flow requirement determines the reservoir releases needed to supplement runoff from the intervening area.

A computer program for conservation multi-reservoir routings (HEC-3) has also been developed by the Center to determine yields for reservoir projects that are not independent (discussed in IHD Volume 9, "Water Resource System Analysis"). Another program, HEC-5C has also been developed to operate for both flood control and conservation requirements (IHD Volume 7, "Reservoir Operation for Flood Control").

An example of manual sequential routings is shown in table 6.02. The routing computations illustrate the general procedures used in the computer program "Reservoir Yield" (single reservoir operation only), and generally reflect the detailed step-by-step procedures required by hand methods. The example problem is for a single reservoir which serves several requirements at the dam and at one downstream point. Some of the columns shown would not be used in normal hand computations, but are included to facilitate a comparison with printout from the Reservoir Yield program.

It is desirable to show operations for each period on a single sheet so that each controlling factor may be readily examined. Therefore, a suitable form for the particular operation, such as table 6.02, should be developed. Basic data and reservoir operational requirements are listed in the appropriate columns. Each column is described in detail in pages 2 through 4 of table 6.02. Routing computations for each period progress from left to right on the example form. All columns through 17 must be determined for each period. Except for basic data, determination of columns 18 through 31 are optional depending on controlling requirements. Columns 32 through 34 are the accomplishments for which the reservoir is operating.

Unless the controlling factor is obvious, required releases are determined for each operational requirement and the largest required release up to outlet or channel capacities is used. The downstream accomplishments for the selected release for that period are then computed.

Although the detailed sequential routing procedure appears quite complex, the technique is based on the principle of conservation of mass. The fundamental relationship used in sequential routing is:

$$I - O = \Delta S \tag{6-3}$$

where:

$I$ = total inflow to the reservoir during a specified time period, in units of volume ($m^3$),

$O$ = total outflow from the reservoir during the same specified time period, in units of volume ($m^3$), and

$\Delta S$ = change in storage during the specified time period, in units of volume ($m^3$).

The inflow and outflow terms in the above equations must include all types of inflow and outflow. Among the inflows to be included are natural streamflow, releases from upstream reservoirs, local inflow to the reservoir, precipitation falling on the reservoir surface (unless included in the computation of net evaporation), and diversions into the reservoir from other streams or reservoirs. Outflows which must be included are releases for all purposes, evaporation losses, leakage, and diversions out of the reservoir.

Equation 6-3 can be transformed into a more usable form for practical applications as follows:

$$\Delta S_t = S_t - S_{t-1} \tag{6-4}$$

where:

$\Delta S_t$ = change in storage ($m^3$) during the time period between t-1 and t,

$S_t$ = storage in volume ($m^3$) at the end of time t, and

$S_{t-1}$ = storage in volume ($m^3$) at the end of time t-1.

By substituting this relationship, equation 6-3 becomes:

$$I_t - O_t = S_t - S_{t-1} \tag{6-5}$$

Rearranging terms produces the useful form:

$$S_t = S_{t-1} + I_t - O_t \tag{6-6}$$

**Table 6.02**
**Clinton reservoir, example of multiple purpose routing**

| ROUTING PERIOD DATA | | | RESERVOIR REGULATION | | | | | |
|---|---|---|---|---|---|---|---|---|
| Col. 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 |
| | N DAYS | | QI | | | | | |
| Date (Mo/Yr) | (No. of days) INPUT | flow conv. factor ($x10^6$) | Res. inflow (cumecs) INPUT | Res. Release (cumecs) | stor. 4 - 5 -17-18 (cumecs) | stor. (6)(3) ($m^3x10^6$) | E.O.P. stor. (P 8 + 7) ($m^3x10^6$) | Max. Consv. Stor. Allow ($m^3x10^6$) INPUT |
| 9/50 | | | | | | | 222.031 | |
| 10/50 | 31 | 2.6784 | 3.0 | 11.5 | - 9.2 | -24.670 | 197.361 | 197.361 |
| 11/50 | 30 | 2.5920 | 0.4 | 1.4 | - 1.7 | - 4.406 | 192.955 | 197.361 |
| 12/50 | 31 | 2.6784 | 0.3 | 0.7 | - 1.1 | - 2.946 | 190.009 | 197.361 |
| 1/51 | 31 | 2.6784 | 0.4 | 0.7 | - 1.0 | - 2.678 | 187.331 | 209.700 |
| 2/51 | 28 | 2.4192 | 0.7 | 0.4 | - 0.4 | - .968 | 186.363 | 221.580 |
| 3/51 | 31 | 2.6784 | 1.3 | 0.4 | + 0.2 | + .536 | 186.899 | 221.580 |
| 4/51 | 30 | 2.5920 | 2.4 | 0.4 | + 1.2 | + 3.110 | 190.009 | 221.580 |
| 5/51 | 31 | 2.6784 | 11.5 | 2.2 | + 8.4 | +22.499 | 212.508 | 221.580 |
| 6/51 | 30 | 2.5920 | 45.6 | 40.8 | + 3.5 | + 9.072 | 221.580 | 221.580 |
| 7/51 | 31 | 2.6784 | 89.0 | 56.6 | + 31.1 | +83.298 | 304.878 | 221.580 |
| 8/51 | 31 | 2.6784 | 1.7 | 31.5 | - 31.1 | -83.298 | 221.580 | 221.580 |
| 9/51 | 30 | 2.5920 | 17.3 | 16.1 | 0 | 0 | 221.580 | 221.580 |

INPUT refers to data determined prior to routing

Installed Capacity = 500 KW

Overload = 1.15

Efficiency = 0.86

Channel Loss = 0.05

Tailwater Elevation = 256.0 m

Outlet Capacity = 56.6 cumecs

**Table 6.02**
**Clinton reservoir, example of multiple purpose routing**
**(continued)**

| RESERVOIR QUALITY | | APPROXIMATIONS | | | EVAPORATION | | |
|---|---|---|---|---|---|---|---|
| 10 | 11 | 12 | 13 | 14 | 15 | 16 | 17 |
| (Qual. Inflow in PPM) | (Qual. Res. Incl. Release in PPM) | Est. Ave. Stor. ($m^3 x 10^6$) | Elev. m | Area (Hectare) | Net Evap. (mm) | Evap. (14)(15) ÷ .10 ($m^3 x 10^6$) | Evap. (16)÷(3) (cumecs) |
| INPUT | | | | | INPUT | | |
| | 100 | | | | | | |
| 129 | 102 | 209.696 | 268.53 | 3241 | 50 | 1.621 | 0.6 |
| 197 | 103 | 195.157 | 268.07 | 3129 | 50 | 1.564 | 0.6 |
| 199 | 104 | 191.148 | 267.92 | 3092 | 50 | 1.546 | 0.6 |
| 198 | 105 | 188.669 | 267.83 | 3070 | 50 | 1.535 | 0.6 |
| 186 | 106 | 186.725 | 267.62 | 3063 | 50 | 1.533 | 0.7 |
| 171 | 108 | 186.388 | 267.62 | 3063 | 50 | 1.532 | 0.6 |
| 142 | 110 | 187.175 | 267.89 | 3085 | 50 | 1.535 | 0.6 |
| 96 | 109 | 198.943 | 267.92 | 3114 | 50 | 1.570 | 0.6 |
| 28 | 81 | 214.728 | 268.87 | 3322 | 75 | 2.492 | 1.0 |
| 24 | 52 | 260.913 | 270.06 | 3630 | 75 | 2.723 | 1.0 |
| 160 | 54 | 260.913 | 270.06 | 3630 | 75 | 2.723 | 1.0 |
| 73 | 58 | 219.264 | 268.87 | 3322 | 75 | 2.504 | 1.0 |

Maximum Downstream Channel Capacity

Oct. - April = 2550 cumecs

May - Sept. = 2410 cumecs

**Table 6.02**
**Clinton reservoir, example of multiple purpose routing**
**(continued)**

| WATER REQUIRED AT DAM | | POWER REQUIREMENT | | | QUALITY REQUIRED DOWNSTREAM | | | |
|---|---|---|---|---|---|---|---|---|
| 18 | 19 | 20 | 21 | 22 | 23 QL | 24 | 25 EFLNT | 26 |
| Req'd Pipe Release (cumecs) | Req'd River Release (cumecs) | Req'd Power ($\times 10^3$ Kw-Hr) | Head (13 - TLWEL) (m) | Req'd Release for Power (cumecs) | Local flow (cumecs) | Qual. of local flow (ppm) | Effluent at D.S. pt. (Kgm/day) | Max. Allow D.S. Qual. (PPM) |
| INPUT | INPUT | INPUT | | | INPUT | INPUT | INPUT | INPUT |
| 0.1 | 0.7 | 26 | 12.53 | 0.3 | 1.4 | 300 | 1815 | 250 |
| 0.1 | 0.7 | 25 | 12.07 | 0.3 | 1.3 | 356 | 1815 | 250 |
| 0.1 | 0.7 | 45 | 11.92 | 0.6 | 8.8 | 200 | 1815 | 250 |
| 0.1 | 0.7 | 45 | 11.83 | 0.6 | 7.5 | 200 | 1815 | 250 |
| 0.1 | 0.3 | 24 | 11.73 | 0.4 | 8.5 | 200 | 1815 | 250 |
| 0.1 | 0.3 | 26 | 11.62 | 0.4 | 16.0 | 200 | 1815 | 250 |
| 0.2 | 0.4 | 25 | 11.81 | 0.3 | 21.4 | 200 | 3630 | 250 |
| 0.3 | 0.4 | 45 | 12.26 | 0.6 | 2.8 | 200 | 3630 | 250 |
| 0.3 | 0.4 | 43 | 12.87 | 0.5 | 22.3 | 200 | 3630 | 200 |
| 0.3 | 0.4 | 89 | 14.06 | 1.0 | 3.8 | 200 | 3630 | 200 |
| 0.3 | 0.4 | 89 | 14.06 | 1.0 | 6.6 | 200 | 3630 | 200 |
| 0.2 | 0.3 | 43 | 12.99 | 0.5 | 12.6 | 200 | 3630 | 200 |

**Table 6.02**
**Clinton reservoir, example of multiple purpose routing**
**(continued)**

| | FLOW REQUIRED DOWNSTREAM | | | | ACCOMPLISHMENTS | | | | |
|---|---|---|---|---|---|---|---|---|---|
| 27 | 28 | 29 | 30 | 31 | 32 | 33 | 34 | 35 | 36 |
| Req'd Release for D.S. Qual. (cumecs) | Req'd Wtr. RTS. (cumecs) | Req'd Add. D.S. flow (cumecs) | ALOS Const. Chan. loss (cumecs) | Req'd Release for D.S. flow (cumecs) | Act. D.S. flow (cumecs) | Act. D.S. Qual. (PPM) | Act. Gen. PWR. ($\times10^3$ Kw-Hr) | Remarks | Case |
| | INPUT | INPUT | INPUT | | | | | | |
| 0.9 | 0.3 | 1.1 | 0.3 | 0.3 | 12.0 | 127 | 428* | | 11 |
| 1.4 | 0.3 | 1.4 | 0.3 | 0.7 | 2.3 | 255* | 101 | | 7 |
| 0 | 0.3 | 1.4 | 0.3 | 0 | 8.9 | 198 | 51 | | 5 |
| 0 | 0.3 | 1.4 | 0.3 | 0 | 7.9 | 198 | 51 | | 5 |
| 0 | 0.3 | 2.0 | 0.3 | 0 | 8.6 | 201 | 26 | | 6 |
| 0 | 0.3 | 2.3 | 0.3 | 0 | 16.1 | 201 | 29 | | 6 |
| 0 | 1.4 | 2.5 | 0.4 | 0 | 21.4 | 202 | 28 | | 5 |
| 0 | 1.4 | 3.1 | 0.4 | 2.2 | 4.5 | 175 | 166 | | 4 |
| 0 | 1.4 | 6.2 | 0.4 | 0 | 60.7 | 125 | 414* | | 11 |
| 0 | 1.4 | 4.5 | 0.4 | 2.7 | 57.2 | 63 | 428* | | 1 |
| 0 | 1.4 | 2.3 | 0.4 | 0 | 36.1 | 82 | 428* | | 11 |
| 0 | 1.4 | 1.7 | 0.4 | 0 | 27.5 | 125 | 414* | | 11 |

*Maximum Allowable

TABLE 6.02 - CLINTON RESERVOIR, EXAMPLE OF MULTIPLE PURPOSE ROUTING (Con't.)

Col. 1 - Date of routing period (month number/year).

Col. 2 - Number of days in routing period. If routing is performed with flows and demands in volume units, this column is not needed.

Col. 3 - Factor used to convert average flow in cumecs to cubic meters ($m^3$) for the period (Col. 2 times 86400).

** Col. 4 - Average inflow to reservoir in cubic meters per second. Volume units can also be used.

Col. 5 - Average reservoir release to channel in cumecs. The controlling requirement is the larger of columns 19, 22, 27, or 31 unless reservoir evacuation for flood control is greater.

Col. 6 - Average change in storage for period ($\pm$) in cumecs (Col. 4 minus Col. 5, 17, and 18).

Col. 7 - Change in storage in cubic meters (Col. 6 times Col. 3).

Col. 8 - End of period storage in cubic meters (Col. 8* + Col. 7).

** Col. 9 - Maximum allowable conservation storage for the period in cubic meters. This column can be omitted if storage is constant for all periods during the year.

** Col. 10 - Quality of reservoir inflow (normally in parts per million).

Col. 11 - The quality of the reservoir contents in ppm is normally determined by assuming complete mixing as follows:

$$\frac{(\text{Col. } 8^*)(\text{Col. } 11^*) + (\text{Col. } 4)(\text{Col. } 10)(\text{Col. } 3)}{(\text{Col. } 8^*) + (\text{Col. } 4)(\text{Col. } 3) - (\text{Col. } 16)}$$

Col. 12 - Estimated average storage for period (used to compute evaporation and power head). Sometimes the end of previous period storage is used, although this introduces a cumulative error.

Col. 13 - Pool elevation in meters corresponding to estimated storage of Col. 12.

Col. 14 - Reservoir area in hectares corresponding to estimated storage.

** Col. 15 - Net reservoir evaporation in mm.

* Denotes use of previous period value

** Furnished from basic data

Col. 16 - Total reservoir evaporation in ($m^3 x 10^6$), determined as follows:

$$\frac{(\text{Col. }14)(\text{Col. }15)}{0.1}$$

Col. 17 - Total reservoir evaporation in cumecs (Col. 16÷Col. 3).

** Col. 18 - Reservoir release to pipeline in **cumecs to draw water directly out of reservoir and bypass the power penstock.**

** Col. 19 - Required release to channel below the dam in cumecs which **can pass through penstock.**

** Col. 20 - Power requirement in 1,000 kw-hr.

Col. 21 - Power head in feet [(Col. 13) - (tailwater elevation) - (head loss)]. In this example, head loss was not considered.

Col. 22 - Required reservoir release (Q) in cumecs, to satisfy the power requirement, determined as follows:

$$Q = \frac{(\text{Col. }20)}{9.817(\text{Col. }21)(\text{Hrs})(\text{Effcy})}$$

where: Hrs = Number of hours in the period
Effcy = Power plant efficiency

** Col. 23 - Tributary flow below the dam and above the downstream control point in cumecs.

** Col. 24 - Quality of tributary flow in units corresponding to Col. 10 units.

** Col. 25 - Effluent at downstream control point in Kgm per day.

** Col. 26 - Maximum allowable quality concentration at the downstream control point in units corresponding to Col. 10 units.

Col. 27 - Required reservoir release to maintain quality standards at the downstream control point, determined as follows:

$$\frac{(\text{Col. }30)(\text{Col. }26 - \text{Col. }11)(1 - \text{Channel Loss}) + (\text{Col. }23)(\text{Col. }24 - \text{Col. }26) + (.0116)(\text{Col. }25)}{(\text{Col. }26 - \text{Col. }11)(1 - \text{Channel Loss})}$$

** Col. 28 - Required flow at downstream control point in cumecs to satisfy water rights.

** Col. 29 - Flow, in cumecs, required at downstream control points in addition to water rights.

** Col. 30 - Constant channel loss in cumecs below reservoir.

Col. 31 - Required reservoir release in cumecs to satisfy total downstream flow requirements, determined as follows:

$$\frac{(1 - \text{Channel Loss})(\text{Col. } 30) + \text{Col. } 28 + \text{Col. } 29 - \text{Col. } 23}{(1 - \text{Channel Loss})}$$

Col. 32 - Resulting flow in cumecs at downstream control point, determined as follows:

$$\text{Col. } 23 + (\text{Col. } 5 - \text{Col. } 30)(1 - \text{Channel Loss})$$

Col. 33 - Actual resulting quality in units corresponding to Col. 10 at downstream control point, determined as follows:

$$\frac{(\text{Col. } 11)[(\text{Col. } 5) - (\text{Col. } 30)]\,(1 - \text{Channel Loss}) + (\text{Col. } 23)(\text{Col. } 24) + (0.0116)\,(\text{Col. } 25)}{(\text{Col. } 32)}$$

Col. 34 - Actual resulting power generated in thousands of kw-hrs, determined as follows:

(Col. 5)(Col. 21)(Efficiency)(.2311)(Col. 2)

up to the maximum power. Where maximum power in kw-hrs. is:

(Installed Capacity)(Overload)(24)(Col. 2)

Col. 35 - Any special remarks.

Col. 36 - Controlling case defined as follows:

1. Release restricted by reservoir outlet capacity.
2. Release restricted by damsite channel capacity.
3. Release restricted by downstream channel capacity.
4. Release to satisfy downstream water requirement.
5. Release to satisfy water requirements at dam.
6. Release to satisfy power requirements.
7. Release required to satisfy downstream water quality requirements.
8. Release required to prevent over-filling flood control storage.
9. Release controlled by declared shortage.
10. Release restricted by bottom conservation pool.
11. Release to empty flood control storage.

A sequential routing study essentially consists of a repetitive solution of equation 6-6 together with an examination of pertinent physical constraints, storage boundaries, and service priorities to determine outflows for each time period.

The solution of equation 6-6 is straight forward if the water demands can be completely prespecified as unique quantities on a period-by-period basis. A form such as table 6.02 can be used and the inflow and demands entered in appropriate columns for each period of the study. A starting value of reservoir storage should then be assumed (usually the storage at the top of the conservation pool). Next, the inflow for the first period is added, and the various demands for the period are examined to determine the total outflow needed to supply all pertinent demands. The required outflow must be checked to insure that none of the physical constraints are violated. The outflow is then subtracted from the previously obtained sum of inflow and initial storage to determine the storage at the end of the first period. This computational sequence is repeated for each period in turn, using the end-of-period storage of the previous period as the initial storage.

A common difficulty with the preceding technique is to maintain consistent units throughout the study. Streamflow data and demand data are usually in rate units (cumecs or c.f.s.) rather than volume units. Consequently, either these data must be converted to volume units by multiplying the rates by a factor which accounts for the time interval in each period, or to units which will permit direct addition and subtraction of rate units. This latter procedure is more frequently used with English units because of the inconsistency of volume units such as acre-feet with rate units such as cubic feet per second. Under any circumstances, all conversions from rate to volume units or vice versa, should be accomplished prior to the beginning of the study so that all data entered onto the form are in the same units.

Unfortunately, not all purposes can be considered through the use of period-by-period prespecified water demands. For example, power demands are usually specified in terms of energy requirements in kilowatt-hours per period. The conversion of this demand to a water quantity is dependent upon the head available during the period and the length of the period. Similarly, the demands for water quality augmentation may be specified in terms of desired quality of outflow or in terms of the desired quality of the combined outflow and intervening flow between the reservoir and some downstream quality control location. In the latter case, the quality of the intervening flow must be known or estimated in addition to the quality of the water stored in the reservoir.

An additional complication arises in the case of detailed sequential studies involving hydroelectric power generation. Since power generation is dependent on head, which may vary significantly during a single routing period, power computations should usually be based on average head during the routing period rather than on the head at the beginning of the period. The average head during a period is a function of the average storage which, in turn, is the average of the beginning and ending value for the period.

The ending storage, however, is dependent upon total outflow during the period which is determined by the head. The computation for each period, therefore, requires successive approximations.

Water required for power generation is computed by the following equation:

$$Q = \frac{E}{9.817hte} \tag{6-7}$$

where:

$Q$ = required release in cumecs

$E$ = energy required in kilowatt-hours

$h$ = average head in meters

$t$ = number of hours in the period

$e$ = power plant efficiency expressed as a ratio

In the solution of equation 6-7 both Q and h are unknown. The normal procedure is to assume a value of h, usually based on the ending storage for the previous period, and compute a value for Q. The ending storage for the current period is calculated, and a new value of h is then determined from the average of the ending storage for the previous period and the computed ending storage for the current period. The value of Q is then re-calculated and the process is repeated until the values of h on two successive trials do not differ significantly. Although the procedure is tedious, good initial estimates of h based on experience can significantly reduce the number of trials needed.

A similar computation is usually necessary when evaporation is an important factor in the sequential analysis. Initially, evaporation rates in millimeters (or inches) per period for the area must be converted to volumes for the analysis. The volume of evaporated water depends upon the surface area of the reservoir which is a function of storage. Again, the average storage during the period must be computed in order to determine the average surface area. The volume of evaporation loss is determined by:

$$E_{vt} = \frac{E_{rt}A_t}{0.1} \tag{6-8}$$

where:

$E_{vt}$ = evaporation volume ($m^3$) in period t

$E_{rt}$ = net evaporation rate (mm/time unit) in period t

$A_t$ = average reservoir area (hectares) in period t

The head and surface area values used in either power or evaporation computations, should not be based on the ending storage of the previous period. By consistent use of the ending storage values, a cumulative error is produced by the power and evaporation calculations. For example, when storage is decreasing, the use of such values causes a consistent overestimate of the head and a resultant underestimate of the water required to meet the power needs. Because of this underestimate of water requirements, the storage does not decrease as rapidly as it actually would, and the effect is thereby compounded. The effect of this error can be serious, especially for long dry periods (two years or more) at large reservoirs. Similarly, the use of end of period values in evaporation computations causes a cumulative effect in the opposite direction. The assumption should not be made, however, that since evaporation and power computations produce opposite effects, these effects cancel each other.

As previously mentioned, the purpose of using a form such as table 6.02 is not only to provide a logical format for manual computations, but also to allow the user to ascertain the controlling factors with as much ease as possible. In most cases, the controlling factor can be located from inspection. During period 10/50 in table 6.02, the inflow is sufficiently high to more than satisfy all other requirements, which makes it obvious that evacuation of flood control storage will control. It is then unnecessary to compute releases required for power, quality and downstream flow requirements for that period. However, these were determined in the example problem only to show that the minimum releases required to satisfy each particular requirement were met. The underlined value for each period is the required controlling release for that period.

Computer programs which perform sequential routing studies must incorporate the logic necessary to automatically perform the successive approximations, make the necessary unit conversions, and examine all demands and constraints to select the appropriate controlling factors that govern reservoir operation in each time period. Projects with unique characteristics often dictate that a special computer program be written. In any case, computer programs for such studies should be carefully checked, perhaps with a few manual computations, to insure that the logic in the program is correct.

## Section 6.04 - Evaluation of Storage Requirements for Simultaneous Supplies of Different Dependabilities

When storage requirements for two or more project purposes are determined for different probabilities of shortage (such as 2% and 10%), the total storage requirement for two project purposes is required along with the reservoir level below which the lower-priority purpose will not be served. Simplified procedures are illustrated in the following paragraphs. These procedures are sufficiently accurate for preliminary studies, but final studies require detailed reservoir routings according to the plan of operation throughout the length of the available flow record. Determination of total storage requirement and a cut-off storage level is most accurately made by successive approximations based on detailed sequential analysis, with first approximations based on a simplified procedure.

One simplified method for solving the problem of two simultaneous supplies at different shortage frequencies is based on the following equations:

$$TRY = YRA + YRB \tag{6-9}$$

$$CY = CYB + (CYA - CYB)\,(YRA/TRY) \tag{6-10}$$

$$CS = CYRAA - CYRAB \tag{6-11}$$

where:

TRY = Total required yield

CY = Storage required for simultaneous supply at different frequencies

CS = Cutoff storage below which water will not be supplied for less severe criteria

YRA = Yield required for the more severe shortage criteria

YRB = Yield required for the less severe shortage criteria

CYB = Storage required to supply total required yield (TRY) at the less severe shortage criteria

CYA= Storage required to supply total required yield (TRY) at the more severe shortage criteria

CYRAA = Storage required for yield YRA at more severe shortage criteria

CYRAB = Storage required for yield YRA at less severe shortage criteria

The use of equations 6-9, 6-10, and 6-11 is illustrated in the following example:

Problem: Find the storage required to yield 1.4 cumecs at a 1% chance of shortage and 0.4 cumecs at a 10% chance of shortage. Use storage-yield curves for 1% and 10% chance of shortage as shown on fig. 6.07.

Solution: More severe shortage criterion = 1% chance of shortage per year

Less severe shortage criterion = 10% chance of shortage per year

YRA = 1.4 cumecs

YRB = 0.4 cumecs

From equation 6-9:

TRY = 1.4 + 0.4 = 1.8 cumecs

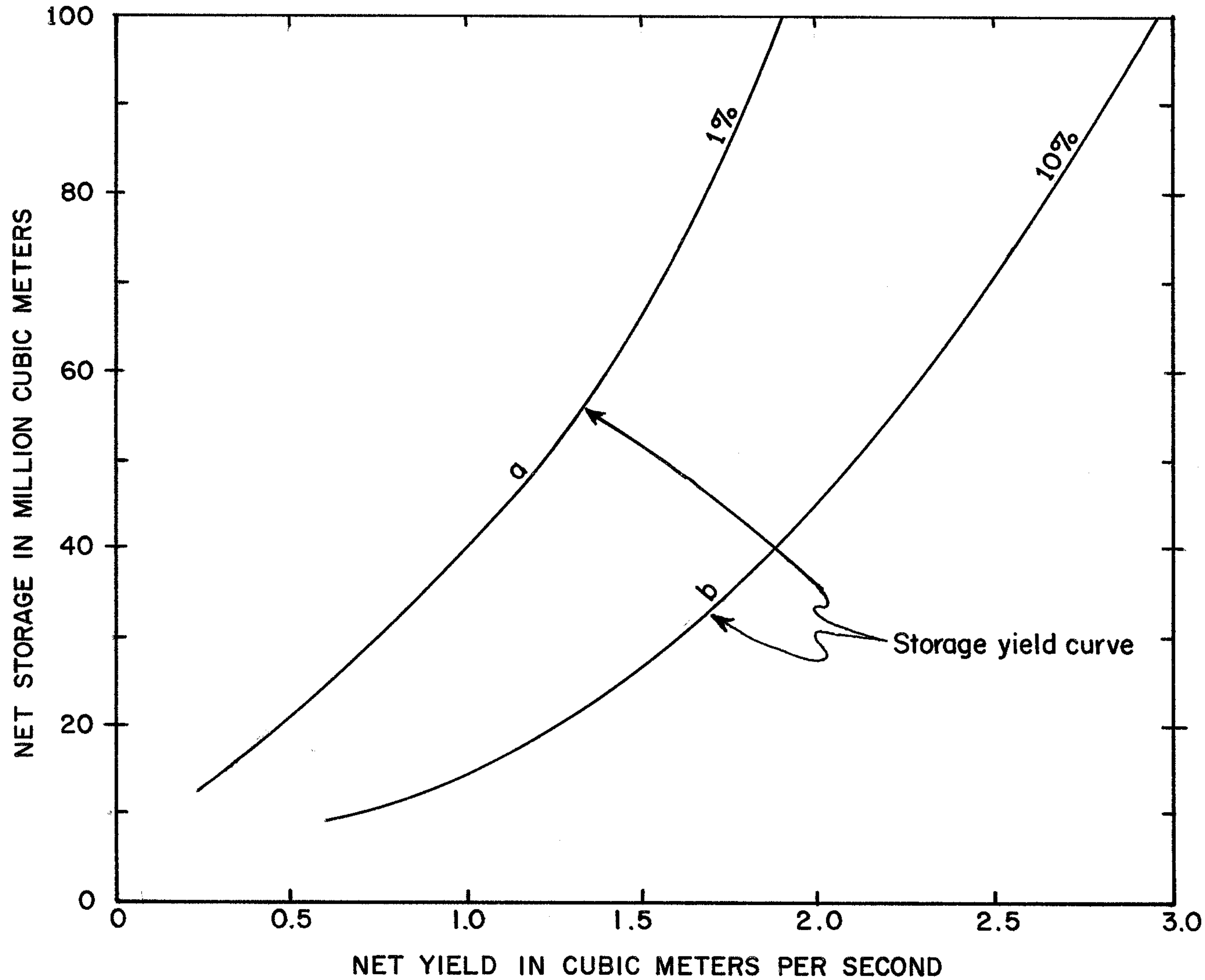


Fig. 6.07 Storage-yield curves for 1 and 10 percent chances of shortage per year

From fig. 6.07:

CYA = 90 million cubic meters (curve a @ 1.8 cumecs)

CYB = 36 million cubic meters (curve b @ 1.8 cumecs)

CYRAA = 60 million cubic meters (curve a @ 1.4 cumecs)

CYRAB = 24 million cubic meters (curve b @ 1.4 cumecs)

From equation 6-10:

$$CY = 36 + (90 - 36)\ (1.4/1.8)$$

$$= 78 \text{ million cubic meters}$$

From equation 6-11:

$$CS = 60 - 24 = 36 \text{ million cubic meters}$$

The simplified method indicates that storage of 78 million cubic meters will be required to supply both demands at the given shortage criteria and that releases for the lower-priority purpose (10% chance of shortage) should be stopped when the storage is reduced to 36 million cubic meters in order to maintain releases for the higher-priority purpose (1% chance of shortage).

## Section 6.05 - Computation Aids

It is necessary to exercise ingenuity in developing computation aids which will significantly decrease the time required to perform a manual routing study, particularly if the proposed study is lengthy or if a series of studies is to be made and electronic computer facilities cannot be employed.

An example of a useful computation aid is the KW/CUMEC or KW/CFS nomograph for use in power routing studies. This nomograph is prepared prior to beginning a power routing study and it incorporates all of the physical data required to determine a power release for a given amount of power generation. Table 6.03 shows the computations necessary to develop a KW/CUMEC curve and the method for constructing the nomograph. The resultant KW/CUMEC curve is illustrated in fig. 6.08.

Some frequently used conversion constants are given in Appendix II.

TABLE 6.03 COMPUTATIONS NECESSARY TO CONSTRUCT A KILOWATTS/CUBIC METER PER SECOND NOMOGRAPH

KILOWATTS/CUMEC COMPUTATION - RESERVOIR AAA

| Pool Elevation (M abv m.s.l.) | Storage (thousand $M^3$) | Net Head(1) (M) | Efficiency(2) (%) | KW/CUMEC(3) |
|---|---|---|---|---|
| 475.6 | 145.2 | 175.6 | 83.2 | 1434 |
| 472.1 | 136.0 | 172.1 | 84.0 | 1419 |
| 468.7 | 127.3 | 168.7 | 84.6 | 1401 |
| 465.2 | 119.0 | 165.2 | 85.1 | 1380 |
| 461.8 | 111.0 | 161.8 | 85.5 | 1358 |
| 458.3 | 103.5 | 158.3 | 85.9 | 1335 |
| 454.9 | 96.3 | 154.9 | 86.1 | 1309 |
| 451.4 | 89.5 | 151.4 | 86.3 | 1283 |
| 448.0 | 83.0 | 148.0 | 86.1 | 1251 |
| 444.6 | 76.9 | 144.6 | 85.9 | 1219 |

Based on constant avg. tailwater @ elev. 299.8 meters above m.s.l. with assumed constant penstock losses of 0.2 meters

(1) Net head = Pool elev. - penstock losses - avg. tailwater (Both penstock loss and avg. tailwater may be varied with pool elevation if relationship is known)

(2) Overall station efficiency (may be assumed constant at all pool elev)

(3) KW/CUMEC = (Head) x (eff) x (9.817)

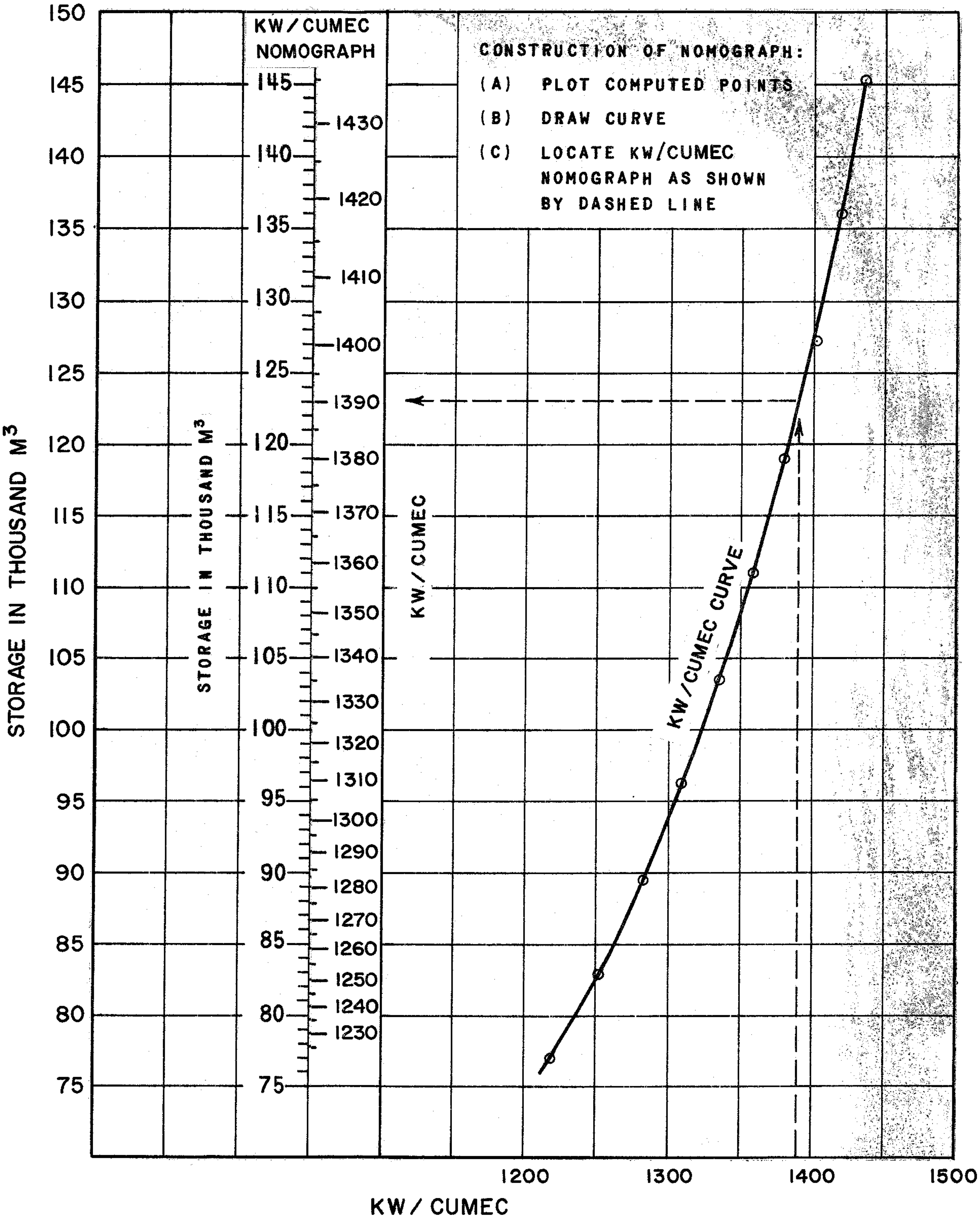


Fig. 6.08 Sample KW/CUMEC nomograph for power routing study

Chapter 7

# Development and Use of Rule Curves

# DEVELOPMENT AND USE OF RULE CURVES

## Section 7.01 - Introduction

Among the most useful devices in the operation of reservoir projects is the rule curve. A rule curve is a guideline for reservoir operation, and is generally based on a detailed sequential analysis of various critical combinations of hydrologic conditions and demands. Frequently, the development of a rule curve is delayed until after the project is completed. This is unfortunate because it is difficult to accurately assess the true accomplishments of a project in the planning stage without knowing the rules which govern the operation of the project.

Rule curves, sometimes called guide curves, have commonly been used in conjunction with the operation of projects with hydroelectric installations. Such curves developed for use in flood control operations have been called flood control diagrams. By whatever name, such curves are intended to provide guides for the operation of reservoir projects during periods of critical hydrologic conditions.

All authorized purposes in multiple-purpose development must be considered in the operation of projects. To best serve flood control, power production, and other uses, an operating procedure must be established which minimizes conflicts between the purposes. The objective of multiple purpose projects with power is to provide a balanced operation for the generation of firm and secondary energy through high-, median-, and low-flow periods while maintaining optimum storage capacity in the reservoirs for the regulation of floods. Thus, reservoir levels at all times should be only as high as are necessary to assure the availability of firm capacity and energy. This objective can be accomplished by proper use of power operating rule curves based on sound engineering criteria and routing studies.

The power operating rule curve is defined by the United States Inter-Agency Committee on Water Resources (reference 17) as "a curve, or family of curves, indicating how a reservoir is to be operated under specific conditions to obtain best or predetermined results." Actually, the rule curve need not be applicable to a single reservoir as is stated in the preceding definition. There may be instances where a single rule curve for a hydraulically integrated system of storage plants would better serve the needs of the system operation. Obviously, rule curves may assume many forms, depending upon the background and experience of the planning engineer and the operating constraints associated with the storage plants involved.

## Section 7.02 - Typical Single-Plant Rule Curve for Power Operation

A rule curve for power operation for an isolated storage plant is shown in fig. 7.01. The curve illustrates the reservoir elevation (and consequently the storage) required to assure generation of firm power at any time of the year. The general shape of the rule curve is logical if it is realized that the power storage during the middle of the calendar year must be a maximum in certain localities in anticipation of high summer power demands coincident with low inflows, and that droughts usually begin during the late spring and early summer. A low pool elevation is also acceptable in the fall and winter season if there is an accompanying lower power demand and prospects of high winter and spring inflows. Firm energy is usually defined as that generation which would exactly draw the reservoir level to the bottom of the power pool during the most severe drought of record. Therefore, if all potential droughts begin with the reservoir level on or above the rule curve elevation, if generation is limited to firm energy production, and if the generation pattern is in general agreement with the assumed monthly distribution used in the studies, the pool should not fall below rated pool unless a drought more severe than any of record is experienced. This is because the rule curve is an approximate enveloping curve of all severe low-flow periods of record. On the other hand, if high flows occur and the pool level rises above the rule curve, secondary energy should be generated to bring the pool back to the rule curve so that optimum storage capacity is provided for the regulation of potential floods.

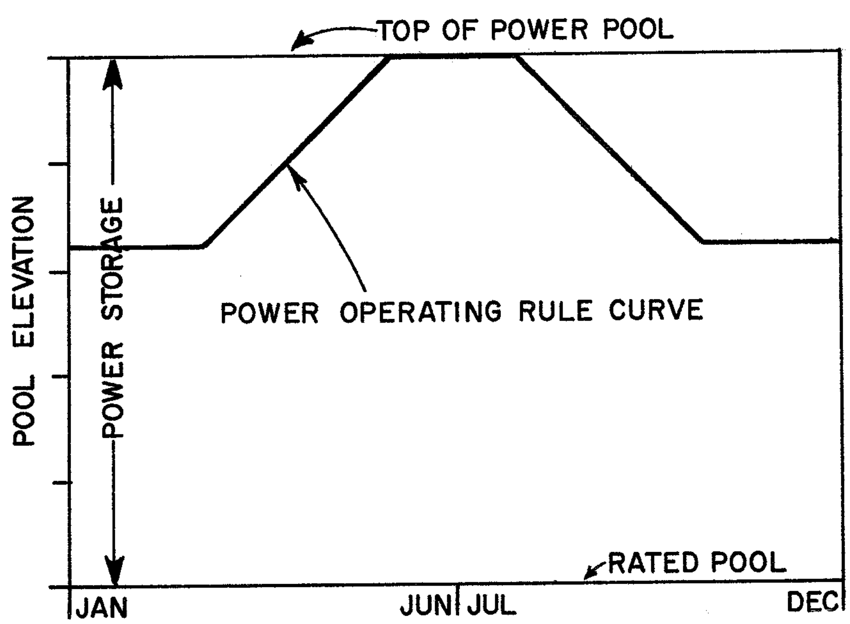


Fig. 7.01 Example of a rule curve for power operation

The power operating rule curve can be described as a guide for scheduling firm and secondary energy and providing optimum storage capacity for the regulation of floods. Its proper use minimizes dump energy generation and spillage because many small floods can be regulated within the power pool with releases through the turbines.

## Section 7.03 - Construction of a Typical Rule Curve for Single-Plant Power Operation

Data are readily available from sequential routings for the plotting of reservoir hydrographs covering the period of maximum drawdown. Fig.7.02 illustrates a pool elevation hydrograph of a typical project during a drought from 1962 to 1965. The period of maximum drawdown in this example extends from May 1962 to February 1965.

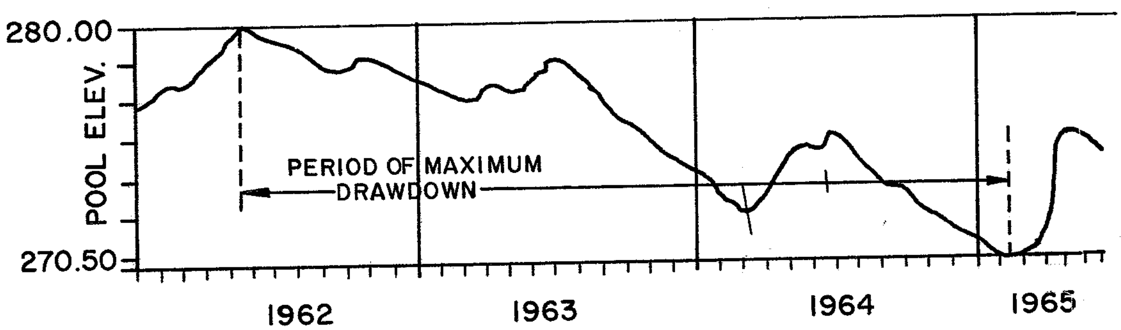


Fig. 7.02 Reservoir pool elevation hydrographs during a period of maximum drawdown

In addition to the most critical period, several other periods of lesser severity should be analyzed to determine whether combinations of demand and hydrologic conditions other than those experienced in the critical period might affect the location of the rule curve. It is reasonable to assume that the power storage could have been depleted at the ends of these lesser periods in determining whether they control the construction of the curve, because the power operating rule curve is a guide to avoid a drawdown below the lower limit of the power storage. Consequently, the reservoir hydrographs shown in fig. 7.03 for three other significant low-flow periods, are the results of reverse routings utilizing the scheduled firm power developed in the period of maximum drawdown routing beginning with the reservoirs at the bottom of the power pool at the end of their respective periods.

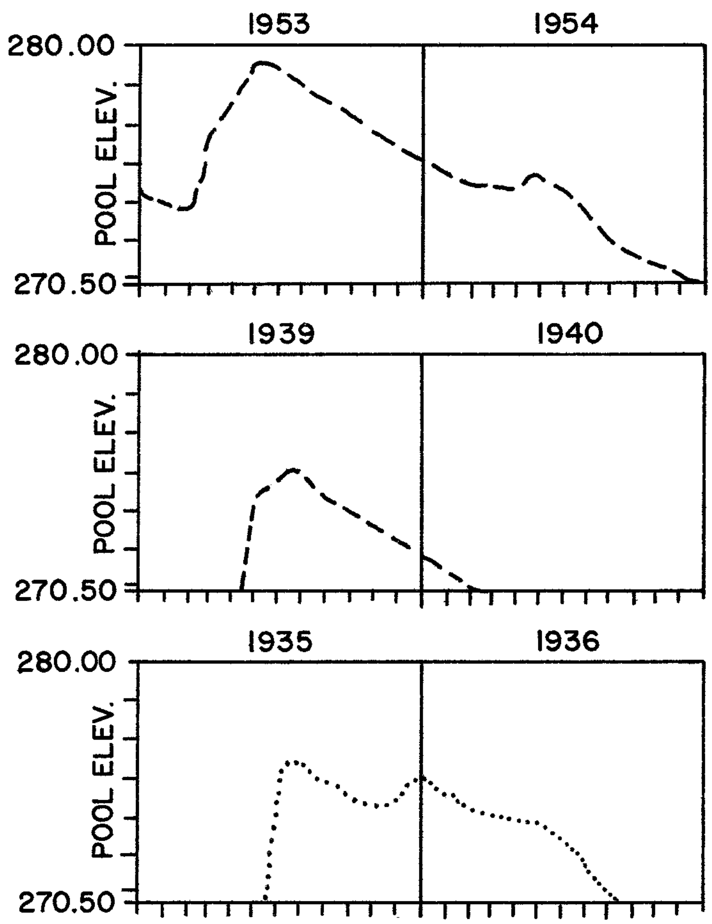


Fig. 7.03

Examples of three low-flow periods used for the development of a rule curve

The reservoir hydrographs of all significant low-flow periods are plotted on a single-year time base, as shown in fig. 7.04. Since an envelope of these hydrographs represents the pool levels required to provide adequate storage at the beginning of the four significant low-flow periods of record, a curve which envelops all hydrograph plots represents the pool elevations required at all times of the year to assure firm energy generation through all droughts of the period of record. The power operating rule curve (the enveloping curve) is also shown on fig. 7.04. In assuring firm energy, the rule curve also assures that the project's installed capacity is protected because the firm energy selected will result in a drawdown only to the bottom of the power pool if no drought more severe than the critical period drought occurs during the project life.

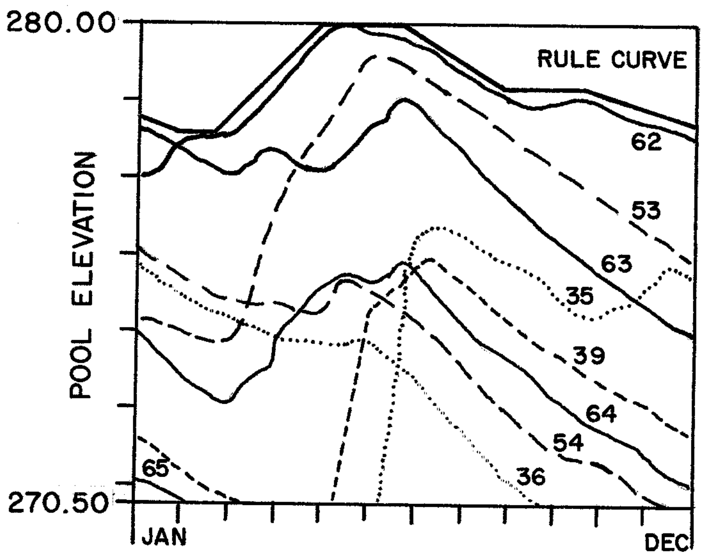


Fig. 7.04 Historical low-flow critical periods with the enveloping rule curve

## Section 7.04 - Significance of a Rule Curve for Power Operation

In order to understand the concept of the power operating rule curve, it is necessary to accept the idea that a drought is in progress whenever the water stored in the power pool is less than that indicated by the curve. The severity of a drought will not be known until the water in storage reaches a minimum level and is subsequently replenished to the amount shown by the rule curve. It is for this reason that only scheduled firm power should be generated when the reservoir level is on or below the curve.

## Section 7.05 - Use of Rule Curves for Power Operation

In order for power operating rule curves to be meaningful in the operation of a system of hydroelectric plants, procedures for their use in estimating power generation and monitoring power production need to be established under a set of definite operating rules. Such procedures, if understood and followed, can give vital assistance to a balanced operation, particularly between power and flood control. They can be the backbone for estimating available power and for the necessary monitoring of power generation.

The problem in estimating the power potential of a system, is to determine if system reservoir conditions require (a) firm energy production, (b) firm energy plus some secondary, or (c) round-the-clock capacity generation. Round-the-clock capacity generation is usually associated with the flood control operation and permissible releases to the downstream channel and is not considered part of the rule-curve solution.

It should not be assumed that actual generation will follow exactly the generation scheduled. The load demands on a hydroelectric system cannot be precisely determined in advance. For this reason, it is the responsibility of the operating agency to monitor actual generation at its hydroelectric stations to make certain that it follows generation potentials and yet is not detrimental to subsequent power generation or to other project purposes.

As mentioned previously, the power operating rule curve indicates the power storage or energy in storage required at all times during the year to assure firm energy generation through all recorded low-flow periods. Consequently, it should be assumed that when the subsystem power storage or energy in storage is reduced below the level indicated by the rule curve, a drought has begun and its extent and severity cannot be determined until the water in storage reaches a low point, is replenished, and again reaches that shown by the rule curve. It is in such a situation of drought that monitoring should determine whether the cumulative total of kilowatt-hours generated is in agreement with firm energy as established in the sequential routing studies.

It is also important to monitor the actual generation during those periods when high inflows tend to place more water in storage than is required to sustain firm power generation. Holding subsystem storage in excess of that required for firm-power generation will increase the frequency of spills and downstream damage. It is necessary, therefore, to keep a close watch on the actual generation and offer suggested changes, when required, if an optimum balance between the power and flood control operation is desired.

Chapter 8

# Analysis of Study Results

## ANALYSIS OF STUDY RESULTS

One of the most frequently overlooked aspects of a storage-yield study is the preparation of a summary which presents the assumptions embodied within the study, the sources of data, the results of the study and an explanation of the significance of results. There are at least two important reasons for providing such a summary. First, the summary stands as a permanent record of the basis for the analysis, which is often needed in future analyses. The second reason, usually of more immediate concern, is to provide a basis for rapid evaluation of a study so that its results may be fully utilized in subsequent studies. In many cases, a successive-approximations approach is necessary to determine storage requirements, and judicious use of the results of each trial can greatly reduce the total number of trials required.

There are many cases where a simple average of the project yield during either the entire study period or the critical period will suffice as a summary of the project accomplishments. This is particularly true in the case of a single-purpose project. Table 6.02 illustrated provisions for obtaining data needed to summarize project accomplishments. If the project provides water for more than one purpose, it may be necessary to obtain more information than needed to summarize a single-purpose study. Additional information such as the length of critical period for each purpose, the storage capacity of the reservoir at the time of critical demand for each purpose, and the concurrent demands for other purposes are the types of data that are useful to evaluate results of a multiple-purpose project analysis.

Evaluation of study results requires knowledge of the critical aspects of the study and the conditions which produced the critical situation. Minimum storage, time of minimum storage, length of critical period, and duration of low-flow conditions at a downstream control point are examples of critical situations which should be examined. Furthermore, conditions prior to the critical occurrence should be examined with regard to possible changes in operating rules or storage designations which might alleviate the critical condition.

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Appendix 1

# Partial Duration Independent Low Flow Events

# PARTIAL DURATION - INDEPENDENT LOW FLOW EVENTS

HYDROLOGIC ENGINEERING CENTER
COMPUTER PROGRAM **723-G1-L2290**

JULY 1966

U. S. ARMY ENGINEER DISTRICT
650 CAPITOL MALL
SACRAMENTO, CALIFORNIA

APP I TP

PARTIAL DURATION - INDEPENDENT LOW FLOW EVENTS

HYDROLOGIC ENGINEERING CENTER
COMPUTER PROGRAM 723-G1-L2290

CONTENTS



EXHIBITS

PARTIAL DURATION - INDEPENDENT LOW FLOW EVENTS

HYDROLOGIC ENGINEERING CENTER

## 1. ORIGIN OF PROGRAM

This program was prepared in the Hydrologic Engineering Center, Corps of Engineers, 650 Capitol Mall, Sacramento, California, by Warren L. Sharp. Up-to-date information and copies of source statement cards for various types of computers can be obtained from the Center upon request by Government and cooperating organizations.

## 2. PURPOSE OF PROGRAM

This program written in Fortran II and IV will compute the data necessary to plot a partial duration curve depicting independent low flow events for a given period of monthly stream flows. Up to twenty durations may be specified in one computer run with partial duration plotting data being determined for each. Storage-yield relations can subsequently be determined from the output of this program.

## 3. DESCRIPTION OF EQUIPMENT

This program was prepared for use in the IBM 1620 (Fortran II) and IBM 7090 (Fortran IV) classes. Due to memory limitations of the IBM 1620 (40,000 digit, variable ward length, card input and output) a much longer record of flows can be analyzed using the larger computer. Limitations of the Fortran II program are explained in the following paragraphs.

## 4. METHODS

a. The method used in determining a partial duration series is similar to that described in the "Handbook of Applied Hydrology", 1964 edition, by V. T. Chow, pages 18-11 to 18-15 and in ASCE Sanitary Engineering Division publication 3283, September 1962, "Reservoir Mass Analysis by A Low-Flow Series" by John B. Stall.

b. Flow volumes are accumulated each month in accordance with a given duration for the entire period of record. That is, each cumulative value represents the flow for the current month plus the summation of previous flows for a number of months equal to the duration minus one. In the Fortran IV program this volume is converted to an average rate of flow in cfs regardless of the input units. This array is then successively scanned to locate low-flow events in an ascending manner. These events are selected without regard to the calendar year and in such a manner as to assure their chronological independence. To avoid overlapping of data, say for a 12 month duration, the prior and subsequent 11 cumulative volumes/average rates are excluded from further consideration. The initial 11 volumes at the beginning of the record (for a 12 month duration) are also excluded since they do not represent full 12 month volumes. The latter exclusion is used in determining the effective number of years of record which is subsequently used in computing the exceedence frequency (plotting positions). Equation (1) in "Statistical Methods" by Leo R. Beard is used to compute the plotting positions. The number of events to be considered is limited to the smaller of the following two-conditions: (1) a recurrence interval no smaller than 2 years or, (2) the total number of monthly periods divided by the months of duration. Neither condition will be reached when the ratio of total months of record to months of duration is small due to the non-overlapping stipulation.

5. INPUT

Input is summarized in exhibit 5 and illustrated in exhibit 6. All data are entered consecutively on each card, using 8 columns (digits, including decimal point if used) per variable and 10 variables per cards, unless fewer are called for. Recorded or simulated monthly stream flows in the form of acre-feet, cubic feet per second or inches of runoff may be supplied (up to 1000 years of record) the Fortran IV program. Flows in cubic feet per-second are not acceptable in the Fortran II program and the record is limited to 75 years.

6. OUTPUT

a. Examples of output are included among the exhibits using various durations for both the Fortran II and Fortran IV programs.

b. As previously stated, input flow data is always converted to an average rate in cfs in the Fortran IV program using 30.4375 days/month. For each event this average rate is converted to a volume in acre-feet (also using 30.4375 days/month) and depth in inches for the given drainage area. The volume, depth and rate are printed for each event.

c. No conversion of the input flow data is made in the Fortran II program. Units of the output volume will be the same as input.

7. OPERATING INSTRUCTIONS

Standard operating instructions for both the Fortran II and Fortran IV programs. No sense switches used in either program.

8. DEFINITION OF TERMS

Terms used in this program are defined in exhibit 3.



App1

9. EXAMPLES

Examples of the Fortran II and Fortran IV programs are shown in exhibits 4 and 5, respectively.

10. PROPOSED FUTURE DEVELOPMENT

It is anticipated that additions to or revisions of this program may be desirable from time to time. It is requested that any user who finds an inadequacy, desirable addition or modification notify the Hydrologic Engineering Center.



App 1

TEST INPUT (IBM 1620)

```
PARTIAL DURATION - INDEPENDENT LOW FLOW EVENTS
LAWRENCE,KANSAS ON WARUSKA RIVER
MONTHLY FLOWS IN ACRE FEET (OCT 1929 THRU SEP 1960)
10      10    1929      9    1959
 6      12

10    1930     472    1140     296     186    1790     406
10    1930     228    4890    5660     180      35      22   15300

10    1960   17570    1890    3310    5460   12980   54330
10    1960   22470    8610    9690    5130    2460     480  144400
```

EXHIBIT 1

EXHIBIT 1

TEST INPUT (IBM 7094)

```
PARTIAL DURATION - INDEPENDENT LOW FLOW EVENTS
CLINTON RESERVOIR ON WARUSKA RIVER,KANSAS
MONTHLY FLOWS IN CFS (500 YEARS SIMULATED STREAMFLOWS)
10        10        0        9       499        2
367
12       120
CLINTON1       1       0.      1.      1.     18.     24.    161.               1
CLINTON1       1     788.    183.    590.     17.     40.      6.    1828.      1
CLINTON1       2     391.     54.     21.     19.      6.     13.               1
CLINTON1       2       0.     36.    380.    324.     62.    157.    1462.      1
CLINTON1       3      95.   5368.    693.   1619.   2407.   1027.               1
CLINTON1       3     658.   3220.     73.    274.    221.   1211.   16867.      1
CLINTON1       4      37.     61.     61.   1284.   1857.   2279.               1
CLINTON1       4     594.    532.    154.     71.     94.   1109.    8133.      1

CLINTON1     494    9211.  16378.    434.   1243.   2719.  19382.               1
CLINTON1     494    3187.   1530.    980.    906.   5671.   1954.   63595.      1
CLINTON1     495   17579.   1742.    345.    141.    285.    617.               1
CLINTON1     495     119.     24.     96.     17.    470.    223.   21658.      1
CLINTON1     496      47.    385.    473.     53.    153.    197.               1
CLINTON1     496      27.    560.    122.    310.     16.      1.    2344.      1
CLINTON1     497       1.     28.     16.      3.      1.    133.               1
CLINTON1     497    1804.     64.    365.     55.     78.    165.    2713.      1
CLINTON1     498     545.    100.     10.     21.     29.     54.               1
CLINTON1     498      12.    893.   1147.    142.     36.     35.    3022.      1
CLINTON1     499       1.     10.     80.    132.     88.    126.               1
CLINTON1     499      25.   1966.      8.      0.     19.   2804.    5260.      1
CLINTON1     500     739.    122.     65.     29.     16.    454.               1
CLINTON1     500     856.   2709.    189.     70.     15.    204.    5467.      1
```

2

PARTIAL DURATION - INDEPENDENT LOW FLOW EVENTS
LAWRENCE,KANSAS ON WARUSKA RIVER
MONTHLY FLOWS IN CFS (OCT 1929 THRU SEP 1964)

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| 10 | 10 | 1929 | 9 | 1963 | 2 | | | |
| 425 | | | | | | | | |
| 24 | 36 | | | | | | | |
| 10 | 1930 | 472 | 1140 | 296 | 186 | 1790 | 406 | |
| 10 | 1930 | 228 | 4890 | 5660 | 180 | 35 | 22 | 15300 |
| 10 | 1931 | 127 | 629 | 365 | 149 | 490 | 2090 | |
| 10 | 1931 | 8520 | 8210 | 4630 | 3580 | 1440 | 621 | 30800 |
| 10 | 1932 | 702 | 77000 | 14100 | 9590 | 3790 | 4690 | |
| 10 | 1957 | 10680 | 15920 | 22880 | 48590 | 227 | 555 | 99170 |
| 10 | 1958 | 3230 | 2900 | 1750 | 3670 | 10310 | 18990 | |
| 10 | 1958 | 11810 | 5870 | 4820 | 70230 | 14900 | 5800 | 154300 |
| 10 | 1959 | 21480 | 27120 | 6100 | 6440 | 11220 | 16000 | |
| 10 | 1959 | 25910 | 11700 | 2630 | 4320 | 430 | 89 | 133400 |
| 10 | 1960 | 17570 | 1890 | 3310 | 5460 | 12980 | 54330 | |
| 10 | 1960 | 22470 | 8610 | 9690 | 5130 | 2460 | 480 | 144400 |
| 10 | 1961 | 35.5 | 10.9 | 33.2 | 11.1 | 73.9 | 909.2 | |
| 10 | 1961 | 632.0 | 1208.3 | 118.7 | 48.9 | 7.6 | 573.9 | |
| 10 | 1962 | 411.8 | 962.2 | 128.6 | 573.2 | 713.8 | 475.1 | |
| 10 | 1962 | 114.7 | 87.6 | 140.8 | 92.6 | 77.8 | 221.4 | |
| 10 | 1963 | 165.9 | 26.6 | 27.5 | 20.2 | 27.4 | 210.7 | |
| 10 | 1963 | 34.5 | 74.2 | 47.6 | 82.6 | 1.9 | 1.1 | |
| 10 | 1964 | .5 | .6 | .5 | 1.7 | 2.0 | 2.2 | |
| 10 | 1964 | 144.3 | 30.5 | 574.0 | 19.1 | 38.4 | 121.8 | |

3

EXHIBIT 1

TEST OUTPUT (IBM 1620)

LAWRENCE,KANSAS ON WARUSKA RIVER
MONTHLY FLOWS IN ACRE FEET (OCT 1929 THRU SEP 1960)

| YEARS (RECORD) | MONTHS | DURATION IN MONTHS |
|---|---|---|
| 31 | 0 | 6 |

EFFECTIVE YEARS
30.58

| NUMBER | VOLUME | FREQ | DATE |
|---|---|---|---|
| 1 | 0.00 | 2.24 | 1 1954 |
| 2 | 2.00 | 5.46 | 2 1939 |
| 3 | 20.00 | 8.69 | 3 1934 |
| 4 | 24.00 | 11.92 | 2 1938 |
| 5 | 214.00 | 15.15 | 2 1956 |
| 6 | 321.00 | 18.38 | 2 1953 |
| 7 | 1280.00 | 21.61 | 3 1933 |
| 8 | 1327.00 | 24.84 | 1 1931 |
| 9 | 1825.60 | 28.07 | 11 1936 |
| 10 | 2791.00 | 31.30 | 12 1934 |
| 11 | 3345.00 | 34.52 | 1 1955 |
| 12 | 4046.00 | 37.75 | 4 1930 |
| 13 | 6442.00 | 40.98 | 2 1957 |
| 14 | 7068.00 | 44.21 | 4 1950 |
| 15 | 7488.00 | 47.44 | 2 1947 |

| YEARS (RECORD) | MONTHS | DURATION IN MONTHS |
|---|---|---|
| 31 | 0 | 12 |

EFFECTIVE YEARS
30.08

| NUMBER | VOLUME | FREQ | DATE |
|---|---|---|---|
| 1 | 1216.00 | 2.27 | 5 1954 |
| 2 | 7825.00 | 5.55 | 4 1938 |
| 3 | 9033.00 | 8.84 | 5 1953 |
| 4 | 9236.00 | 12.12 | 5 1939 |
| 5 | 9364.00 | 15.40 | 6 1955 |
| 6 | 10530.00 | 18.68 | 9 1934 |
| 7 | 13181.00 | 21.96 | 2 1931 |
| 8 | 14589.00 | 25.25 | 7 1956 |
| 9 | 25872.60 | 28.53 | 12 1936 |
| 10 | 33167.00 | 31.81 | 12 1940 |
| 11 | 41118.00 | 35.09 | 7 1933 |
| 12 | 52702.00 | 38.37 | 5 1950 |
| 13 | 77584.00 | 41.65 | 4 1948 |
| 14 | 94384.00 | 44.94 | 9 1935 |
| 15 | 100529.00 | 48.22 | 1 1960 |

PARTIAL DURATION - INDEPENDENT LOW FLOW EVENTS

EXHIBIT 2

TEST OUTPUT (IBM 7094)

PARTIAL DURATION - INDEPENDENT LOW FLOW EVENTS
CLINTON RESERVOIR ON WARUSKA RIVER,KANSAS
MONTHLY FLOWS IN CFS (500 YEARS SIMULATED STREAMFLOWS)

| YEARS (RECORD) | MONTHS | DURATION IN MONTHS |
|---|---|---|
| 500 | 0 | 12 |

| EFFECTIVE YEARS | DRAINAGE AREA (SQ MI) |
|---|---|
| 499.08 | 367.00 |

| NUMBER | VOLUME AC-FT | DEPTH INCHES | RATE CFS | EXCEED FREQ | RECUR INT | ENDING DATE | |
|---|---|---|---|---|---|---|---|
| 1 | 241 | 0,01 | 0,3 | 0.14 | 720.5 | 6 | 472 |
| 2 | 302 | 0,02 | 0,4 | 0.34 | 295.0 | 11 | 342 |
| 3 | 423 | 0,02 | 0,6 | 0.54 | 185.5 | 5 | 88 |
| 4 | 483 | 0,02 | 0,7 | 0.74 | 135.2 | 3 | 129 |
| 5 | 664 | 0,03 | 0,9 | 0.94 | 106.4 | 4 | 46 |
| 6 | 724 | 0,04 | 1,0 | 1.14 | 87.7 | 5 | 416 |
| 7 | 785 | 0,04 | 1,1 | 1.34 | 74.6 | 6 | 473 |
| 8 | 845 | 0,04 | 1,2 | 1.54 | 64.9 | 10 | 147 |
| 9 | 906 | 0,05 | 1,3 | 1.74 | 57.5 | 4 | 471 |
| 10 | 1026 | 0,05 | 1,4 | 1.94 | 51.5 | 7 | 289 |
| 11 | 1509 | 0,08 | 2,1 | 2.14 | 46.7 | 9 | 116 |
| 12 | 1570 | 0,08 | 2,2 | 2.34 | 42.7 | 2 | 139 |
| 13 | 1630 | 0,08 | 2,3 | 2.54 | 39.3 | 7 | 243 |
| 14 | 1690 | 0,09 | 2,3 | 2.74 | 36.5 | 5 | 395 |
| 15 | 1751 | 0,09 | 2,4 | 2.94 | 34.0 | 5 | 354 |
| 16 | 1811 | 0,09 | 2,5 | 3.14 | 31.8 | 12 | 283 |
| 17 | 1992 | 0,10 | 2,8 | 3.34 | 29.9 | 6 | 254 |
| 237 | 64478 | 3,29 | 89,0 | 47,39 | 2.1 | 5 | 173 |
| 238 | 65022 | 3,32 | 89,8 | 47.59 | 2.1 | 3 | 406 |
| 239 | 65927 | 3,37 | 91,0 | 47,79 | 2.1 | 8 | 101 |
| 240 | 66410 | 3,39 | 91,7 | 47.99 | 2.1 | 9 | 439 |
| 241 | 66410 | 3,39 | 91,7 | 48,19 | 2.1 | 4 | 442 |
| 242 | 66531 | 3,40 | 91,8 | 48.39 | 2.1 | 6 | 474 |
| 243 | 66591 | 3,40 | 91,9 | 48,59 | 2.1 | 3 | 131 |
| 244 | 67014 | 3,42 | 92,5 | 48,79 | 2.0 | 12 | 235 |
| 245 | 67497 | 3,45 | 93,2 | 48,99 | 2.0 | 4 | 464 |
| 246 | 67739 | 3,46 | 93,5 | 49,19 | 2.0 | 4 | 64 |
| 247 | 68342 | 3,49 | 94,3 | 49,39 | 2.0 | 7 | 17 |
| 248 | 68463 | 3,50 | 94,5 | 49.59 | 2.0 | 5 | 228 |
| 249 | 71180 | 3,64 | 98,3 | 49,79 | 2.0 | 2 | 364 |
| 250 | 71361 | 3,65 | 98,5 | 49.99 | 2.0 | 5 | 159 |
| 251 | 71421 | 3,65 | 98,6 | 50.19 | 2.0 | 4 | 365 |

EXHIBIT 2

| YEARS (RECORD) | MONTHS | DURATION IN MONTHS |
|---|---|---|
| 500 | 0 | 120 |

| EFFECTIVE YEARS | DRAINAGE AREA (SQ MI) |
|---|---|
| 490.08 | 367.00 |

| NUMBER | VOLUME AC-FT | DEPTH INCHES | RATE CFS | EXCEED FREQ | RECUR INT | ENDING DATE | |
|---|---|---|---|---|---|---|---|
| 1 | 246805 | 12.61 | 34.1 | 0.14 | 707.5 | 12 | 244 |
| 2 | 289851 | 14.81 | 40.0 | 0.35 | 289.7 | 10 | 187 |
| 3 | 299390 | 15.30 | 41.3 | 0.55 | 182.1 | 4 | 54 |
| 4 | 319434 | 16.32 | 44.1 | 0.75 | 132.8 | 6 | 137 |
| 5 | 338934 | 17.32 | 46.8 | 0.96 | 104.5 | 3 | 477 |
| 6 | 364713 | 18.63 | 50.3 | 1.16 | 86.2 | 5 | 437 |
| 7 | 368879 | 18.85 | 50.9 | 1.36 | 73.3 | 3 | 213 |
| 8 | 371234 | 18.97 | 51.2 | 1.57 | 63.8 | 5 | 93 |
| 9 | 441689 | 22.57 | 61.0 | 1.77 | 56.4 | 4 | 123 |
| 10 | 449598 | 22.97 | 62.1 | 1.98 | 50.6 | 5 | 292 |
| 11 | 570525 | 29.15 | 78.8 | 2.18 | 45.9 | 4 | 202 |
| 12 | 634158 | 32.40 | 87.5 | 2.38 | 41.9 | 3 | 355 |
| 13 | 693625 | 35.44 | 95.7 | 2.59 | 38.6 | 4 | 304 |
| 14 | 694531 | 35.48 | 95.9 | 2.79 | 35.8 | 5 | 447 |
| 15 | 739026 | 37.76 | 102.0 | 3.00 | 33.4 | 1 | 386 |
| 16 | 748746 | 38.25 | 103.3 | 3.20 | 31.3 | 4 | 174 |
| 17 | 765107 | 39.09 | 105.6 | 3.40 | 29.4 | 6 | 260 |
| 18 | 765892 | 39.13 | 105.7 | 3.61 | 27.7 | 1 | 318 |
| 19 | 814432 | 41.61 | 112.4 | 3.81 | 26.2 | 5 | 81 |
| 20 | 846671 | 43.26 | 116.9 | 4.02 | 24.9 | 4 | 44 |
| 21 | 854338 | 43.65 | 117.9 | 4.22 | 23.7 | 5 | 423 |
| 22 | 961078 | 49.10 | 132.7 | 4.42 | 22.6 | 10 | 369 |
| 23 | 989936 | 50.58 | 136.6 | 4.63 | 21.6 | 4 | 282 |
| 24 | 992774 | 50.72 | 137.0 | 4.83 | 20.7 | 3 | 148 |
| 25 | 1002011 | 51.19 | 138.3 | 5.03 | 19.9 | 5 | 160 |
| 26 | 1007746 | 51.49 | 139.1 | 5.24 | 19.1 | 7 | 397 |
| 27 | 1009074 | 51.55 | 139.3 | 5.44 | 18.4 | 5 | 106 |
| 28 | 1050852 | 53.69 | 145.1 | 5.65 | 17.7 | 3 | 331 |
| 29 | 1228832 | 62.78 | 169.6 | 5.85 | 17.1 | 5 | 71 |
| 30 | 1259562 | 64.35 | 173.9 | 6.05 | 16.5 | 3 | 21 |
| 31 | 1483002 | 75.77 | 204.7 | 6.26 | 16.0 | 3 | 408 |
| 32 | 1762288 | 90.04 | 243.3 | 6.46 | 15.5 | 7 | 270 |
| 33 | 1846146 | 94.32 | 254.8 | 6.67 | 15.0 | 12 | 343 |
| 34 | 1856289 | 94.84 | 256.2 | 6.87 | 14.6 | 11 | 492 |
| 35 | 2192264 | 112.00 | 302.6 | 7.07 | 14.1 | 4 | 224 |
| 36 | 2236578 | 114.27 | 308.7 | 7.28 | 13.7 | 3 | 467 |
| 37 | 2642224 | 134.99 | 364.7 | 7.48 | 13.4 | 9 | 10 |
| 38 | 3369840 | 172.16 | 465.1 | 7.69 | 13.0 | 8 | 234 |
| 39 | 3660173 | 187.00 | 505.2 | 7.89 | 12.7 | 2 | 32 |

INDEPENDENT EVENTS EXHAUSTED



EXHIBIT 2

PARTIAL DURATION - INDEPENDENT LOW FLOW EVENTS
LAWRENCE, KANSAS ON WARUSKA RIVER
MONTHLY FLOWS IN CFS (OCT 1929 THRU SEP 1964)

| YEARS (RECORD) | MONTHS | DURATION IN MONTHS |
|---|---|---|
| 35 | 0 | 24 |

| EFFECTIVE YEARS | DRAINAGE AREA (SQ MI) |
|---|---|
| 33.08 | 425.00 |

| NUMBER | VOLUME AC-FT | DEPTH INCHES | RATE CFS | EXCEED FREQ | RECUR INT | ENDING DATE |
|---|---|---|---|---|---|---|
| 1 | 10172 | 0.45 | 7.0 | 2.07 | 48.2 | 5 1954 |
| 2 | 25860 | 1.14 | 17.8 | 5.06 | 19.8 | 6 1956 |
| 3 | 39683 | 1.75 | 27.4 | 8.05 | 12.4 | 5 1939 |
| 4 | 42382 | 1.87 | 29.3 | 11.04 | 9.1 | 4 1935 |
| 5 | 51924 | 2.29 | 35.8 | 14.02 | 7.1 | 9 1931 |
| 6 | 86641 | 3.82 | 59.8 | 17.01 | 5.9 | 5 1964 |
| 7 | 99302 | 4.38 | 68.5 | 20.00 | 5.0 | 5 1941 |
| 8 | 153981 | 6.79 | 106.3 | 22.99 | 4.4 | 5 1937 |
| 9 | 170227 | 7.51 | 117.5 | 25.97 | 3.8 | 6 1958 |
| 10 | 215257 | 9.50 | 148.6 | 28.96 | 3.5 | 2 1961 |
| 11 | 234700 | 10.35 | 162.0 | 31.95 | 3.1 | 5 1950 |
| 12 | 292374 | 12.90 | 201.8 | 34.94 | 2.9 | 5 1948 |
| 13 | 345503 | 15.24 | 238.4 | 37.92 | 2.6 | 2 1944 |
| 14 | 549388 | 24.24 | 379.2 | 40.91 | 2.4 | 4 1946 |
| 15 | 714351 | 31.52 | 493.0 | 43.90 | 2.3 | 5 1952 |

INDEPENDENT EVENTS EXHAUSTED

| YEARS (RECORD) | MONTHS | DURATION IN MONTHS |
|---|---|---|
| 35 | 0 | 36 |

| EFFECTIVE YEARS | DRAINAGE AREA (SQ MI) |
|---|---|
| 32.08 | 425.00 |

| NUMBER | VOLUME AC-FT | DEPTH INCHES | RATE CFS | EXCEED FREQ | RECUR INT | ENDING DATE |
|---|---|---|---|---|---|---|
| 1 | 27164 | 1.20 | 12.5 | 2.14 | 46.8 | 6 1955 |
| 2 | 56364 | 2.49 | 25.9 | 5.22 | 19.2 | 4 1940 |
| 3 | 92986 | 4.10 | 42.8 | 8.30 | 12.1 | 4 1935 |
| 4 | 186557 | 8.23 | 85.8 | 11.38 | 8.8 | 6 1958 |
| 5 | 314658 | 13.88 | 144.8 | 14.46 | 6.9 | 4 1950 |
| 6 | 341180 | 15.05 | 157.0 | 17.54 | 5.7 | 5 1964 |
| 7 | 431673 | 19.04 | 198.6 | 20.62 | 4.9 | 4 1943 |
| 8 | 773034 | 34.10 | 355.7 | 23.69 | 4.2 | 4 1947 |

INDEPENDENT EVENTS EXHAUSTED

EXHIBIT 2

DEFINITIONS 23-J2-L247

| | | |
|---|---|---|
| ACCUM | - | Summation of flow volumes for MD months in the Fortran II program. Average rate of flow in cfs for current duration in the Fortran IV program. |
| BPP | - | Beards plotting position (exceedence frequency). |
| * DA | - | Drainage area in square miles. |
| ** DEPTH | - | Depth in inches for given drainage area. |
| EFFYRS | - | Effective years of record used in determining plotting positions. Based on the total number of months for which full duration volumes can be determined. |
| FLOW | - | Array of monthly flows. |
| I1 | - | Number of non-applicable months read at beginning of flow record. |
| I2 | - | Number of non-applicable months read at end of flow record. |
| ICYCLE | - | Number of durations. |
| ID | - | Identification number on flow data cards. |
| * INUNIT | - | Units of input flow data. |

| Flow Unit: | Supply to INUNIT: |
|---|---|
| acre-feet | 1 |
| cfs | 2 |
| inches runoff | 3 |

| | | |
|---|---|---|
| IYEAR | - | Year of event. |
| IYR1 | - | Starting year of flow record. |
| lYR | - | Year on flow cards. |
| LMON | - | Last calendar month of flow record. |
| LVP | - | Period of low volume plus MON1. |
| LYR | - | Last year of flow record. |
| M1 | - | Calendar month of first monthly flow on flow data cards. |
| MOE | - | Month of event. |

DEFINITIONS (cont'd)

MD - Current duration in months.

MON - Month on flow cards.

MON1 - Starting calendar month of flow record.

MONDUR - Array of durations in months.

NLFP - Maximum number of low-flow events to be selected from the record for a given duration.

NP - Total number of monthly flow periods. Starting with M1 of the first year and ending with LMON of the last year.

P1 - Beards plotting position for <u>lowest</u> volume event.

$$1 - P_1 = .5^{\frac{1}{n}}$$

P2 - Beards plotting position for highest volume event. Complement of P1.

* RATE - Average rate of flow in cfs.

* R1 - Recurrence interval in years.

T - Same as MD.

* VOL - Volume of an event for current duration.

VOLUME - Volume of an event for current duration in the Fortran II program. Average rate of flow in cfs during an event in the Fortran IV program.

* Not applicable to the Fortran II program.

EXHIBIT 3

EXHIBIT 4

```
*07044
C      PARTIAL DURATION - INDEPENDENT EVENTS
C      PROGRAMMER- W.L. SHARP       PROGRAM NO. 23-J2-J247    FOR IBM 1620
C      SELECTION OF LOW FLOW PERIODS FROM PERIOD OF RECORD FLOWS
C      BASED ON NO OVERLAP OF DURATIONS
C      LIMITED TO  75 YEAR FLOW RECORD AND 20 DURATIONS
       DIMENSION MONDUR(20), FLOW( 900), ACCUM( 900)
   100 FORMAT (10F8.0)
   110 FORMAT (10I8)
   111 FORMAT (40A2)
   112 ICYCLE=0
C      READ THREE TITLE CARDS
       READ 111, (FLOW(I),I=1,120)
       PUNCH 111, (FLOW(I),I=1,120)
       READ 110,M1,MON1,IYR1,LMON,LYR
       READ 110,(MONDUR(N),N=1,20)
       J=(LYR-IYR1)*2+2
       N=1
       K=6
       DO 120 I=1,J
   115 FORMAT (2I8,6F8.0)
       READ 115,ID,IYR,( FLOW(N),N=N,K)
       N=K+1
   120 K=K+6
       NP=K-6
       IF(MON1-M1)122,124,124
   122 I1=I2+MON1-M1
       GO TO 125
   124 I1=MON1-M1
   125 IF(LMON-M1)126,127,127
   126 I2=M1-LMON-1
       GO TO 128
   127 I2=LMON-12+M1-1
   128 XNP=NP-I1-I2
       NP=NP-I2
       IYEARS=XNP/12.0
       YEARS=IYEARS
       MONTHS=XNP-(YEARS*12.0)
   130 ICYCLE=ICYCLE+1
       IF(ICYCLE-20)140,140,400
   140 MD=MONDUR(ICYCLE)
```

EXHIBIT 4

2

```
      IF(MD)400,400,150
C     COMPUTE BEARDS PLOTTING POSITIONS FOR SMALLEST AND LARGEST EVENTS
  150 XMD=MD
      EFFYRS=(XNP-(XMD-1.))/12.0
      P2=(0.5**(1.0/EFFYRS))*100.0
      P1=100.0-P2
  160 FORMAT (21HYEARS (RECORD) MONTHS,5X,18HDURATION IN MONTHS)
      PUNCH 160
  170 FORMAT (I4,10X,I5,15X,I3/79X,1H1)
      PUNCH 170,IYEARS,MONTHS,MD
  172 FORMAT (15HEFFECTIVE YEARS)
      PUNCH 172
  174 FORMAT (F10.2)
      PUNCH 174,EFFYRS
  180 FORMAT (79X,1H1/3X,6HNUMBER,3X,6HVOLUME,4X,4HFREQ,5X,4HDATE)
      PUNCH 180
C     ACCUMULATE FLOWS FOR GIVEN DURATION
      ACCUM(1)=FLOW(1)
      DO 220 N=2,MD
  220 ACCUM(N)=ACCUM(N-1)+FLOW(N)
      N=MD+1
      DO 230 N=N,NP
      J=N-MD
  230 ACCUM(N)=ACCUM(N-1)-FLOW(J)+FLOW(N)
      NLFP=NP/MD
      IF(NLFP-((NP/12)/2))250,250,240
  240 NLFP=(NP/12)/2
C     DETERMINE FLOW PERIODS IN ASCENDING ORDER AND PRINT
  250 DO 380 I=1,NLFP
      LVP=I1+MD
      VOLUME=ACCUM(LVP)
      J=LVP+1
      DO 270 N=J,NP
      IF(ACCUM(N)-VOLUME)260,270,270
  260 VOLUME=ACCUM(N)
      LVP=N
  270 CONTINUE
      IF(VOLUME-(9999999.))275,274,274
  272 FORMAT (29H INDEPENDENT EVENTS EXHAUSTED)
  274 PUNCH 272
      GO TO 390
```

```
C      ASSIGN LARGE NUMBERS TO ADJACENT ACCUMULATIVE FLOW VOLUMES
   275 N=LVP-MD+1
       J=LVP+MD-1
       IF(J-NP)290,290,280
   280 J=NP
   290 DO 300 N=N,J
   300 ACCUM(N)=9999999.
C      COMPUTE BEARDS PLOTTING POSITION (FREQUENCY)
       IF(I-1)310,310,320
   310 BPP=P1
       GO TO 330
   320 EVENT=I
       BPP=P2-(((EFFYRS-EVENT)/(EFFYRS-1.))*(P2-P1))
       IF(BPP-60.0)330,330,390
C      COMPUTE DATE OF EVENT
   330 LVP=LVP+M1-1
       IF(LVP-12)340,340,350
   340 IYEAR=IYR1
       MOE=LVP
       GO TO 370
   350 XLVP=LVP
       YEARS=(XLVP/12.0)-.001
       J=YEARS
       IYEAR=J+IYR1
       MOE=LVP-J*12
   360 FORMAT (I6,F12.2,F8.2,I6,I5)
   370 PUNCH 360,I,VOLUME,BPP,MOE,IYEAR
   380 CONTINUE
   385 FORMAT (79X,1H9)
   390 PUNCH 385
       GO TO 130
   400 GO TO 112
       END
```



EXHIBIT 4

EXHIBIT 4

```
C     PARTIAL DURATION - INDEPENDENT EVENTS
C     PROGRAMMER- W.L. SHARP      PROGRAM NO. 23-J2-J247   FOR IBM 7094
C     SELECTION OF LOW FLOW PERIODS FROM PERIOD OF RECORD FLOWS
C     BASED ON NO OVERLAP OF DURATIONS
C     LIMITED TO 1000 YEAR FLOW RECORD AND 20 DURATIONS
      DIMENSION MONDUR(20),FLOW(12000),ACCUM(12000)
  100 FORMAT (10F8.0)
  110 FORMAT (10I8)
  111 FORMAT (20A4)
  112 ICYCLE=0
C     READ THREE TITLE CARDS
      READ (5,111) (FLOW(I),I=1,60)
      WRITE (6,111) (FLOW(I),I=1,60)
      READ (5,110)M1,MON1,IYR1,LMON,LYR,INUNIT
      READ (5,100) DA
      READ (5,110) (MONDUR(N),N=1,20)
      J=(LYR-IYR1)*2+2
      N=1
      K=6
      DO 120 I=1,J
  115 FORMAT (2A4,I8,6F8.0)
      READ (5,115) ID,ID,IYR,(FLOW(N),N=N,K)
      N=K+1
  120 K=K+6
      NP=K-6
      IF(MON1-M1)122,124,124
  122 I1=12+MON1-M1
      GO TO 125
  124 I1=MON1-M1
  125 IF(LMON-M1)126,127,127
  126 I2=M1-LMON-1
      GO TO 128
  127 I2=LMON-12+M1-1
  128 XNP=NP-I1-I2
      NP=NP-I2
      IYEARS=XNP/12.0
      YEARS=IYEARS
      MONTHS=XNP-(YEARS*12.0)
  130 ICYCLE=ICYCLE+1
      IF(ICYCLE-20)140,140,400
  140 MD=MONDUR(ICYCLE)
```

```
      IF(MD)400,400,150
C     COMPUTE BEARDS PLOTTING POSITIONS FOR SMALLEST AND LARGEST EVENTS
  150 XMD=MD
      EFFYRS=(XNP-(XMD-1.))/12.0
      P2=(0.5**(1.0/EFFYRS))*100.0
      P1=100.0-P2
  160 FORMAT (1X,21HYEARS (RECORD) MONTHS,5X,18HDURATION IN MONTHS)
      WRITE (6,160)
  170 FORMAT (I5,10X,I5,15X,I3)
      WRITE (6,170) IYEARS,MONTHS,MD
  172 FORMAT (/1X,15HEFFECTIVE YEARS,11X,21HDRAINAGE AREA (SQ MI))
      WRITE (6,172)
  174 FORMAT (F11.2,16X,F10.2)
      WRITE (6,174) EFFYRS,DA
  180 FORMAT (/3X,6HNUMBER,3X,6HVOLUME,2X,5HDEPTH,6X,4HRATE,2X,6HEXCEED,
     12X,5HRECUR,2X,6HENDING/13X,5HAC-FT,1X6HINCHES,7X3HCFS,3X,4HFREQ,
     24X,3HINT,4X,4HDATE)
      WRITE (6,180)
C     ACCUMULATE FLOWS FOR GIVEN DURATION
      ACCUM(1)=FLOW(1)
      DO 220 N=2,MD
  220 ACCUM(N)=ACCUM(N-1)+FLOW(N)
      N=MD+1
      DO 230 N=N,NP
      J=N-MD
  230 ACCUM(N)=ACCUM(N-1)-FLOW(J)+FLOW(N)
      T=MD
      IF(INUNIT-2)231,233,235
C     CONVERT AC-FT TO AVERAGE CFS
  231 DO 232 N=MD,NP
  232 ACCUM(N)=ACCUM(N)*.016598/T
      GO TO 238
C     CONVERT CFS TO AVERAGE CFS
  233 DO 234 N=MD,NP
  234 ACCUM(N)=ACCUM(N)/T
      GO TO 238
C     CONVERT INCHES RUNOFF TO AVERAGE CFS
  235 DO 236 N=MD,NP
  236 ACCUM(N)=ACCUM(N)*DA*.88523/T
  238 NLFP=NP/MD
      IF(NLFP-((NP/12)/2))250,250,240
```



EXHIBIT 4

EXHIBIT 4

```
  240 NLFP=((NP/12)/2) +1
C     DETERMINE FLOW PERIODS IN ASCENDING ORDER AND PRINT
  250 DO 380 I=1,NLFP
      LVP=I1+MD
      VOLUME=ACCUM(LVP)
      J=LVP+1
      DO 270 N=J,NP
      IF(ACCUM(N)-VOLUME)260,270,270
  260 VOLUME=ACCUM(N)
      LVP=N
  270 CONTINUE
      IF(VOLUME-9999999.)275,274,274
  272 FORMAT (29H INDEPENDENT EVENTS EXHAUSTED)
  274 WRITE (6,272)
      GO TO 390
C     ASSIGN LARGE NUMBERS TO ADJACENT ACCUMULATIVE FLOW VOLUMES
  275 N=LVP-MD+1
      J=LVP+MD-1
      IF(J-NP)290,290,280
  280 J=NP
  290 DO 300 N=N,J
  300 ACCUM(N)=9999999.
C     COMPUTE BEARDS PLOTTING POSITION (FREQUENCY)
      IF(I-1)310,310,320
  310 BPP=P1
      GO TO 330
  320 EVENT=I
      BPP=P2-(((EFFYRS-EVENT)/(EFFYRS-1.))*(P2-P1))
      IF(BPP-60.0)330,330,390
C     COMPUTE DATE OF EVENT
  330 LVP=LVP+M1-1
      IF(LVP-12)340,340,350
  340 IYEAR=IYR1
      MOE=LVP
      GO TO 370
  350 XLVP=LVP
      YEARS=(XLVP/12.0)-.001
      J=YEARS
      IYEAR=J+IYR1
      MOE=LVP-J*12
  370 RATE=VOLUME
```

```
      VOL=RATE *60.373*T
      DEPTH=(VOL*.01875)/DA
      RI=100./BPP
  375 FORMAT (I6,F12.0,F7.2,F10.1,F8.2,F7.1,I4,I5)
      WRITE (6,375) I,VOL,DEPTH,RATE,BPP,RI,MOE,IYEAR
  380 CONTINUE
  385 FORMAT (1H1)
  390 WRITE (6,385)
      GO TO 130
  400 GO TO 112
      END
```



EXHIBIT 4

INPUT 23-J2-L247

A Three output title cards

B Job data card

1. M1 - Calendar month of first monthly flow on flow data cards. Not necessarily the starting month.

2. MON1 - Starting calendar month of flow record. Can be any calendar month.

3. IYR1 - Starting calendar year of flow record coinciding with M1.

4. LMON - Last calendar month of flow record. Can be any calendar month.

5. LYR - Last calendar year of flow record, must coincide with beginning month of last year's flow.

* 6. INUNIT - Units of input flow data.

| Flow Unit: | Supply to INUNIT: |
|---|---|
| acre-feet | 1 |
| cfs | 2 |
| inches runoff | 3 |

*C Job data card

DA - Drainage area in square miles.

D Job data cards

MONDUR (1) thru (20) - Number of months within a duration. Limited to 20 durations. <u>Always</u> furnish 2 cards. Cannot specify a one month duration.

E Flow data cards. Two cards per year.

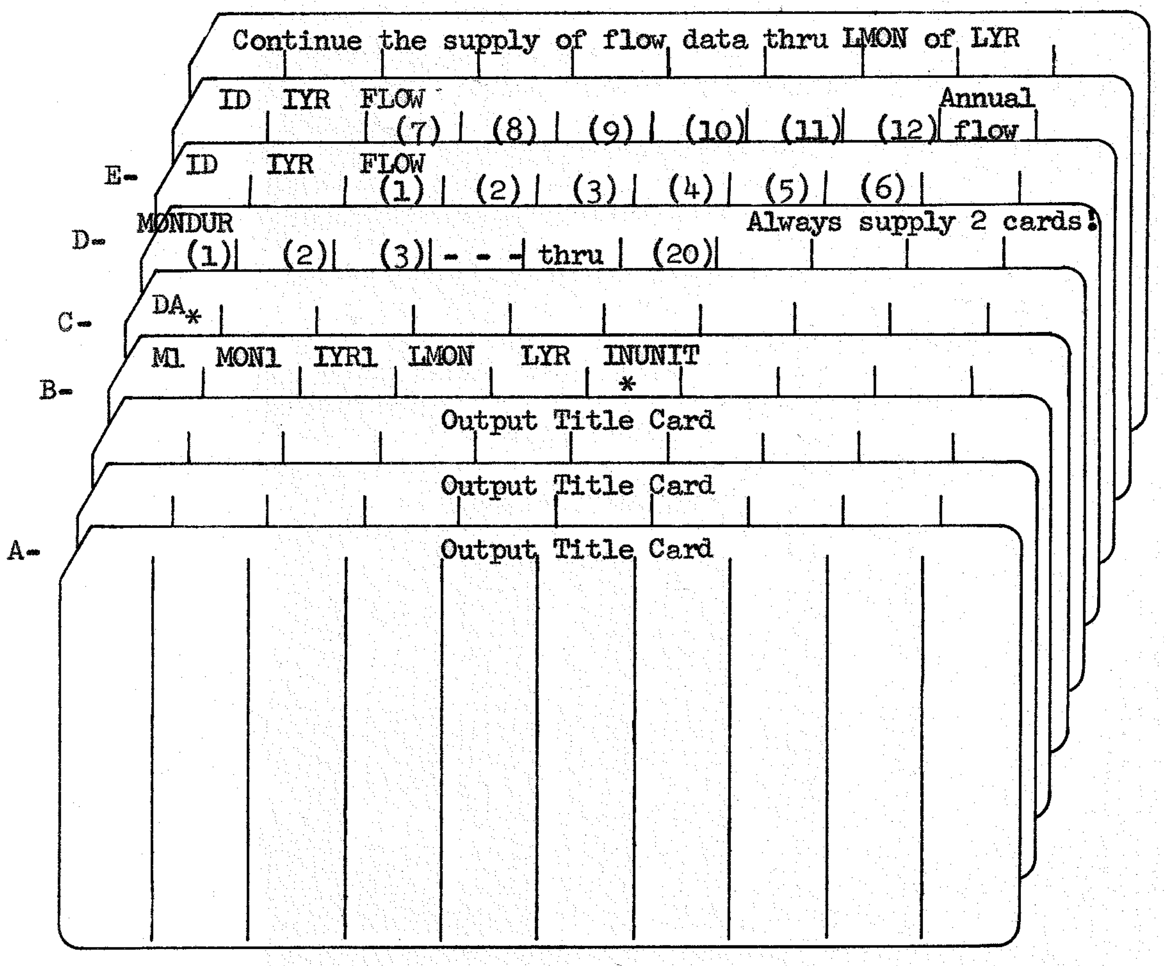
1. ID - Station identification number.

2. IYR - Year. Can be a water year.

3. FLOW (1) thru (6) on first card, (7) thru (12) on second card. - Monthly flows. Both cards contain ID and IYR. Suggest the annual flow be included on the second card of each year in columns 65-72 for identification.

* Omit when using the Fortran II program.

SUMMARY OF REQUIRED CARDS
23-J2-L247




Notes:

1. * - omit in Fortran II program.
2. Multiple runs may be performed. (Include title cards for each successive run).
3. Furnish four blank cards at end of data for normal exit.

EXHIBIT 6

Appendix 2

# Conversion Constants

APPENDIX II

## CONVERSION CONSTANTS

| | | |
|---|---|---|
| 1 cubic meter | = | 35.314 cubic feet |
| 1 acre-foot | = | 1233.505 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 | = | 10.76365 square feet |
| 1 acre | = | 4046.95 square meters |
| 1 square mile | = | 2.59 square kilometers |

| | | |
|---|---|---|
| 1 Inch | = | 2.54 centimeters |
| 1 Meter | = | 3.28083 feet |
| 1 Kilometer | = | 3280.83 feet |
| 1 Mile | = | 1.609 kilometers |

| | | |
|---|---|---|
| 24 hour-cfs | = | 1.9835 acre-feet |
| 28 day-cfs | = | 55.538 acre-feet |
| 30 day-cfs | = | 59.505 acre-feet |
| 31 day-cfs | = | 61.489 acre-feet |
| 30.475 day-cfs | = | 60.373 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 U.S. gallons per minute

| | | |
|---|---|---|
| .167 inches/week | = | cfs/1000 acres |
| .667 inches/28 day | = | cfs/1000 acres |
| .714 inches/30 day | = | cfs/1000 acres |
| .724 inches/30.47 days | = | cfs/1000 acres |
| .737 inches/31 days | = | cfs/1000 acres |
| 8.688 inches/year | = | cfs/1000 acres |

Appendix 3

# Glossary

## GLOSSARY

Base-load plant. A hydroelectric plant which is designed to supply power to meet the base load and operates essentially at a constant load, thus having a high plant factor.

Critical duration. The length of time during which the largest volume must be released from storage in order to provide a specified yield.

Critical period. The actual period in a sequential record, either historical or simulated, which requires the largest volume from storage to provide a specified yield. The critical period is often taken as time from beginning of storage utilization to the time that the conservation pool refills during the period when the reservoir is drawn down to its lowest level. The period from beginning of storage utilization to minimum pool level is referred to as the critical drawdown period.

Cutoff storage. The remaining active conservation storage volume at which it is desirable to discontinue releases from a reservoir for one purpose in order to assure future releases for a higher priority purpose.

Independent events. Statistically, independent events are events which do not affect the probability of occurrence of one another in a given series. The specialized case of this definition used in this manual refers to successive flow volumes for a given duration. A degree of independence is assured by selecting volume events so that no flow data is used in more than one volume event.

Natural flow. The flow resulting from natural hydrologic conditions. (Unaffected by man-made structures which would alter the natural regime).

Non-project conditions. The conditions that would be expected to exist in the future if a project were not built, also called pre-project conditions.

Non-sequential mass curve. A curve showing the relationship between various durations and minimum recorded flow volumes or minimum flow volumes with specified probabilities. The curve is developed without regard to sequential occurrences of flows and therefore the critical duration can be obtained from the curve, but the critical period cannot.

Peaking plant. A hydroelectric plant which is designed to supply power during maximum load periods. Peaking plants ordinarily have low plant factors.

Plant factor. The ratio of the average hydroelectric load on the plant for a stated period, to the aggregate rating of all the generating equipment installed in the plant.

Probability of shortage. The likelihood or chance that a shortage will occur in any given year based on sample data. (Sometimes expressed as a percentage, i.e., 10% probability of shortage indicates that there is one chance in ten that a shortage will occur in any given year.)

Project conditions. The conditions that would be expected to exist in the future if a project were built.

Recurrence frequency. The frequency per year with which an event of a given magnitude can be expected to be surpassed. For example, an event with a recurrence frequency of .01 will be surpassed, on the average, once in a hundred years.

Recurrence interval. The average interval of time between values more extreme than a specified magnitude. Reciprocal of the recurrence frequency (may also be called the return period or exceedence interval or non-exceedence interval).

<u>Routing interval</u>. The basic time interval involved in a sequential routing (i.e., a weekly routing interval indicates that the routing will be composed of sequential periods one week in length).

<u>Runoff</u>. The portion of rainfall and snowmelt which runs off a drainage area and appears in surface streams.

<u>Sequential mass curve</u>. A curve showing the relationship between accumulated sequential flow volumes and time. The curve is developed by accumulating sequential flows and plotting the accumulated flow volume versus the actual time when that accumulation occurred. A sequential mass curve may be used in analysis of both historical and synthesized flow records.

<u>Sequential routing study</u>. A study which simulates the operation of a reservoir or system of reservoirs using historical or synthesized flow data in sequence.

<u>Serial correlation</u>. The correlation of an event with a preceding event.

<u>Shortage</u>. A deficit in supply, often expressed as a ratio to or percentage of a specified demand or target yield for a given period such as one year, (i.e., a 20% shortage indicates that there is a deficit in supply equivalent to 20% of the demand or target yield).

<u>Shortage index</u>. As defined in this text, the sum of the squares of the annual shortages over a 100-year period, each shortage expressed as a ratio to the annual target yield.

Simulated (or synthesized) flows. Flow values which have been sequentially synthesized using the statistical characteristics of actual flow records.

Storage. The volume of water in a reservoir.

Yield. The amount or schedule of supply at one or more specified locations (usually expressed in terms of a draft rate, i.e., a yield of 120 cubic meters per second).

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