FEASIBILITY OF SEASONAL MULTIPURPOSE RESERVOIR OPERATION IN TEXAS

A Thesis

by

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ABSTRACT

Feasibility of Seasonal Multipurpose
Reservoir Operation in Texas. (May 1986)
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The purpose of the study reported here was to evaluate the potential for increasing the beneficial use of existing reservoirs in the state of Texas through seasonal reallocations of storage capacity between flood control and conservation purposes. The research included an investigation of: (1) water resources problems and needs in the state, (2) reservoir operating procedures, (3) seasonal factors affecting reservoir operation, (4) pertinent state-of-the-art modeling capabilities, and (5) a case study. The findings of the research associated with these tasks are respectively: (1) increasing population and economic growth and depleting groundwater reserves are causing a greater reliance on surface water supplies, (2) reservoir planning and design in Texas uses a top of conservation pool that stays constant all year, (3) two of the factors affecting seasonal reservoir operation (flood threat and water availability) are not characterized by strong seasonal trends as are flood damage potential and water demand, (4) although many models are proposed and available to increase the beneficial use of reservoirs, there exists a large gap between the research community and actual practice, and (5) the results of the case study involving Waco Lake in McLennan County lead to the conlusion that seasonal rule curve operation offers significant potential for improving reservoir

operations in the state in those situations in which needs for flood control and conservation purposes are severely taxing the available storage capacity. In an analysis of historical flood data, seasonal rule curve operation reduced flood damages in three of the seven recorded flood events as compared to reservoir operating plans using a constant top of conservation pool elevation.

Managing Texas reservoirs by seasonal rule curve operation shows the potential for increasing the firm yield from a reservoir and at the same time decreasing damages due to flooding. However, seasonal rule curve operation may adversely affect lake-front property.

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Dr. Ralph A. Wurbs of the Civil Engineering faculty of Texas A&M University deserves foremost credit for his patience, inspiration, and leadership during the preparation of this thesis. His knowledge of reservoir operations was an invaluable asset upon which the author relied heavily. Dr. Kenneth M. Strzepek also deserves credit in this thesis work for using his special skills with computers. Without his expertise, many of the investigations of this report would have proven extremely time consuming at best.

Many other people have contributed to the knowledge contained in this report. The United States Army Corps of Engineers Forth Worth District provided critical information on a majority of the physical data required by this thesis. It would have literally been impossible to obtain some this information from any other source.

L. Moris Cabezas provided necessary information and worked with the author in the final results of this report, as did Lonnie C. Roy.

John R. Denison of the Paris Junior College drafting department is responsible for much of the graphics presented herein. His knowledge of computer aided graphics was most beneficial in the preparation of the final portions of this thesis.

The author appreciates the City of Paris engineering department for its indulgence during the completion of this study. The proficient typing of department secretary Gloria Schubert was instrumental in much of the work presented herein.

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CHAPTER I

INTRODUCTION

Problem Statement

Population and economic growth combined with diminishing ground water reserves are resulting in ever-increasing demands on surface water resources in Texas as well as elsewhere. The climate of the state is characterized by extremes of floods and droughts. Reservoirs are necessary to control and utilize the highly variable streamflow. Due to a number of economic, environmental, and institutional constraints, construction of additional new reservoir projects is becoming much more difficult now than in the past. Consequently, optimizing the beneficial use of existing reservoirs is becoming increasingly more important.

Reservoir operation is based on the conflicting objectives of maximizing the amount of water available for conservation purposes and maximizing the amount of empty space available for storing flood waters to reduce downstream flooding damages. Reservoirs are normally operated either for conservation purposes only, for flood control only, or for combined conservation and flood control with pools for each being separated by a designated top of conservation pool elevation. Institutional arrangements for constructing and operating reservoirs are based on having separate pools for flood control and conservation. Planning, design, and operational problems associated with flood control are handled separately from those associated with conservation.

This thesis conforms in style and format to the standards of the Journal of the Hydraulics Division, Proceedings of the American Society of Civil Engineers.

However, increasing needs for providing water for various uses and reducing flood damages necessitate that limited reservoir storage capacity be used as beneficially as possible. Consequently, consideration of the interactions and tradeoffs between conservation and flood control operations is becoming increasingly more important.

Risk of flooding, flood damage susceptibility, water supply demands, and water availability vary seasonally. Rule curve operating policies can be adopted to reflect seasonally varying conditions. A rule curve specifies the top of conservation pool elevation as a function of time of the year. Storage capacity is reallocated between flood control and conservation purposes in a set annual cycle. Seasonally varying operating procedures have been adopted to only a very limited extent in Texas. Release policies are generally constant throughout the year. The purpose of this study was to evaluate the potential for increasing the beneficial use of existing reservoirs in Texas through seasonal reallocations between flood control and conservation purposes.

Study Purpose and Scope

The objectives of the study were as follows: (1) to evaluate the potential for increasing the overall benefits provided by reservoirs in Texas by seasonal reallocations of storage capacity between flood control and conservation or otherwise operating conjunctively for flood control and conservation purposes considering the seasonal variations in flood threat, flood damage potential, water demand, and water availability; and (2) to develop a detailed strategy or method for determining optimum seasonal reservoir release policies.

These objectives were accomplished through the following research procedures: (1) review of reservoir operating procedures in Texas; (2)

assessment of the water resources problems and needs in Texas; (3) investigation of the seasonal factors affecting reservoir operation; (4) review of state-of-the-art modeling capabilities; (5) development of a modeling strategy or method; and (6) case study. Based on the knowledge and experience gained in the above tasks, an assessment was made of (1) the potential for improving reservoir effectiveness through seasonal deviation in operating plans; and (2) generalized analysis capabilities for evaluating seasonal operating policies for various reservoirs.

This research was accomplished as a part of a project sponsored by the Texas Water Resources Institute entitled "Optimum Reservoir Operation for Flood Control and Conservation Purposes" (25). The thesis author began work as a graduate research assistant on this project in September, 1983. The overall project investigated reservoir operation in Texas in general and reservoir modeling capabilities in general and then focused on storage capacity reallocations between flood control and conservation purposes. Both long-term and seasonal storage reallocations were addressed. This thesis deals specifically with the portion of the overall project concerned with evaluating seasonal storage reallocation.

CHAPTER II

RESERVOIR OPERATION IN TEXAS

Water Resources Problems and Needs in Texas

Rapid population growth and economic development, coupled with a climate in which water resources are often scarce, have created potential water supply problems in many areas of the state. A recent planning report by the Texas Department of Water Resources (16) states that the population of the state is expected to increase to between 28.2 and 34.3 million by the year 2030, from a population of 14.2 million in 1980. Past, present, and projected population levels are shown in Figure 1.

This expected population rise, coupled with expanding economic activity, is expected to place increased demands upon the available water resources of the state. Serious water shortages are expected in many areas of the state in the coming years. Depleting groundwater reserves are resulting in an increased reliance on surface water. The rising cost of fossil fuel during the 1970's has focused attention on increasing hydroelectric power generation. Instream flow needs for fish and wildlife habitat and maintenance of fresh water inflows to bays and estuaries have received increased attention in recent years.

The two main sources of useable water in Texas today are groundwater and surface water supplies. Groundwater has historically been the primary water supply source for much of the state. According to the Texas Department of Water Resources (16), about 61 percent of the water used in 1980 was from groundwater, with the the remainder from surface water.

Extensive development of groundwater has resulted in several problems, some local in nature, while others are more widespread. About 89 percent of

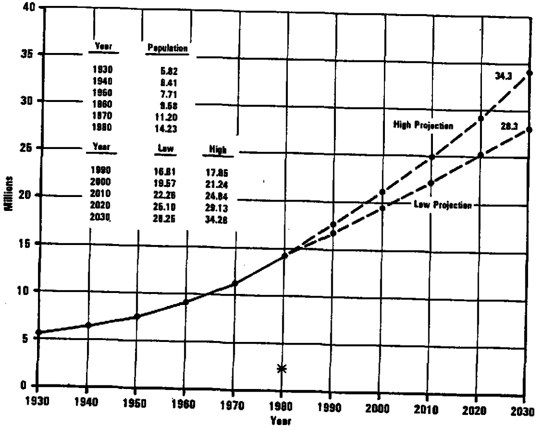


Figure 1.
TDWR Population Projections

the recoverable ground water is in the Ogallala Aquifer in the High Plains. For most of the aquifers, water withdrawal is occurring at a greater rate than recharge. In the Ogallala Aquifer of the Texas High Plains, the natural recharge is far less than the amount of pumpage. This is sometimes referred to as "*mining*" of groundwater. *Ground water mining is causing water-level declines,* decreased well yields, land subsidence, and saline water encroachment.

By the year 2000, if current water use trends continue, the state's aquifers are projected to be capable of supplying about 6.8 million acre-feet annually, or about 63 percent of the present level. Consequently, greatly increased demands will be placed upon surface water reservoirs.

Increased population and economic growth not only contribute significantly to the problems associated with groundwater overdraft, but also cause increased flooding problems. As economic activity and residential development expand into floodplain areas, watershed runoff also increases. Higher runoff leads to a greater need for flood control to reduce the associated higher flood damages.

In addition to ever-increasing water related needs, other factors affecting reservoir operation change over time as well. Watershed and floodplain conditions are dynamic. Construction of numerous small flood retarding dams by the Soil Conservation Service and other entities in the watersheds of major reservoirs have reduced flood inflows to the reservoirs. Construction of numerous small ponds for recreation or watering livestock has also decreased reservoir inflows and yields. Increased runoff caused by watershed urbanization is significantly contributing to flooding problems in certain locations. The existing flood control reservoirs were planned and designed based on the expectation of ever-increasing intensification of floodplain use. However, the

National Flood Insurance Program has resulted in zoning and regulation of 100-year floodplains. With stringent floodplain management, susceptibility to flooding could actually decrease over time as existing property owners choose to relocate and regulation prevents others from moving into the floodplain. Reservoir sedimentation reduces available storage capacity. Construction of additional reservoirs, as well as other related types of projects such as conveyance facilities, flood control levees and channel improvements, and electric power plants, affect the operation of existing reservoirs. Technological advancements in hydrologic data collection, streamflow forecasting, system modeling and analysis, and computer technology provide opportunities for refining operating policies.

Anticipated shortages, groundwater overdraft, and the need for more flood control have caused the state to shift to a greater reliance upon surface water reservoirs to meet its water-related needs. However, most of the feasible sites for major reservoirs in Texas have already been developed. Construction of new reservoirs is very expensive, and takes many years to complete. Therefore, there is a need to operate existing reservoirs as efficiently as possible.

Figure 2 shows decline in the use of groundwater since 1970 in comparison to the amount of surface water used. According to the Texas Department of Water Resources (16), this trend is expected to continue. Table 1 also lists these projections of water use.

However, groundwater overdraft is a serious problem, causing lower water tables and piezometric levels, increasing pumping costs, water quality problems, and damage from land subsidence. High chloride levels are a water supply problem in many Texas reservoirs today according to the Texas

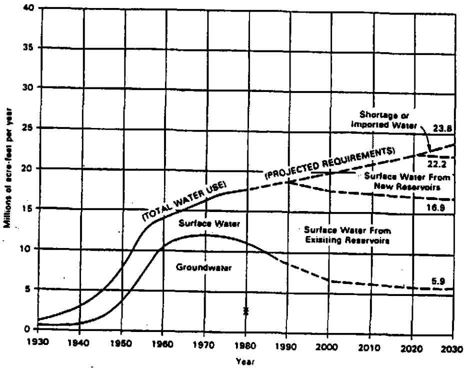


Figure 2.
Water Use and Source of Supply Projections

Table 1.

TDWR Water Use Projections

1980 Reported Water Use in Texas in acre-feet/year

municipal and domestic	2,813,000
manufacturing	1,520,000
mining	239,000
steam-electric	330,000
agricultural	12,851,000
total	17,853,000

Projected Water Use in acre-feet/year

Year 2000	Low	High
municipal and domestic	3,512,000	5,081,000
manufacturing	2,407,000	2,718,000
mining	268,000	268,000
steam-electric	717,000	817,000
agricultural	10,427,000	16,543,000
total	17,331,000	25,425,000

Year 2030	Low	High
municipal and domestic	5,059,000	8,178,000
manufacturing	4,231,000	5,014,000
mining	387,000	387,000
steam-electric	1,119,000	1,417,000
agriculture	11,385,000	15,351,000
total	22,181,000	30,347,000

Department of Water Resources (16). Some of these problems are naturally-occuring, some are man-made. Problems of naturally-occuring salinity are particularly severe in the upper reaches of the Red, Brazos, Colorado, and Pecos River Basins. These problems continue to plague development and full beneficial use of water resources in these basins.

Serious flooding conditions have at one time or another struck most portions of the state. Flooding results in loss of life and millions of dollars in damages annually. Flash flooding resulting from high intensity rainstorms is common and not easily predicted. Also, flat coastal areas are vulnerable both to high tides and to heavy runoff from rainfall associated with tropical storms.

Flood protection measures include flood control storage in reservoirs, channel modifications, levee works, and nonstructural floodplain management measures such as floodproofing, flood warning systems, and relocation. Despite the existence of these flood protection programs, damages from flooding will continue to increase along floodplains and in coastal areas if these areas continue to be selected for business and/or residential locations. Commonly, however, people do not perceive or consider the risk of flooding, and flood prone areas continue to be developed.

Basically, Texas has two contrasting conditions with respect to water-related issues. Both conditions coexist within the state. On one hand, flooding is causing much damage while on the other, water shortages abound. These problems are both aggravated by increasing population, expanding economic activity, decreasing amounts of available groundwater, and fewer potential feasible reservoir sites.

The traditional analysis methods and practices followed in planning and design of reservoir projects and during real-time operations have not really addressed the tradeoffs and interactions between flood control and conservation purposes. In general, expanded analysis capabilities are needed for periodically reevaluating the operating policies of existing reservoir systems. A particular need in this regard is improved methods for evaluating the tradeoffs involved in reallocating storage capacity between flood control and conservation purposes.

There is a need to develop a comprehensive, far-sighted approach or plan to meet future water resources problems due to flooding, lack of available water, and undesirable water quality within the state.

Improved reservoir operating plans would take advantage of the fact that existing reservoirs do not require extra funds for design and construction as would be needed if a new reservoir was built. Since most feasible reservoir sights have already been utilized, improving operation of these reservoirs would increase their associated benefits.

Many of the reservoirs across the state were designed years ago based on assumptions of future watershed conditions. Some of these assumptions were accurate and some were not. Improved operating policies could be used to better reflect these different watershed conditions. A less rigid form of operating policy would allow for gradual change in conditions affecting reservoir use. Changing conditions include, but are not limited to, flood plain development, water use patterns, reservoir sedimentation, and water quality.

Storage reallocation is one method of possibly operating reservoirs more effectively. It can be implemented for many of the existing reservoirs in the state, thus fulfilling the need to improve the operation of existing reservoirs before building new ones. Also, storage reallocation is flexible in concept, allowing reallocation volumes to change with respect to associated reservoir needs, constraints, and conditions.

Reservoirs in Texas

Surface water management in Texas is facilitated by 182 major reservoirs with storage capacities greater then 5,000 acre-feet (24). Five additional reservoir projects are presently under construction. The 187 major reservoirs contain conservation, flood control, and total capacities of 40.0 million, 18.5 million, and 58.5 million acre-feet, respectively. Streamflow in the state's 15 major river basins and eight coastal basins is highly variable. Consequently, the major reservoirs are essential for controlling and utilizing this surface water resource.

Institutional Framework for Reservoir Management

Reservoir development and management is accomplished within a complex system of organizations, laws, and traditions. The water management community consists of water users, floodplain occupants, tax payer, concerned citizens, public officials, environmental groups, special interest groups, universities, consulting firms, professional organizations, businesses, industries, utilities, and municipal, county, state, federal, and international agencies. Water policy is formulated and management decisions are made within a framework of legal and political systems. Water is a publicly-owned resource, and its allocation and use is governed by law.

Within this complex institutional framework, a number of entities are primarily responsible for the actual reservoir operations. The 187 major reservoirs in the state are owned, maintained, and operated by four federal agencies, 43 water districts and river authorities, 39 cities, two counties, a state agency, and 22 private companies (24). Table 2 shows the number of reservoirs and storage capacity owned by various types of entities.

The reservoir management agencies can be categorized as federal agencies, state and local governmental entities, and private companies. Most of the major reservoirs in Texas were constructed by state and local governmental agencies or private industry for conservation purposes. However, two-thirds of the total storage capacity is contained in reservoirs constructed by federal agencies. Most of the federal reservoirs are large multipurpose projects.

Federal agencies have constructed 38 major reservoirs and significantly modified two others. Four additional projects are presently under construction. The federal government is responsible for construction of eight of the ten largest and 21 of the 28 reservoirs with capacities exceeding 500,000 acre-feet. Eight federally-constructed projects have been turned over to non-federal entities for operation and maintenance. The others are operated by federal agencies. The 43 projects with federal involvement contain 52 percent, 99.9 percent, and 67 percent of the conservation, flood control, and total capacities, respectively, of the 187 major reservoirs. Federal involvemnt in reservoir construction and operation in Texas is summarized in Table 3.

The five projects constructed by the Bureau of Reclamation were turned over to local sponsors for maintenance and operation. The Bureau of Reclamation continues to own the projects until the local sponsor has completed payments to the federal government for reimburseable costs. The Soil Conservation Service (SCS) has also constructed two major water supply reservoirs which are owned, operated, and maintained by nonfederal sponsors. About 1900 flood retarding dams constructed by the SCS in Texas are not included in the data presented here because the controlled storage capacities

Table 2.

Types of Reservoir Owners

Type of Owner	Number of Reservoirs	Storage Capacity (acre-feet)		
		Conservation	Flood Control	Total
Federal Agencies	36	17,358,240	16,518,120	33,876,360
International Boundary and Water Commission	(2)	(5,772,600)	(2,654,000)	(8,426,600)
Corps of Engineers	(32)	(11,559,490)	(13,864,120)	(25,423,610)
Other	(2)	(26,150)	---	(26,150)
Water Districts and River Authorities	57	16,080,060	1,324,600	17,404,660
Jointly Owned by Cities and Water Districts or River Authorities	4	2,539,490	248,300	2,787,790
Cities	48	2,843,470	467,000	3,310,470
Counties	5	54,810	---	54,810
Other State Agencies	1	5,420	---	5,420
Private Companies	36	1,093,060	---	1,093,060
Totals	187	39,974,550	18,558,020	58,532,570

Table 3.

Federal Involvement in Reservoir Development and Management

Federal Involvement	Number of Reservoirs	Storage Capacity (acre-feet) Conservation	Flood Control	Total
Constructed, Owned, and Operated by International Boundary and Water Commission	2	5,772,600	2,654,000	8,426,600
Constructed, Owned, and Operated by Corps of Engineers	27	10,081,790	13,344,820	23,426,610
Presently Under Construction by Corps of Engineers	4	1,348,700	519,300	1,868,000
Major Modification by Corps of Engineers	2	448,600	248,300	696,900
Constructed by Bureau of Reclamation and Maintained and Operated by Nonfederal Sponsors	5	3,081,100	1,779,000	4,860,100
Constructed by Soil Conservation Service and Maintained and Operated by Nonfederal Sponsors	2	17,850	---	17,850
Constructed by Soil Conservation Service and Owned and Operated by U.S. Fish and Wildlife Service	1	18,150	---	18,150
Constructed, Owned, and Operated by Forest Service	1	8,000	---	8,000
Total	44	20,776,790	18,545,420	39,322,210

This data does not include federal grants and loans, such as those provided by the early Works Progress Administration (WPA) Program, which helped finance several of the nonfederal projects.

are less than 5,000 acre-feet for each dam. The Corps of Engineers operates and maintains its projects upon completion of construction. Withdrawals or releases form conservation storage are made at the discretion of the nonfederal sponsors. The International Falcon and Amistad Reservoirs on the Rio Grande River are owned and operated jointly by the United States and Mexico sections of the International Boundary and Water Commission. The Texas Department of Water Resources is responsible for administering the water allocation system and specifying releases from the United States share of the conservation storage.

Institutional arrangements for developing and managing reservoirs are based on project purposes. Practically all reservoirs in Texas containing controlled flood control storage were constructed and are operated by the federal agencies. Almost all the federal projects include flood control. The federal governmental has borne essentially all costs associated with flood control. The Corps of Engineers is responsible for flood control operations of its own reservoirs and those constructed by the Bureau of Reclamation. The International Boundary and Water Commission handles the flood control operations of its projects.

Municipal and industrial water supply has traditionally been a local responsibility, with the federal government confining itself to a secondary role. However, municipal and industrial water supply storage is included in all but two of the federal reservoirs in Texas, subject to nonfederal cost sharing.

The conservation storage in several of the federal reservoirs is used for irrigation as well as municipal and industrial water supply. However, the Bureau of Reclamation has not constructed large federally-subsidized reservoirs devoted primarily to irrigation in Texas like it has in several other western states.

In general, nonfederal sponsorship of conservation storage in federal reservoirs has been handled similarly for irrigation and municipal and industrial uses.

The Southwestern Power Administration (SWPA) markets the power from the three Corps of Engineers hydroelectric power projects. The Western Area Power Administration (WAPA) markets the power from the two International Boundary Commission projects. The SWPA and WAPA sell the power to electric cooperatives, municipalities, and utilities companies.

The federal projects all include public access and recreational facilities. Prior to 1965, recreation was included in federal projects as a fully federal expense. The Federal Water Recreation Act of 1965 established development of full recreational potential at federal projects as a full project purpose subject to nonfederal cost-sharing. Recreation contracts have been executed for two projects, which are both presently under construction, under the provisions of this act.

State and local governmental entities have constructed 108 major reservoirs and one other is presently under construction. These reservoirs contain 45 percent, 0.1 percent, and 31 percent, respectively, of the conservation, flood control, and total capacities of the 187 major reservoirs (24). This does not include the seven projects constructed by the Bureau of Reclamation and Soil Conservation Service which are operated and maintained by nonfederal sponsors. Nonfederal sponsors also control all the water supply storage in the Corps of Engineers reservoirs and are reimbursing all costs allocated to water supply.

River authorities own a number of the nonfederal reservoirs and have contracted for the conservation storage in many of the Corps of Engineers projects. River authorities are a special type of water district created to develop and manage water resources from a basinwide perspective. Some river basins in Texas are served by a single river authority while other basins are served by several authorities. The Brazos River Authority, created in 1929, was the first authority ever set up in the United States to administer the waters of a major river. Thus, Texas created its first river authority four years before the creation of the Tennessee Valley Authority by the federal government. The 19 river authorities finance their activities primarily through operation and service fees.

Private companies constructed, own, and operate 36 major reservoirs containing no flood control and less than three percent of the total conservation storage of the major reservoirs. Most of these projects were constructed by electric companies to provide cooling water for steam-electric generating plants.

Flood Control Versus Conservation Purposes

Reservoir operation is based on the conflicting objectives of maximizing the amount of water available for conservation purposes and maximizing the amount of empty space available for storing flood waters to reduce downstream damages. Each of the major reservoirs in Texas is operated for only conservation purposes or only flood control or a certain reservoir volume, or pool, is designated for conservation purposes and a separate pool for flood control. The pools are separated by a designated top of conservation pool elevation. Institutional arrangements for constructing and operating reservoirs are based on having separate pools for flood control and conservation. Planning, design, and operational problems associated with flood control are handled separately from those associated with conservation.

Construction of a conservation reservoir can actually worsen downstream flood conditions due to loss of valley storage, decrease in flood wave attenuation, and increase in travel time. However, conservation capacity provides some incidental flood protection whenever the flood event coincides with a partially drawn-down pool. Drought periods in Texas have often been ended by a major flood event such that empty conservation storage space was available to store the flood waters. Surcharge storage in conservation only reservoirs may also provide some incidental flood protection. Likewise, temporary storage of flood water in flood control pools may provide some incidental benefits for conservation purposes, particularly hydroelectric power generation. However, reservoir operation throughout the state is based on treating flood control and conservation capacities as distinctly separate pools serving different purposes.

Conservation Operations

All of the major reservoirs in Texas except three contain conservation storage capacity (24). The primary conservation purposes served are municipal, industrial, and agricultural (irrigation) water supply, cooling water for steam-electric plants, hydroelectric power generation, and recreation. Reservoir operation involves both complementary and conflicting or competitive interactions between these purposes. Numerous municipal, industrial, and irrigation users are dependent upon the limited resource water. Allocation between competing users is governed by the water law of the state. Hydroelectric power can often be generated with water that is released for downstream municipal, industrial, and agricultural uses. In other cases, water may be released specifically and only for hydroelectric power generation. Reservoir recreation is extremely popular and a major consideration in reservoir operation in Texas. Recreation is generally complementary with other conservation purposes. However, since operation for recreation essentially means maintaining a full pool without fluctuations in water surface level, releases and withdrawals for other purposes can be detrimental to recreation uses.

Municipal and/or industrial water supply is provided by 163 of the 184 conservation reservoirs. Irrigation is a designated purpose of many of the major reservoirs. Most of the reservoirs providing water for irrigation also supply municipal and industrial uses. About half of the irrigation from surface water occurs in the lower Rio Grande Valley using water regulated by International Falcon and Amistad Reservoirs.

Water supply withdrawals are made at many projects through pumping plants with intake structures located in the reservoir. In many other cases, releases are made through outlet works and spillway structures to be withdrawn from the river at downstream diversion facilities. The water may be actually withdrawn at locations several hundred river miles below the dam from which it was released. Although most of the surface water used in the state is used within the river basin from which it originates, significant interbasin transfers do occur, particularly to the coastal basins. A majority of the water supply reservoirs are operated as individual units to supply specific customers. However, a number of reservoirs are operated as systems with some degree of interaction between the component reservoirs. Systems operation typically involves maintaining a balance between storage depletions and water surface fluctuations in the component reservoirs. Hydroelectric power generation is also a concern in system operation. Releases are coordinated to meet water supply demands while minimizing the amount of water bypassing the turbines.

Reservoir operation procedures for water supply purposes are based essentially on meeting water demands subject to institutional constraints related to water rights, project ownership, and contractual agreements. The complex organizational framework for water supply operations involves a multitude of water users and suppliers working under various contractual arrangements. Water suppliers may either own and operate reservoirs or contract with other reservoir owners for storage capacity or water use. A number of entities both own and operate their own reservoirs and contract for the use of additional capacity.

Water use permits are administered by the Texas Department of Water Resources in accordance with the water law of the state. Permits may involve a variety of arrangements. Permits may be regular, seasonal or temporary, or emergency in nature. Special provisions may be made for special circumstances. The legal right to use or sell the water from a reservoir is usually granted to the owner prior to construction of the project. Many reservoirs are owned and operated by cities to provide water to their citizens for domestic, commercial, and public use. The city holds the permit or water right and sells the water to its citizen customers. Another common case is a reservoir or system of several reservoirs owned and operated by a river authority which sells the water to a number of cities, industries, and/or farmers. The river authority operates the reservoirs to meet its contractual obligations to its customers. The cities, industries, and farmers purchase the water from the river authority without having to obtain a water right permit through the TDWR. The river authority operates the reservoirs to meet its contractual obligations to its customers. The federal government does not get involved with water rights. The nonfederal project sponsors which contract for the conservation storage in federal projects are responsible for obtaining the appropriate water rights permits through the TDWR.

Individual farmers, industries, and cities also hold water rights permits not associated with reservoirs. In several of the river basins, a number of reservoir operators, all holding appropriate water rights permits, operate reservoirs in the same basin. Reservoir operators are often required to make releases, typically not exceeding inflows, to allow downstream users not associated with the reservoir access to the water for which they are legally entitled.

In 1980, total flow through the turbines of the state's 21 hydroelectric power plants exceeded 11 million acre-feet (16). Although large volumes of water are used for hydroelectric power generation, the water is not consumed and is usually used downstream for other purposes after passing through the turbines. At several of the hydroelectric plants, reservoir water diverted through the turbines is strictly limited to releases being made for other water supply or flood control purposes. At some projects, hydropower releases may be in excess of those needed for other purposes, but the multiple purposes are still closely coordinated. Several of the hydroelectric plants are located downstream of other plants such that the same water flows through two or more turbines. Hydroelectric power accounts for about 0.6 percent of the electrical power produced in Texas, with most of the electricity being produced by natural gas, and to lesser extent coal-fired thermal electric plants. Hydroelectric power is used primarily for peak loads.

Instream flow needs included maintenance of sufficient streamflow for water quality, fish and wildlife habitat, livestock water, river recreation, and aesthetics. Water law and reservoir operation practices have traditionally favored offstream needs over instream needs. Releases for hydroelectric power and also water supply releases which are withdrawn from the river for municipal, industrial, or agricultural use at significant distances below the dam contribute to instream environmental needs as well. Operating procedures for some reservoirs include providing minimum instream flow levels for maintenance of fish and wildlife habitats. Some reservoirs have multi-level outlet works which allow selective blending of discharge waters for optimal downstream water quality. The role of reservoirs in contributing toward the maintenance of desirable levels of freshwater inflows to the state's bays and estuaries has recently received considerable attention and will likely continue to be scrutinized in the future.

Flood Control Operations

Whereas conservation operations throughout the state are the responsibility of a multitude of entities, the responsibility for flood control operations is highly centralized. The International Boundary and Water Commission is responsible for flood control operations of Falcon and Amistad Reservoirs on the Rio Grande River. These two multiple purpose projects contain 2.7 million acre-feet of flood control storage. The 12,600 acre-foot Olmos Reservoir is a flood control only project owned by the City of San Antonio. It is the smallest, oldest, and only nonfederal project of the major reservoirs containing flood control storage. The U.S. Army Corps of Engineers is responsible for the 15.9 million acre-feet of flood control capacity in the remaining 32 flood control reservoirs.

The discussion here is limited to controlled storage. Releases are controlled by the operator through the use of spillway and outlet works gates. All of the large flood control reservoirs have gated spillways and/or outlet works.

Numerous other small uncontrolled flood retarding and detention basin structures are in use throughout the state. The ungated outlet structures are designed with limited discharge capacities which result in outflow rates being less than inflow and storage occuring during a flood event. Streamflows are automatically reduced without requiring release decisions to be made by an operator. These small uncontrolled flood control reservoirs are not addressed in this report.

The Fort Worth District (FWD) of the Corps of Engineers is responsible for about 58 percent of the flood control storage capacity of the major reservoirs in the state. Fort Worth, Tulsa, and Galveston Districts operate a total of about 86 percent of the flood control storage capacity.

A reservoir control center in the Southwestern Division office in Dallas provides overall management and coordination of reservoir operation activities in the several districts of the division. The district offices are responsible for the actual operation of the reservoirs. Each district organization includes an operations division responsible for operation and maintenance of completed projects. However, real-time reservoir regulation and associated water control activities are the responsibility of a reservoir control unit which is a part of the hydraulics and hydrology branch of the engineering division. Thus, a central reservoir control organization within the district office is responsible for determining the releases to be made at all of the reservoirs within the district. Reservoir managers and supporting personnel at the individual reservoir projects operate the spillway and outlet works gates as instructed by the district office. Telecommunications between the the reservoir control and the reservoir project offices occur at least daily and can be essentially continuous during major flood events. Emergency operating procedures are established for each project as a contingency in case communications should be disrupted during a flood.

The projects have two general types of outlet structure configurations. A number of the projects have an uncontrolled broadcrested or ogee spillway, with the crest elevation at the top of flood control pool, combined with an outlet works structure consisting of a gated intake structure, conduit with an outlet works structure, conduit through the dam, and downstream stilling basin. The gates are located at various depths below the top of conservation pool. Other projects have a controlled spillway with a set of several tainter gates. Tainter gates (also called radial gates) rest upon the spillway crest when fully closed. A gate is opened by lifting, with water flowing under the gate and over the spillway crest. Controlled releases from the flood control pool are made by raising the tainter gates. Sluices with gates at lower elevations are also provided for relatively small releases.

Flood control and conservation pools in a multiple purpose reservoir are designated by set pool elevations. The top of conservation pool (bottom of flood control), top of flood control pool, and maximum design water surface are key pool levels or elevations in flood regulation schedules. Releases are made from the conservation pool for water supply purposes at the request of the local sponsors which have contracted for the storage. The flood control pool is the space between the top of conservation pool and the top of flood control pool. Releases from the flood control pool are regulated by opening and closing spillway and/or outlet works gates. Surcharge storage occurs whenever the flood control is full and inflows exceed discharges through the spillway. The maximum design water surface is the critical condition for which the dam and appurtenant structures were designed. Consequently, release policies are predicated on never under any circumstances allowing surcharge storage to exceed the design water surface.

Reservoirs designed and constructed by the Corps of Engineers are normally sized to contain a flood with an associated recurrence interval of 50 to 100 years, or in some cases greater, without exceeding the capacity of the flood control pool. Consequently, filling to the top of flood control pool is an infrequent event. Many of the projects have never had the flood control pool completely full. This is not necessarily the case for multipurpose projects constructed by others for which the Corps of Engineers is not responsible for flood control operations.

Operating Procedures

Current operating policy of multipurpose Texas reservoirs is based on the division of available storage space. This space is conceptually divided into two basic zones. These zones are for flood control and conservation-related purposes. Reservoir operation throughout the state is based on treating flood control and conservation capacities as distinctly separate pools serving different purposes.

Portions of each of the flood control and conservation zones are set aside for sediment deposition. Sediment storage space is set aside for the deposition of sediment that is carried into the reservoir by inflowing streams. The sediment is in suspension or bed load while it is carried by these streams. When these streams enter the reservoir, their velocity greatly decreases and their sediment loads are then deposited in the reservoir. Typically the total volume for storing sediment is the estimated amount of sediment that will be deposited in the reservoir over the next fifty to one hundred years of reservoir operation. Inverts of water conveyance structures for flood control and/or conservation purposes are usually set at the maximum height of deposited sediment that is expected over the period of reservoir use. During the initial years after construction of the reservoir, the top of this material will be below the intake invert, but should rise with the years as inflowing streams release their loads of silt and sand into the reservoir.

The operating policies of Texas reservoirs do not directly deal with sediment storage. This space is expected to eventually fill with sediment and therefore be of no use for either conservation or flood control purposes. However, a secondary benefit available to some reservoirs is the additional hydroelectric power production which is possible because of the increased head associated with extra volume from sediment storage.

The operation of the conservation storage portion is relatively simple and is usually operated on an on-demand basis. The agency with authority over the conservation storage simply requests a withdrawal or release of water when they have need of the water. The operation of the flood control portion of the reservoir is more complex. The basic idea behind regulation of multipurpose reservoirs with flood control storage is to keep the flood control storage space empty until it is needed to contain a flood that would otherwise cause damaging flows downstream of the reservoir.

Regulation of the water in the flood control pool is, in a sense, direct and indirect. Basically, indirect regulation takes place without human involvement. Indirect regulation of this water is accomplished by an uncontrolled spillway. An uncontrolled spillway is a cut-out or slot in the dam with its lowest entrance elevation equal to or slightly above the elevation of the top of the conservation pool (bottom of the flood control pool). Any time inflows to the reservoir cause the water level to rise above the bottom of the spillway entrance, all water in the reservoir above that level goes over the spillway. The uncontrolled outlet structures are designed with limited discharge capacities which result in outflow rates being less than inflow occuring during a flood event. Streamflows are automatically reduced without requiring release decisions to be made by an operator. This water is released as quickly as the opening will permit. In this case, the only restriction to the rate of flow is the size of the opening. Water will continue to flow across the spillway until the reservoir level has receded to the bottom elevation of the entrance. This is why reservoirs with uncontrolled spillways do not always make best use of available storage space. Uncontrolled spillways do not have much capability of holding back incoming water in the case that flooding is going on in areas directly downstream of the reservoir. Empty storage space in the reservoir above the top of the conservation pool is not utilized to help reduce flooding as effectively as it could be if the reservoir had a controlled spillway. While it is true that an uncontrolled spillway will hold back flood waters if reservoir inflow is in excess of spillway discharge capacity, it has no capability of varying the discharge rate or even halting discharge dependent upon reservoir and/or downstream flooding conditions as would be possible with direct regulation.

However, it may not be feasible to use controlled spillways for flood control on some of the medium to smaller reservoirs across the state. This is because the cost of buying, installing, maintaining, and operating a controlled spillway may exceed the benefits that would be obtained on these reservoirs if the spillways were controlled instead of uncontrolled.

On the other hand, direct regulation of reservoirs with water in the flood control pool requires human involvement. Direct regulation involves closing or opening gates, valves, or other openings in the dam.

Although uncontrolled spillways do not directly regulate water in the flood control pool, some reservoirs with uncontrolled spillways still have the ability to directly regulate the water in the reservoir. This is done by use of gated conduits or sluices through their dams. These conveyance structures allow controlled releases of water for either flood control or conservation purposes. These conduits can be used to lower the reservoir level in the case that a rainfall event is predicted to occur which will cause the flood control pool to fill, or be exceeded, and cause serious downstream flooding. These conduits are also used to lower the reservoir level down to the top of the conservation pool in the case that a rainfall event has caused the reservoir to rise into the flood control pool but not above the bottom of the spillway. However, they are intended mainly for conservation releases since their discharge capacity is usually much less than that of a spillway.

Basically, direct regulation of a single reservoir consists of passing all flows through the reservoir and dam (after the desired conservation storage is obtained, in a multipurpose reservoir) up to the value of the downstream channel capacity. (The channel capacity is the amount of water that will flow in the stream without causing flooding. It is based on channel slope, roughness, and hydraulic radius). When inflow to the reservoir causes downstream flooding, releases are reduced and all inflows in excess of the desired releases are stored in the flood control pool. When downstream flooding subsides, releases are increased to empty the flood control pool as quickly as possible (without causing flooding again) so that the flood control space will be available to capture the next potential flood-causing storm.

This highly simplified rule becomes much more complicated when uncontrolled areas downstream of the dam (which contribute to the total downstream flow) cause downstream flooding when added to releases made from the reservoir. It also becomes more complicated when inflows to the reservoir are expected to exceed the capacity of the combined sediment, conservation, and flood control storage of the reservoir when the downstream channel is at or near capacity. This is when the more complicated operating rules come into effect.

The flood control regulation schedule for a reservoir with direct regulation actually consists of two schedules. Both schedules are followed, and the one requiring the largest release rate controls for a given set of conditions. The regular schedule, which usually controls, is based on the assumption that ample storage capacity is available to handle the flood without special precautions being necessary to prevent the water surface from rising above the top of flood control pool. Operation is switched over to an alternative schedule during extreme flooding conditions when the anticipated runoff from a storm is predicted to exceed the controlled capacity remaining in the reservoir. If the water surface level significantly exceeds the top of flood control pool, downstream damage will necessarily occur. The objective is to assure that reservoir releases do not contribute to downstream damages as long as the storage capacity is not exceeded. However, for extreme flood events which would exceed the reservoir storage capacity, moderately high damaging discharge rates beginning before the flood control pool is full are considered preferable to waiting until a full reservoir necessitates much higher release rates.

Release decisions are based on a current reservoir water surface elevation and inflow. The required outflow for a given reservoir elevation and inflow is read from the graph. If this outflow is less than that specified by the regular schedule, the regular schedule is followed. Otherwise, the gates are operated to release the outflow indicated by the graph.

Filling a flood control reservoir to the top of the flood control pool is an infrequent event. Many of the projects have never had the flood control pool completely full. This in no way diminishes the great importance of flood control, but the majority of the year reservoir operations will be the opening of conduits to allow for conservation releases. An even smaller part of the time will the reservoir be operated by inducing surcharge storage. (Induced surcharge will be further explained in coming pages). This is because reservoirs with flood control operations are designed such that the probable maximum flood (PMF) not top the dam crest. This gives storage in the designated flood control space capable of storing a flood of great magnitude, such as the 100-year flood. Therefore, the reservoir level should not rise above the top of the flood control pool more than once or twice in the entire life of the project, assuming all calculations and assumptions involved in the design of the reservoir are correct. Accordingly, the reservoir should not rise very high in flood control pool very often either. These last two statements also assume correct operation of the gates, conduits, and any other outlet works according to given reservoir regulation rules (19).

Although only a small portion of the time during the life of the reservoir will flood control operation be used, the bulk of the operating rules deal mainly with these periods of time in which the water level has risen above the top of the conservation pool into the flood control pool.

Direct regulation, in the case of larger reservoirs, makes better use of available storage space above the conservation pool by utilizing this space to store water in the event that flooding is already going on downstream of the reservoir.

Until the 1940's and 1950's, it was common practice on gated reservoirs not to allow the water level to exceed the top of the static full pool level at the dam until all gates were opened. Then the outflow would be uncontrolled as long as the inflow to the reservoir was in excess of the spillway capacity at static-full-pool elevation. However, due to reasons presented previously, it was found that significant flood damages could arise because of the fact that reservoir releases under this plan may be larger than during times when the reservoir is at or near full capacity, than would have been the case under pre-reservoir conditions. Therefore, reservoirs with gated spillways attempt to meet 2 goals: (1) When reservoirs are at or near capacity, peak reservoir discharge rates should not exceed flow rates at downstream points that occured under the same watershed and precipitation conditions before reservoir construction, and (2) the rate of reservoir releases during a significant increment of time should be limited to values that would not constitute a major flooding threat to downstream interests (18).

There are two basic ways to meet these two goals. One method is to forecast the amount of storm runoff due to a given storm. If this runoff, when added to current reservoir contents, is expected to exceed the total capacity, then releases are begun prior to the reservoir filling. These releases are scheduled to limit outflow to a level not in excess of downstream channel capacity, or some other appropriate value.

The other method, an alternative to the one mentioned previously, makes use of what is called "induced surcharge storage". Induced surcharge is water in storage that is above the static full pool level. Surcharge storage occurs whenever the flood control pool is full and inflows exceed discharges through the spillway. After the reservoir has filled to the static full pool level, this surcharge storage is used to partially control reservoir discharge. Figures 3 and 4 show examples of two different regulation schedules based on reservoir inflow and rate-of-rise of reservoir elevation, respectively.

This is done by raising the gates a little at a time, forcing all incoming flow (in excess of the spillway outflow at that gate opening) into surcharge or storage above the static full pool elevation. This shows the desirability of a gated spillway, as previously alluded to in this text. The maximum desirable rise of this extra storage is usually no more than four to eight feet.

There can be induced surcharge of water only when the depth of water at the spillway gates is in excess of the top of the flood control pool. Water in induced surcharge is that water which is above the top of the flood control pool (which is at the top of the gates in their lowest position). This is water which has backed up behind the gates when inflow to the reservoir is in excess of the outflow that corresponds to different gate openings.

The induced surcharge envelope curve represents the highest water surface level that would be allowed at different spillway release rates when operating under the induced surcharge plan.

Conversely, in many other reservoir projects, the stated goals or objectives cannot be fully met within the range of available induced storage operation, and a part of the available storage below the static full pool level must be used. To develop a schedule to meet these requirements under operating

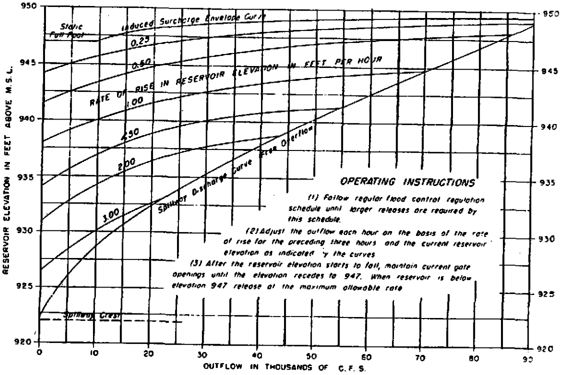


Figure 3.

Example Reservoir Regulation, Schedule A

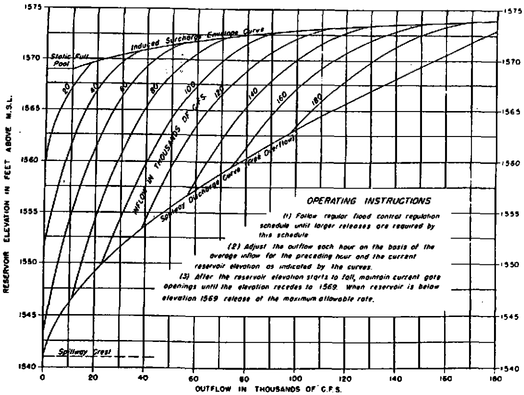


Figure 4.

Example Reservoir Regulation, Schedule B

conditions, the runoff volume of the flood must be predicted before reservoir discharges can be scheduled. This would be to allow release of water in the conservation pool so that more empty space would be available for storing the anticipated flood.

Even though this would probably give very effective operation, a more conservative approach is warranted when designing release schedules for damtenders (operators) under emergency conditions. Figure 5 is the emergency operating schedule for Waco Lake.

The resulting schedules can vary somewhat during critical floods when supplemented by other available information. This conservative approach is based on the fact that during critical floods, communications may cease. The only information then available to the damtender might be the water levels at the dam and the time it took to change to that level from the previous level (rate of rise).

These two pieces of information are then used in a procedure to estimate the storm runoff volume to aid in scheduling releases during critical flood periods.

This inflow volume is obtained by assuming that the inflow hydrograph has crested and computing the volume under the recession side of the hydrograph (18). By this assumption, reservoir operators are saying that the releases they make (or don't make) at that time period are based on a peaking inflow volume. These release decisions are updated each time period. If the previous assumptions are not correct, then the release decisions are updated. The volume is not computed under the rising limb of the inflow hydrograph because it assumed that this volume is already in the reservoir. The receding limb is considered to be a little steeper than the actual hydrograph to give a

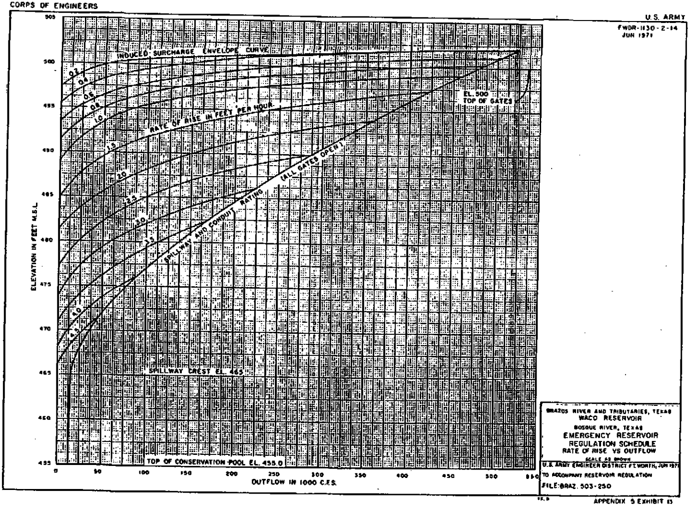


Figure 5.

Waco Lake Emergency Regulation Schedule

conservative volume estimate. This is a conservative estimate because it will give a "minimum" volume estimate. A minimum estimate assures that all releases made from a reservoir are necessary. For example, reservoir operations personnel do not want to make releases when there is flooding in progress downstream of the dam. However, if their minimum inflow volume estimate combined with present reservoir storage levels shows that a dangerously high water level will result, then they can justify making releases even though flooding is going on downstream.

After having computed this inflow volume, the outflow required to limit storage to that of available capacity can be determined. Using various combinations of assumed inflow volumes and available storage, a complete schedule can be developed before a flooding situation occurs that will allow outflow to be adjusted based on the actual values of inflow and remaining storage.

Portions of the operator's manual from Waco Lake in McLennan County will be used for examples of current operating policies in Texas. Waco Lake is a flood control and conservation reservoir in McLennan County constructed by the Corps of Engineers in 1965. It is situated on the Bosque River five miles upstream of the City of Waco. The reservoir is used to control flooding on the Brazos River (of which the Bosque is a tributary) and for storage of water for conservation purposes by the City of Waco and the Brazos River Authority.

Waco Lake was chosen for use in the case study portion of this thesis because of its central location with respect to the rest of the state. It was also chosen because much of the conditions surrounding the reservoir are representative of other reservoirs across the state. It was hoped that the results of the case study would be helpful in applying seasonal control to other reservoirs. Much of the information necessary to carry out the case study was available from the Corps of Engineers, City of Waco, and the United States Geological Survey.

The conservation storage in Waco Lake is below 455 feet elevation. The Brazos River Authority has contracted with the Corps to purchase usage rights to the water in conservation storage between elevations 427 and 455 feet elevation. The operating rules for Waco Lake state that water should be released from this storage upon request from the Brazos River Authority.

To keep flowrates downstream of Waco Lake from exceeding those of like precipitation conditions before the reservoir was put into use, forecasts are computed by use of API (Anticipated Precipitation Index, based on soil moisture and average daily precipitation), week of year, storm duration, and average precipitation parameters. Figure 6 shows the computation of runoff to Waco Lake using these parameters.

This gives a value for runoff in inches which is applied to the unit hydrograph of the Bosque River Basin of appropriate duration. This is done to forecast the inflow hydrograph to the reservoir. When this inflow is expected to cause available flood storage space to be exceeded, action can be taken to avoid this. These actions are included in the reservoir regulation manual for Waco Lake to be listed later.

Flood control operations, as can be readily seen, are much more complicated than just releasing water on demand. When the water level on Waco Lake is rising and inflow forecasts predict the level to rise above the top of the conservation storage but not above the top of the flood control pool, the total releases (conservation requests and flood control) must be less than the downstream capacity of 50,000 cfs on the Bosque River near Waco or the

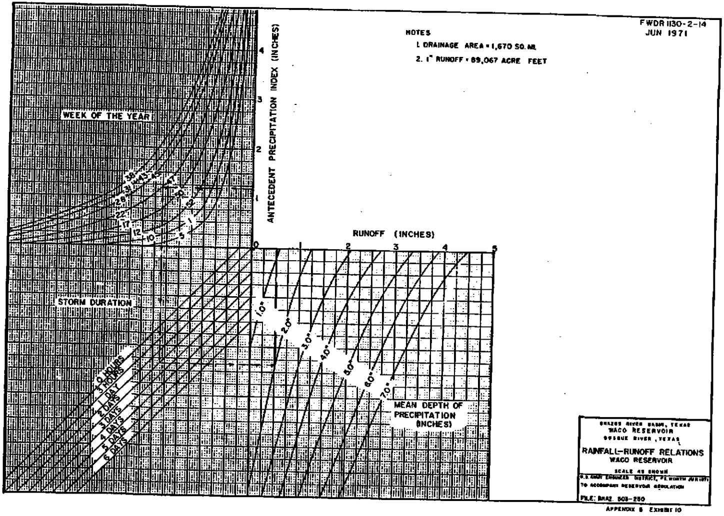


Figure 6.

Waco Lake Rainfall Runoff Relation

capacity at Richmond on the Brazos, which is 60,000 cfs. Therefore the total releases under these conditions cannot exceed 50,000 cfs at Waco or 60,000 when routed to Richmond. Releases will probably be less due to areas *downstream of the dam which usually contribute to the total streamflow at* Richmond and therefore decrease the amount of water which can be released. However, Waco Lake is used in conjunction with Lakes Belton and Whitney for flood control on the Brazos River. Therefore, if the percentage of water in flood control in Waco Lake is less than that being utilized in the other two reservoirs, no releases will be made (19).

The second condition of flood control operations is when the level is at the top of the conservation pool (455). This rule is the same as the previous condition, except that releases are made to maintain the reservoir at the top of the conservation pool.

Condition III is when inflow forecasts predict the level to exceed the top of the flood control pool at 500 feet elevation. Releases are to be less than 50,000 cfs on the Bosque until the release indicated by Exhibit 12 is larger than the *release that is necessary to keep a flow of 50,000 cfs maintained. Then, the* reservoir will be operated with Exhibit 12 to induce surcharge. Exhibit 12 is shown in Figure 7.

Condition IV is when the reservoir level is falling but is still above the top of the flood control pool (500). The reservoir is still operated in accordance with Figure 7 (Exhibit 12) until the water level recedes to an elevation of 500 feet. All inflow to the reservoir is discharged unless it is lower than the amount of releases required to maintain a downstream flow of 50,000 cfs at Waco. If not, releases are still made from the reservoir at 30 to 50,000 cfs to lower the reservoir level as quickly as possible without exceeding downstream capacity.

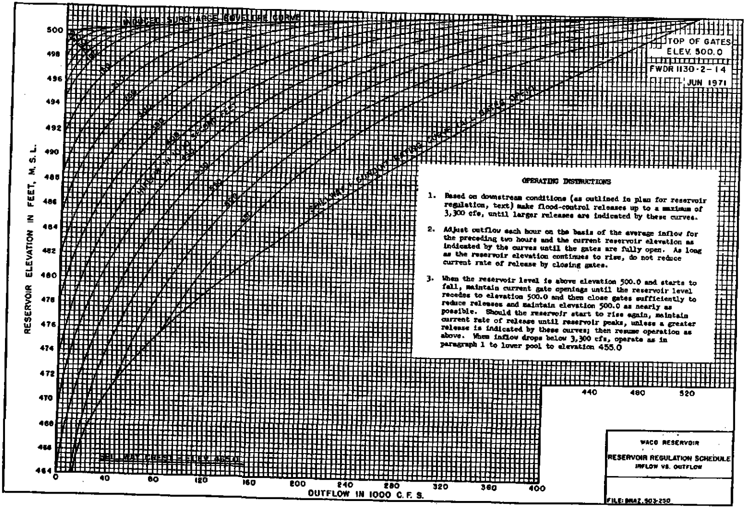


Figure 7.
Waco Lake Reservoir Regulation Schedule

Condition V is when the reservoir level is falling and between 500 and 455. Releases are made to maintain flows on the Bosque near Waco between 30 and 50,000 cfs but not in excess of 60,000 on the Brazos at Waco or Richmond.

Condition VI is when the pool elevation is falling and is between elevations 500 and 455. Controlled releases are in progress but rainfall of one inch or more has occured at the damsite in 24 hours or less. The operation for condition VI requires the operator to make a forecast to obtain the runoff downstream and open the gates such that the flow near Waco is between 30,000 and 50,000 cfs. If the forecast shows that this rate will be exceeded (due to dam releases and uncontrolled flow below the dam) the gates will be closed until the flow near Waco begins to recede. Then the gates will be adjusted again to keep the flow between 30 and 50,000 cfs at Waco on the Bosque and not to exceed 60,000 on the Brazos at Waco and Richmond.

The regulation manual has a second schedule (Exhibit 13) which is for use in emergencies. As previously defined, this is for use only when communications fail. The operator cannot receive instructions from headquarters nor can he receive all the needed information from stream gages or precipitation stations concerning the conditions of the watershed.

This emergency schedule is the same as the regular schedule concerning releases made from conservation storage. They are made any time the Brazos River Authority requests water.

For flood control operations, if the reservoir level is above 455 but below 500 and is rising, standing, or falling, then the following regulation rules are obeyed: If flood control releases are going on when communications cease, the gates will be closed as soon as one of the following conditions occurs: (1) One or more inches of rain is received at or below the dam in 6 hours or less; or (2) 6 hours of time elapses after communications cease gates will be closed until the communications are restored unless the rate of rise of the reservoir (when applied to Exhibit 13) are required. The reservoir is then operated according to Exhibit 13 (Figure 5).

When the level is standing or falling at or above an elevation of 500 feet, the reservoir is to be operated according to Exhibit 13.

Under no circumstances will the total releases be allowed to go below the amount required for conservation purposes.

Operating policies for other Texas reservoirs are similar to that of Waco Lake if they too are multipurpose in nature. Although 147 of the 189 reservoir projects in the state contain about half of the state's conservation storage capacity, they contain essentially no flood control storage capacity, and therefore have no operating rules pertaining to flood control. The sole use of these reservoirs is for water supply, hydroelectric power generation, and other related conservation purposes.

Seasonal Rule Curve Operation in Texas

Rule curve operation is a basic concept used in controlling the amount of water which is stored in a reservoir and how much is released, according to the time of the year. Generally a rule curve is graphically presented as a plot of allowable storage or elevation in a reservoir versus time of year for one or all of the designated pools within the reservoir.

Although seasonal rule curves are fairly common in other parts of the United States, this type of operating policy has not been widely adopted in Texas. The top of conservation pool has been varied seasonally for only four reservoirs in the state. In other words, the elevation delineating the conservation storage pool remains constant all year for all but four of the major multipurpose reservoirs in the state with flood control storage space.

The four multipurpose reservoirs in the state which contain conservation storage as well as flood control and are seasonally operated are Lake-o-the-Pines, Wright Patman, Falcon, and Amistad. The first two projects are both in the Texas portion of the Red River basin. Control over these reservoirs was transferred from the New Orleans District of the Corps of Engineers to the Fort Worth District in 1979. These are the only two reservoirs in Texas operated seasonally by the Fort Worth District. Seasonal control was implemented for these two reservoirs while still under the control of the New Orleans district.

The operating rule curve for Lake O' the Pines provides for raising the top of conservation pool 1.5 feet from mid-May through September for recreation purposes. The rule curve for Wright Patman varies significantly during the year in response to an interim agreement with the conservation storage sponsor to provide additional municipal and industrial water supply. The top of conservation pool is constant from November through March, and varies with date from April through October. The top of conservation pool peaks on June 1 at a level 6.9 feet above the winter pool level. A permanent reallocation of flood control to conservation is planned for Wright Patman Reservoir upon completion of Cooper Reservoir upstream. The seasonal rule curve is being followed until that time.

The top of conservation pool elevations for Falcon and Amistad Reservoirs on the Rio Grande can be, at the discretion of the International Boundary and Water Commission, temporarily raised for seasonal rule curve operation. However, the optional encroachment into the flood control pool does not necessarily occur routinely each year and the magnitude can be varied within a fixed maximum limit.

CHAPTER III

SEASONAL FACTORS AFFECTING RESERVOIR OPERATION

Maximizing the beneficial use of multipurpose reservoirs in Texas is tied closely to four seasonally varying factors. These factors include water availability, water demand, flood threat, and flood damage potential, all of which are related to the highly variable Texas weather. These factors are very important to the operation of reservoirs across the state and are therefore presented and discussed in the following chapter.

Water Availability

The source of water in each area of the state is precipitation, although everyday current supplies are obtained from storage in aquifers, reservoirs, and flowing streams. In Texas, the particular climate and physiography combine to affect the distribution of precipitation across the state. Also, certain characteristics of the climate-temperature, drought, hurricanes, and other weather phenomena-affect the quantity of precipitation that occurs in different regions of the state.

The physiography (physical geography) of Texas affects the variation and distribution of precipitation. Areas of the state with higher elevations have a cooler, drier climate than others areas of the state. These areas are not as affected by the general circulation of moist Gulf air that is characteristic for the lower easternmost portions of the state. Because the Gulf of Mexico is a major source of moisture for precipitation across the state, rainfall gradually decreases with greater distance westward from the Gulf. Generally, rainfall decreases from east to west across Texas at a rate of about one inch every fifteen miles. Figure 8 is provided to show how the average annual precipitation varies from east to west across the state.

Variations in precipitation and temperature are determined primarily by the confluence of warm, moist Gulf air and relatively cool, dry air from the continental United States. The western half of the state has a semi-arid, continental-type climate characterized by rapid and drastic fluctuations in temperature. The remainder of the state is influenced by a humid, subtropical climate having moderate temperatures.

Temporal variation in average annual rainfall is also a feature of the Texas climate. The wettest year, according the Texas Department of Water Resources (16), occured with a statewide average of more than 42 inches of rain. The driest of record was 1917, with only a 14 inch statewide average. Although an integral part of the climate, these variations are difficult to predict. Table 4 shows the average, maximum, and minimum recorded precipitation readings across the state. For a given location, the average inflow itself varies with time of year. This is demonstrated by the following graph, Figure 9, which is a plot of average inflow versus month of year for Waco Lake.

Average inflow during April and May is above the yearly average, with inflow below the yearly average the remainder of the year. This graph implies that, on the average, more water is available for use in April and May, and less is available in July and August. The low inflows of the summer months are followed by a relative increase during September and October. This rise corresponds to increased flood threat during these first months of fall, as is pointed out in the section on flood threat. The same observation holds true for April and May also. Low winter flows are followed by increased inflows of spring. (Flood threat is also higher in the spring than in the winter).

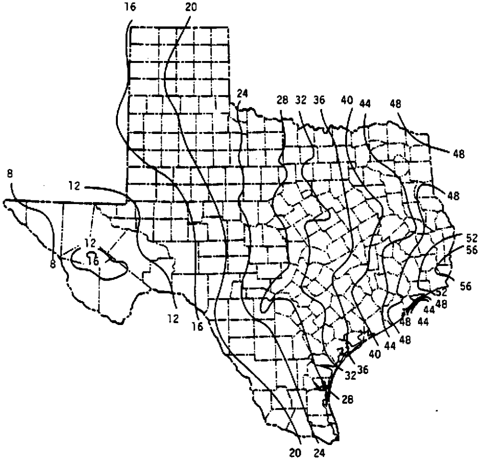


Figure 8.
Normal Annual Precipitation

Table 4.

Monthly Statewide Precipitation

Month	Average	Minimum	Maximum
January	1.7	0.2	3.9
February	1.7	0.3	2.9
March	1.6	0.3	3.2
April	2.5	0.8	6.7
May	3.4	1.2	7.1
June	2.8	0.7	5.6
July	2.4	0.9	5.1
August	2.4	0.6	5.7
September	3.2	0.6	6.9
October	2.4	0.0	5.9
November	1.7	0.1	5.3
December	1.8	0.2	4.0

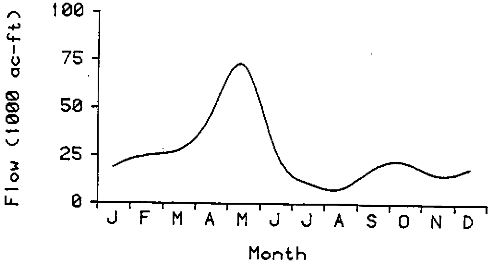


Figure 9.
Waco Lake Average Monthly Inflow

The next graph, Figure 10, which also plots monthly flow versus time for Waco Lake, is similar to the previous graph, but in addition it shows the extreme monthly flows for each month over the period of record. It also shows the limits of the average flow plus or minus one standard deviation from the average. Table 5 shown after the graph gives information pertaining to it.

Both the chart and table are based on the period of record from 1907-1970 (19). Waco Lake, U.S. Army Corps of Engineers (19). 84 to 94 percent of all monthly inflows of record are within one standard deviation of the monthly average inflow. This graph shows that inflows are almost never average, but tend to fluctuate greatly.

Drought is a major factor which influences availability of water in Texas. Drought is a period of time in which there is little or no rain. Because it occurs with no known pattern, there is little or no predictable cycle of drought in the state. The Texas Department of Water Resources (16) says the water supply is directly related to drought conditions since the pattern of rainfall is interrupted with sustained higher temperatures. At least 14 significant periods of drought of varying severity and geographical extent have occured in Texas during the 20th century. The most severe drought on record occured in Texas during the period 1950-1956. Beginning in the western part of the state, it spread across the state until about 94 percent of Texas' 254 counties were classified as disaster areas at the end of 1956. Another drought of almost equal severity began in 1916 and lasted three years.

During years of drought, evaporation from lakes and transpiration rates of vegetation increase and more rapidly deplete water supplies. These losses are an important consideration in reservoir design and in the volume of reservoir storage required to meet water supply requirements in years of drought. As with

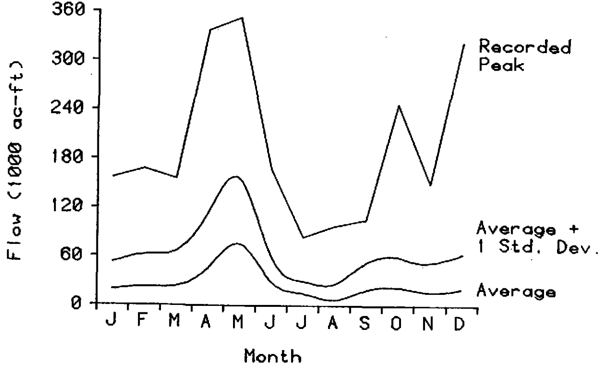


Figure 10.
Monthly Average, Recorded Extremes, and Standard Deviations of Inflow to Waco Lake

Table 5.

Monthly Average, Recorded Extremes, and Standard Deviation of Inflow to Waco Lake

Month	Average Flow (ac-ft)	One Std Dev (ac-ft)	Avg Plus One Std Dev	% of Flows Within One Std Dev	Highest Monthly Flow
January	18,354	32,121	50,475	91	161,102
February	24,539	38,109	62,648	86	166,880
March	26,618	36,990	63,608	88	163,500
April	47,899	68,304	116,203	89	335,100
May	75,094	81,071	156,165	84	354,300
June	30,386	32,342	62,728	92	168,700
July	12,652	19,130	31,782	89	89,142
August	8,668	19,079	27,747	92	98,300
September	18,252	35,842	54,094	91	226,600
October	21,968	42,120	64,088	89	249,670
November	16,320	32,746	49,066	92	151,300
December	19,933	44,784	64,717	94	324,400

Note: Based on period of record 1907 - 1970 other factors, evaporation, which affects water availability, varies with season of the year, not just location. Table 6 lists the average surface evaporation at different months of the year for Waco Lake. Figure 11 shows how average yearly evaporation varies with location across the state.

A series of several low flows may combine to form a "critical period". A critical period may be defined as a span of time in which rainfall is very sparse for several time periods. Therefore, reservoir inflows during this period are much below the amounts of water leaving the reservoir (due to evaporation, transpiration, pumpage, etc.). In fact, several periods of moderately low flows in series can be worse than just one period of extremely low flow preceded and /or proceeded by average or higher flows. Most reservoirs have demands placed upon them for a few periods of low inflow, but when these periods occur in sequence without a period of higher flow to refill the depleted storage, the reservoir may not be able to meet the demands placed upon it. If not, the reservoir level will fall to the bottom of the designated conservation storage pool. The length of this "critical" series of time periods is largely dependent on how low the flow is in comparison to the average flow and the magnitude of the demands placed upon the reservoir.

The amount of water that is available at any particular place is especially important to today's water resource planners. Only a finite quantity of available water exists for any one location. This gives the planner a basis for computations concerning reservoir size, outlet works, and the amount of electricity that can be produced at the site by hydroelectric power generators.

Table 6.

Average Monthly Waco Lake Surface Evaporation

Month	Average Surface Evaporation	Month	Average Surface Evaporation
January	1.90	July	8.69
February	2.51	August	7.93
March	4.20	September	5.41
April	5.04	October	4.63
May	5.23	November	2.94
June	7.08	December	2.13

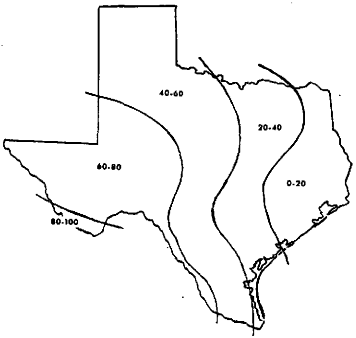


Figure 11.
Average Annual Net Lake Surface Evaporation in Inches, 1940-1965

Water Demand

Demand for water is seasonal in nature as are some of the other important factors previously discussed in this chapter. While water availability seems to peak in April and May, and have a low point in August, water demand seems to be greatest during the summer. This is due in part to the increased amount of lawn watering necessary during the summer to keep yards, trees, gardens, etc. from dying due to the low average expected precipitation during these summer months.

Figure 12 is provided to show how demand changes throughout the year. This chart shows the present demand for water by the City of Waco from our now-familiar example, Waco Lake.

Higher temperatures correlate very well with higher demands for water. As shown in Figure 12, as temperature increases, so does the demand for water. As the average monthly temperature decreases, the demand for water decreases at approximately the same rate. This fact has been documented for several cities across the state in works by Maidment (9). He attempted to establish a relationship between water demand (at various times of the year) and several different climatic parameters for 6 different Texas cities. Three of the cities were in the humid East Texas region and the other 3 were in the dry High Plains region. The High Plains cities were more responsive to changes in climate (especially rainfall) than were the cities of East Texas. An inch less than average rainfall for the month increased water use in one of the High Plains cities 2 to 7% of the average monthly demand. Demand was also responsive to evaporation. (Evaporation varies seasonally since it is a function of temperature). For example, an inch more than average pan evaporation increases water use by 3-8% of the monthly average in the High Plains cities.

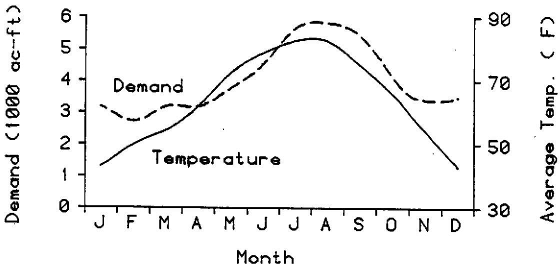


Figure 12.
Average Monthly Temperature and Water Demand for Waco

Demand for water increases during the summer for reasons other than just demand for drinking water. As stated previously, demand for electricity is higher during the summer, and highest then during hot weekday afternoons. Hydroelectric power production is well suited to handling the extra load or demand for electricity incurred during these periods because it has a relatively short start-up period compared to steam-driven electric power production. Therefore, demand for hydroelectric power production, and therefore demand for water to turn the turbines and generators, is seasonal in nature, peaking in the summer months.

Hydroelectric power is typically used for periods of peak electricity demand. In Texas, this is usually during the hot summer months, specifically week-day afternoons. People arrive at their homes after work and begin to turn on air-conditioning, fans, lights, and other electricity-consuming appliances. Currently, there is not enough installed hydropower capacity in the state to meet the state's baseload. (The baseload is the amount of electricity that is continuously required, or the minimum constant demand). Gas or coal-fired steam driven electrical generating plants supply the baseload, while hydropower is used to supply the periods of "peak" demand. Usually, it is not economical to use gas or coal-fired generating plants to meet peaking demands. It takes much longer to bring them up to the capability of producing peak power than it does to bring a hydropower plant on-line to help meet these demands. Therefore, since reliable hydropower is so important, it is also important to know the amount of water available at the reservoir for production of hydroelectric power.

Water-based recreation causes a seasonal demand for water also. As temperatures rise (spring,summer), fishermen, swimmers, boaters , and water skiers increase their use of reservoirs for their activities. As temperature falls, so does the level of activity on the reservoir. Recreational use of Texas' reservoirs is very popular, and is steadily increasing. This is due in part to the increased economic affluence of many people, increased leisure-activity time, and increased popularity of water-based recreation sports. These sports include fishing, boating, swimming, and water skiing as the main activities. Participants of these sports put pressure on reservoir managers to keep water levels high and as constant as possible.

Each sport has its own reasons for this high and constant water level demand. Fisherman want constant reservoir levels to insure large and healthy fish populations. A reservoir that fluctuates greatly over the course of a year will, in many cases, adversely affect fish spawns. It can also decrease water quality to the point of poor fish growth or even fish kills.

Swimmers want high water quality for hygienic reasons and want constant reservoir levels to insure access to cleared beaches and other swimming areas. Boaters, water skiers, and to a certain extent fishermen, want high reservoirs levels to avoid hitting tree stumps, flooded timber and other obstructions hazardous to boat travel and water skiing. Of course, higher reservoir levels mean larger surface areas available for use. For any given number of users, more area for use means less users per acre, and therefore less crowding.

Agricultural demand for water is also seasonal in nature. This is especially true for agriculture. Certain plants, classified as "determinants", have stages in their growth and development in which they are very susceptible to severe stress if not provided with proper amounts of water. This water can be from rainfall or irrigation, but in many areas of the state, rainfall is not in sufficient quantity, and/or it does not arrive at the precise time needed by the crop for proper development. This forces the farmer to use irrigation water to help insure a good crop. This places added emphasis on wise planning and management of all water resources, including surface water. Designers and planners must be able to provide the farmer with the amount of water necessary to irrigate properly (8).

Demand for water by the animal life of Texas' bays and estuaries varies seasonally also. Fresh water inflow into these areas from Texas rivers is very essential for marine animal life cycles. This inflow affects the salinity of these areas, and brings needed nutrients and sediment. These brackish water areas are the major spawning grounds for much of Texas' shrimp, shellfish, sport, and commercial fishes. Inefficient or unwise management of upstream reservoirs on rivers which flow to the Gulf could have a detrimental effect on Early spring through summer is an especially critical period for many of these different species. Spawning takes place during this time for these animals. Freshwater inflow to the bays and estuaries is necessary to maintain proper salinity levels for spawning. these areas and the economies of the surrounding cities which rely upon healthy marine life populations. Sport and commercial fishing is a multi-billion dollar a year industry along the Texas Gulf Coast (16).

Flood Threat

As previously stated, flooding is a major water-related problem in the state of Texas. As with drought, flooding is not easily predicted, and damage-causing floods have been recorded in every month and season of the year across the state (16). Unlike many parts of the world in which almost all floods occur in a distinct season of the year, floods can, and have, occured at any time of the year in Texas.

Table 7 is a tabulation of recorded precipitation events in which a station received 15 inches or more during a 24-hour period (5). Forty-four percent of these extreme rainfall measurements occurred in the month of September. Most of the other events occurred during the summer months. The data in this table are based on official precipitation gage readings. Unofficial measurements of 45 inches of rainfall was reported northwest of Alvin during Tropical Storm Claudette in July of 1979 along with several other reports of more than 25 inches near the cities of Freeport and Clear Lake. During a storm in September of 1921, more than 38 inches of rain was unofficially reported to have fallen in 24 hours at a point near Thrall, in Central Texas.

To get a site-specific idea of the variability of flood-producing storms in Texas, Table 8 has been provided. This chart is a presentation of the 100-year instantaneous flood peaks for inflow to Waco Lake for each month of the year and the percentage difference between that flow and the average 100-year instantaneous flow summed over each month of the year. Peak instantaneous flows vary greatly with time of the year. The extreme variations go from a low of more than 87 percent below average in August to more than 150 percent above in October. These figures for flow data were based on the Gumbel Distribution of extreme events and 18 years of flow data. More information on the development of this chart is included in the Case Study, Chapter 5.

As already mentioned, floods have occured for every season of the year. Table 9 shows flood events which have recorded at the Waco Lake dam site.

The Texas Agricultural Experiment Station, or TAES, (2) has produced tables of precipitation amount probabilities for one week to one month at many locations throughout Texas. Table 10 is for Temple, Texas, about 25 miles

Table 7.

Gaged Rainfall Events of 15 Inches or More in 24 Hours

Rainfall (inches)	Station	County	Date
29.05	Albany	Shackelford	4 Aug 1978
25.75	Alvin	Brazoria	26 Jul 1979
25.27	Orange	Orange	Sep 1963
25.24	Point Comfort	Calhoun	Jun 1960
25.06	Galveston Airport	Galveston	Sep 1958
25.06	Harleton	Harrison	Apr 1966
25.01	Armstrong	Kenedy	Sep 1967
23.11	Taylor	Williamson	9-10 Sep 1921
21.02	Kaffie Ranch	Jim Hogg	12 Sep 1971
20.70	Hye	Blanco	11 Sep 1952
20.60	Montell	Uvalde	27 Jun 1913
19.29	Danevang	Wharton	27-28 Aug 1945
19.20	Benavides No. 2	Duval	11 Sep 1971
19.03	Austin	Travis	9-10 Sep 1921
18.00	Fort Clark	Kinney	14-15 Jun 1899
17.76	Port Arthur	Jefferson	27-28 Jul 1943
17.47	Blanco	Blanco	11 Sep 1952
16.72	Freeport 2NW	Brazoria	26 Jul 1979
16.05	Smithville	Bastrop	30 Jun 1940
16.02	Hills Ranch	Travis	10 Sep 1921
16.02	Pandale	Val Verde	27 Jun 1954
16.00	Hempstead	Waller	24 Nov 1940
15.87	Anahuac	Chambers	27-28 Aug 1945
15.80	Orange	Orange	18 Sep 1963
15.71	Matagorda	Matagorda	1 May 1911
15.69	Whitsett 2SW	Live Oak	22 Sep 1967
15.65	Houston Airport	Harris	27-28 Aug 1945
15.60	Eagle Pass	Maverick	29 Jun 1936
15.49	Deweyville 5S	Orange	28 Oct 1970
15.20	World's End Ranch	Kerr	2 Aug 1978
15.00	Mercedes	Hidalgo	5 Sep 1933

Oct 1949 - 8 other stations reported 20" or more
Sep 1967 - 17 other stations reported 20" or more

Table 8.

100-Year Instantaneous Flood Peaks for Inflow to Waco Lake by Month

Month	100-Year Flow (cfs)	% Difference Between This Flow and Avg.	Month	100-Year Flow (cfs)	% Difference Between This Flow and Avg.
January	34,050	- 29.6	July	49,950	+ 3.2
February	35,550	- 26.5	August	6,000	- 87.6
March	30,600	- 36.8	September	35,620	- 26.4
April	44,100	- 8.9	October	121,200	+150.4
May	87,900	+ 81.6	November	33,000	- 31.8
June	84,300	+ 74.2	December	18,450	- 61.9

Note: Average 100 year flood magnitude = 48,390 cfs

Table 9.
Major Flood Events at Waco Lake

Date	Peak Flow at Reservoir Site (cfs)
September 1936	100,000
January 1938	43,100
April 1942	72,000
September 1942	39,000
May 1944	73,200
April 1945	153,800
February 1948	21,900
May 1952	19,200
April 1957	134,400
May 1968	40,000
April 1977	22,710
June 1981	27,170

Table 10.

Precipitation Amount Probabilities for 1-Week Periods at Temple

PERIOD BEGINS	MEAN MM	MEAN IN	MAXIMUM MM	MAXIMUM IN	MM 0.25 / IN 0.01	MM 6.4 / IN 0.3	MM 12.7 / IN 0.5	MM 25.4 / IN 1.0	MM 38.1 / IN 1.5	MM 50.8 / IN 2.0	MM 76.2 / IN 3.0	MM 101.6 / IN 4.0	MM 127.0 / IN 5.0
	PRECIPITATION MEANS, MAXIMUMS, AND PROBABILITIES FOR A 1-WEEK PERIOD				PROBABILITY (%) OF RECEIVING AT LEAST THE FOLLOWING AMOUNTS OF PRECIPITATION								
MAR 1	13	0.54	87	3.4	72	53	39	20	10	5	2	1	1
MAR 8	12	0.48	91	3.6	78	51	35	16	8	4	2	1	1
MAR 15	12	0.51	83	3.3	74	50	36	18	9	5	2	1	1
MAR 22	14	0.57	113	4.4	74	53	38	21	11	6	3	1	1
MAR 29	12	0.50	148	5.9	66	46	33	18	10	5	2	1	1
APR 5	19	0.75	93	3.7	77	58	45	27	17	11	4	2	1
APR 12	20	0.82	117	4.6	75	56	45	30	19	13	5	3	2
APR 19	28	1.13	168	6.6	86	69	58	40	28	19	10	5	3
APR 26	35	1.41	177	7.0	83	72	63	47	35	26	15	8	4
MAY 3	23	0.94	126	5.0	85	67	54	35	22	14	6	3	2
MAY 10	29	1.15	164	6.5	83	70	59	42	29	20	10	5	3
MAY 17	29	1.16	202	8.0	77	66	55	40	28	21	11	6	4
MAY 24	25	1.02	279	11.0	78	60	50	36	24	18	9	5	3
MAY 31	18	0.73	84	3.3	67	54	43	27	17	11	4	2	1
JUN 7	13	0.52	115	4.6	56	41	31	19	11	7	3	2	1
JUN 14	18	0.73	144	5.7	56	45	37	26	18	12	6	3	2
JUN 21	20	0.81	208	8.2	63	49	39	28	19	14	7	4	3
JUN 28	13	0.52	132	5.2	60	40	30	18	11	7	3	2	1
JUL 5	10	0.42	101	4.0	51	35	26	15	9	5	2	1	1
JUL 12	12	0.48	107	4.2	50	37	28	17	10	7	3	2	1
JUL 19	11	0.46	243	9.6	55	37	27	16	10	6	3	2	1
JUL 26	12	0.50	201	7.9	49	33	26	17	11	8	4	2	2
AUG 2	10	0.43	117	4.6	42	31	24	16	10	6	3	2	1
AUG 9	10	0.42	104	4.1	51	37	27	15	9	5	2	1	1
AUG 16	11	0.46	128	5.1	55	36	27	16	10	6	3	2	1
AUG 23	14	0.57	75	3.0	62	49	37	22	12	7	3	1	1
AUG 30	15	0.59	109	4.3	63	45	34	21	13	9	4	2	1
SEP 6	24	0.96	303	11.9	69	53	43	31	23	17	9	5	4
SEP 13	19	0.76	178	7.0	66	48	39	26	18	12	6	4	3
SEP 20	17	0.69	123	4.9	70	53	41	25	15	10	4	2	1
SEP 27	14	0.58	221	8.7	53	37	29	20	13	9	5	3	2
OCT 4	16	0.64	185	7.3	60	42	34	22	18	10	5	3	2
OCT 11	20	0.81	202	8.0	55	44	36	26	20	14	8	5	3
OCT 18	17	0.69	128	5.1	60	50	41	26	17	11	4	2	1
OCT 25	18	0.71	112	4.4	65	54	43	27	16	10	3	2	1
NOV 1	17	0.69	85	3.4	62	47	36	25	17	11	5	3	2
NOV 8	15	0.61	126	5.0	61	48	37	23	14	9	3	2	1
NOV 15	17	0.70	192	7.6	70	49	39	25	16	11	5	3	2
NOV 22	20	0.79	165	6.5	65	55	45	30	20	13	5	3	2
NOV 29	19	0.78	283	11.2	68	48	38	26	17	13	7	4	3
DEC 6	17	0.69	121	4.8	71	52	40	25	15	10	4	3	1
DEC 12	17	0.70	102	4.1	67	50	40	26	16	11	4	3	1
DEC 19	10	0.41	63	2.5	61	40	28	14	7	4	2	1	1
DEC 26	12	0.48	68	2.7	66	49	35	16	8	4	1	1	1
JAN 3	10	0.43	73	2.8	67	46	32	14	6	3	1	1	1
JAN 10	13	0.52	94	3.7	70	50	37	19	10	5	2	1	1
JAN 17	14	0.56	128	5.0	68	48	35	20	12	7	3	2	1
JAN 24	8	0.32	53	2.1	69	38	23	9	4	2	1	1	1
JAN 31	14	0.56	95	3.8	75	56	40	20	10	5	2	1	1
FEB 7	15	0.62	85	3.4	70	54	41	23	13	7	3	1	1
FEB 14	15	0.60	75	3.0	79	58	43	21	11	5	2	1	1
FEB 21	16	0.65	145	5.7	68	54	41	24	14	9	3	2	1

south southeast of Waco Lake.

This table shows the variance of average rainfall for this location. As mentioned in the section on water availability, inflow to Waco Lake varies with the season also. This is to be expected since inflow to most reservoirs is watershed runoff from seasonally varying rainfall. Although some snow does occur, most precipitation is in the form of rain.

According to the Texas Agricultural Experimental Station (2), the probability of receiving stated amounts of precipitation vary with the week of the year for all parts of Texas. However, most of the peaks of average rainfall do not occur at the same week of the year for all locations. For instance, the Temple station has a spring peak expected average rainfall of 1.16 inches for the week of May 17 and a fall peak of 0.96 the week of September 6, whereas Amarillo has a peak of 1.30 inches the week of June 7, and another peak of 0.86 inches for the week of August 9.

Even the estimates of the upper limit of rainfall that the atmosphere can produce (probable maximum precipitation or PMP) vary with season of the year and with location (22). Probable maximum precipitation means the theoretically greatest depth of precipitation for a given duration that is physically possible over a particular drainage basin. The magnitude of this type storm varies with month of the year and with location.

Maximum recorded rainfall readings from across the nation were used to obtain the estimates of PMP. Four rain gaging stations in Texas were used to help calculate the estimates. These Texas stations and observed depths in inches are as follows: Del Rio, 26.2 in 24 hours; Vic Pierce, 16.0 in 6 hours; 26.7 in 24, and 34.6 in 72; Thrall, 36.5 in 24 hours; and Hempstead, 18.6 in 24 and 21.1 inches in 72 hours.

The PMP for a storm of 24 hours duration is listed in Table 11 for different months of the year for two different locations in Texas. This is to show how the PMP varies with time of the year and with location. The first location is at 31 degrees latitude and 97 degrees longitude, near Waco Lake. The second is at 35 degrees latitude and 101 degrees longitude, near Amarillo.

It is probably no coincidence that 6 out of 7 months at Waco and 5 out 7 months at Amarillo have the highest PMP values of the year and at the same time coincide with the hurricane season. The hurricane season in Texas extends from June to October, although more frequent occurrences happen in August and September.

There are two major classed of storms (23). One is cyclonic, of which hurricanes are a part, and convective, of which thunderstorms are a part. Hurricanes, like drought, are a facet of the climate that affect the quantity of water supplies in the regions in which they occur. Tropical cyclones, particularly tropical storms and hurricanes, are a reoccuring threat to the Texas Gulf Coast region during the summer and fall. Nearly all of the tropical cyclones that affect the Texas coast start in the Gulf of Mexico, Caribbean Sea, or in other parts of the North Atlantic Ocean. Although the hurricane season extends from June to October, tropical cyclones are most frequent in August and September and rarely affect the coast before mid-July or after mid-October. Hurricanes contribute large quantities of precipitation in addition to producing high winds and storm tides.

Hurricanes contribute large amounts of rainfall in addition to producing high winds, significant storm tides, and usually result in significant property damage, and sometimes loss of life (16). Vic Pierce, Texas reported 26.7 inches of rain in 24 hours on June 24, 1954 as the direct result of precipitation

Table 11.

Probable Maximum Precipitation

Month	24 - Hour 10 mi Rainfall Depth (in) 97 Longitude 31 Latitude	 101 Longitude 35 Latitude
January - February	24	14
March	29	17
April	38	27
May	43	33
June	44	35
July - August	43	34
September	44	35
October	38	26
November	29	18
December	26	15

from Hurricane Alice. Precipitation from this cyclonic storm was heaviest about 90 miles northwest of Del Rio, Texas as the storm was losing its warm-core tropical structure.

According to the Texas Department of Water Resources (16), the Gulf of Mexico is the biggest source of moisture for precipitation in Texas. As with total amounts of water available for man's use, the amount of rainfall from the PMP decreases in Texas as the distance from the Gulf increases.

To show how the rainfall (and hence potential flooding from watershed runoff) varies across the state, the rainfall amounts at various locations for 1, 6, and 24 hour duration 10 and 50 year return period storms are listed in Table 12. Each of these cities is in a different major geographic region of the state.

For certain locations, damages due to floods, as well as the flood events themselves, are seasonal in nature. Agriculture production in floodplains takes place only during certain parts of the year. Flooding during these parts of the year causes higher damages than during the remainder of the year when crops are not in production. An example of this is the damage versus discharge relationships developed by the U.S. Army Corps of Engineers for several primarily agricultural regions downstream of Whitney and Waco Reservoirs on the Brazos River in Texas. The main crops of highest value, maize and cotton, are grown during the months of May through July. The damage discharge curves for these months and this location show higher damages than do the same curves for the remainder of the year.

For example, a river discharge of 160,000 cfs on the Brazos River at Waco shows a damage value of $11,000 in October, November, and December. The same flowrate in May, June, and July, when the land is used for crop production, is higher at $80,000.

Table 12.
Texas Storm Rainfall Geographic Variability

City	10 Year Return Period Rainfall (in) 1 Hour	6 Hour	24 Hour
El Paso	1.4	2.0	3.0
Lubbock	2.3	3.3	4.5
San Antonio	2.9	4.8	6.5
Brownsville	3.3	5.4	7.5
Waco	2.9	4.8	6.5
Wichita Falls	2.7	4.3	5.5
Houston	3.4	5.8	8.4
Tyler	3.0	5.0	7.0

City	50 Year Return Period Rainfall (in) 1 Hour	6 Hour	24 Hour
El Paso	1.8	3.0	3.5
Lubbock	2.9	4.5	6.0
San Antonio	3.8	6.3	8.7
Brownsville	4.2	7.2	10.1
Waco	3.9	6.4	8.7
Wichita Falls	3.5	5.5	7.6
Houston	4.2	7.5	11.0
Tyler	3.9	6.6	9.3

However, not all flood damage is seasonal in nature. Residential and industrial areas without agricultural crop production have discharge versus damage relations that do not vary with the season of the year. Only when the value of a potentially inundated area is higher for certain seasons than others can the damage due to flooding be seasonal in nature.

Seasonal Factors Discussion

Up to this point, Chapter 3 has attempted to present and explain the major factors which affect reservoir operation. This portion of Chapter 3 will discuss and present conclusions derived from the previous pages which described the four major seasonal factors: water availability, water demand, flood threat, and flood damage potential. Discussion of the degree to which these factors coincide and their pertinence to seasonal rule curve operation is presented.

As far as water resources is concerned, the ways of humans are much more predictable than those of nature. In other words, people have preferences concerning direct or indirect water use that repeat themselves in fairly easily predicted patterns. Nature (i.e., weather), on the other hand, also repeats itself, but not in patterns as easily predictable as those of humans.

As pointed out in the section concerning water demand, water use increases when the temperature goes up and/or the total rainfall for the year is below that deemed desirable. The correlation for this is high and fully accepted by water resource planners. It is an accepted fact that as the weather gets hotter and drier that people's water demand increases, and goes down as temperature and rainfall deficits decrease. The public's demand for water varies with temperature and rainfall throughout the year and is thus fittingly described by the term "seasonal".

Irrigation of man's crops ties agricultural practices directly to water demand as an influence upon seasonal reservoir operation. Seasonally varying crop water requirements are well documented (8). Certain crops grow only in certain months of the year and thus easily show the times of the year in which irrigation may be required.

Other aspects of human nature which are easily predictable and influence seasonal reservoir operation are electrical power demands and recreational needs. Daily and monthly electrical power demands are expected to vary with the time of year as a fairly direct result of human nature. When the weather gets hot and people arrive at home after work, they turn on their home air conditioners. Water-based recreation increases as the temperature increases also.

Even marine animals are more predictable in their demands upon reservoirs than is nature itself (i.e., weather). Their need for fresh water during spawning is a water demand that reoccurs in a set, predictable yearly cycle that peaks during certain parts of the year, and is thus "seasonal" also.

Another major factor involved in reservoir operation which is well described as "seasonal" and is linked to human behavior is flood damage potential. This factor is also tied directly to man's agricultural practices. It is easy to say that flood damage potential varies during the year (i.e., seasonal) when man's crops which are grown in a floodplain only grow during certain months of the year.

Unfortunately, describing the other two major factors as truly seasonal is not as clearcut as the first two. The first two are linked to human activities (i.e., farming, electricical demands, etc.). The third and fourth are linked directly to the weather-related aspects of nature. These two factors, water availability and flood threat, as they affect reservoir operation in Texas, appear to be very seasonal in nature when average monthly values are compared. Figure 9 and Table 5 show how average inflow to Waco Lake varies by month, with a peak in May and a low inflow in August. The 100-year instantaneous flood flow peaks show a very definite seasonal trend (Table 8). Even the probable maximum precipitation (PMP) shown in Table 12 shows definite seasonal trends.

However, when individual months are compared to the average values for these months, for whatever component of water availability or flood threat, the "seasonality" becomes much less definite. Wide fluctuations in monthly reservoir inflow are shown in Figure 7. This is in contrast to Figure 9 which showed average monthly inflow to be very seasonal. The peak recorded flows are seasonal in nature, but do not all occur within the same month of the year from year to year.

Drought is a facet of water availability that does not appear to be seasonal. Since it occurs with no known pattern, it fails to meet the criteria for being seasonal.

Although average monthly flows are seasonal, damaging floods have been recorded in every month of the year in Texas (24). A list of the month of the year and the number of storms cited appears in Table 13. This shows that more damaging storms of record occured in Texas during September than any other month.

However, average monthly inflow for Waco Lake showed a higher volume of inflow during May than for any other month, including September. This leads the author to conclude that water availability and flood threat do not exactly coincide. In other words, damaging floods do not always fill up a reservoir's

Table 13.
Damaging Texas Floods by Month

Month	Storms Sited	Percent of Total	Month	Storms Sited	Percent of Total
January	1	2.4	July	4	9.5
February	1	2.4	August	2	4.8
March	2	4.5	September	10	23.8
April	5	11.9	October	1	2.4
May	7	16.7	November	1	2.4
June	6	14.3	December	2	4.8

conservation pool to increase water availability. On the other hand, many of the damaging flows which were recorded occured before flood control reservoirs were available to help stem the flow resulting from damaging storms.

The basis for the belief that water availability and flood threat do not always occur at the same time of the year is further supported by a comparison between the 100-year instantaneous monthly flood flows of Table 8 previously mentioned and the monthly inflow volumes for Waco Lake (Table 5). October shows a instantaneous 100-year peak flow of 121,490 cfs and May shows 87,230 cfs. However, the October average monthly inflow is 22,498 acre-feet and May is 69,978 acre-feet. In fact, October is only the sixth wettest month, on the average, with May first and August last. At the Riesel, Temple, and McGregor rainfall gaging stations maintained by the Texas Agricultural Experiment Station (2), October registers the 4th, 4th, and 5th wettest month of the year, respectively, for average recorded rainfall. Riesel and McGregor are approximately 15 miles southeast and southwest of Waco Lake respectively. Temple is about 45 miles southwest of Waco Lake. The closeness of these stations to Waco Lake allows the researcher to assume that the average rainfall occuring at these stations is representative of conditions at the reservoir. Table 14 shows these recorded averages.

The peak instantaneous flows for each month were based on applying the Gumbel Distribution to parameters derived from 22 years of recorded or derived monthly instantaneous flow data from 1962 through 1981. The peak instantaneous flow data for each month of the year from 1962 through 1969 were recorded by the Corps of Engineers. The peak flows were read directly from the monthly graphs for these years. However, the peak instantaneous monthly flows for the period 1970 through 1981 had to be converted from daily

Table 14.

Monthly Average and Maximum Recorded Rainfall at Riesel, Temple, and McGregor

Month	Riesel Avg (in)	Riesel Max (in)	Temple Avg (in)	Temple Max (in)	McGregor Avg (in)	McGregor Max (in)
January	2.11	5.1	2.04	7.5	2.16	7.2
February	2.53	5.7	2.43	6.7	2.27	5.2
March	2.75	7.5	2.30	6.8	2.31	6.9
April	4.02	15.6	4.03	11.6	3.91	17.6
May	4.33	12.6	4.70	14.5	4.22	12.2
June	3.50	8.8	2.90	9.5	3.09	14.3
July	1.85	11.3	2.13	19.8	2.14	13.3
August	2.08	8.9	2.10	11.6	1.95	7.8
September	2.94	8.9	2.10	11.9	3.05	14.7
October	2.96	9.0	3.07	9.6	2.99	10.2
November	2.85	10.2	2.97	13.1	2.48	10.7
December	2.56	7.0	2.75	11.2	2.46	13.1

recorded values. (The specific procedure involved is explained in detail in the Case Study, Chapter 5).

Therefore, with only 22 years to serve as a data base upon which to extrapolate the peak instantaneous flow of the 100-year return period storm for each month of the year and since only ten of those 22 years had actual data to use (the other 12 being derived), there was probably a degree of error, or at least uncertainty, introduced into the calculations (Table 8).

On the other hand, the average monthly inflows at Waco Lake are probably more indicative of future total monthly flows than the 22 years of peak monthly instantaneous flows are indicative of future peak instantaneous flows. The foundation for this statement is twofold: First, the average monthly inflow totals are from actual recorded data, not partially derived and partially recorded as were the set of peak instantaneous flows. Second, the peak instantaneous flows only had a 22 year base whereas the monthly totals had a base of 75 complete years (1907- 1981).

To summarize the previous paragraphs of this seasonality discussion, it appears that the 4 major factors which effect reservoir operation in Texas range from definitely seasonal to slightly seasonal. The two factors which are directly linked to human behavior, water demand and flood damage potential, appear to be solidly seasonal in nature. The other two factors, flood threat and water availability, which are tied closely to the weather aspect of nature, are not as solidly seasonal as the previous two. When monthly average values of the components of water availability and flood threat are analyzed, it appears that they too are very seasonal. However, it is the individual monthly values of the components of these two factors that fluctuate greatly from year to year and thus do not follow a seasonal trend annually as closely as do the first two factors.

Therefore, factors which influence reservoir operation are not as solidly seasonal in nature as might appear when only average monthly values of their respective components are analyzed. This statement of findings should warn the researcher interested in seasonal reservoir operation to be careful to examine the deviations from the norm. These deviations should be expected instead of unexpected, and therefore cause less problems to reservoir users in the future. After all, if floods occur in months in which they do not normally occur, but the unexpected has been planned for in advance, damaging surprises may be avoided. In the same vein, months that normally receive plenty of rain to meet water demands will not cause undue hardships if plans are made in advance what to do if average seasonal rainfall does not take place as expected.

It is unfortunate from a water supply standpoint, that water availability and water demand do not usually occur during the same part of the year. Demand peaks in late summer with the greatest amounts of available water occuring in mid to late spring.

From a flood control viewpoint it is also unfortunate that water demand and flood damage potential do coincide around the same time of the year. This creates somewhat of a problem situation. Flood threat is not as seasonal as the other factors and damaging floods can actually occur at any time. Therefore, since water demand peaks in the late summer when flood damage potential is also high, a flood of damaging proportions could occur and there would not be as much empty storage space in the reservoir to capture excess runoff as might be possible if that volume was not already being used to store water to meet the high summer water demand.

However, seasonal rule curve operating procedures have the potential to partially alleviate some of these problems. Rule curves can be fashioned in such a way as to minimize the effects of the previously mentioned problems. For example, although water demand at Waco Lake peaks in August and water availability peaks three months earlier in May, a rule curve can be constructed in such a way as to allow more water to be stored in the reservoir during April and May. This water will be used during the summer through the peak demand period in August. At that point the rule curve can begin to allow less water to be stored in the reservoir, descending dayly, weekly, or monthly until an acceptable amount of empty storage space is available to store floods occurring between early fall and late spring. Any floods occuring during this period would be stored only long enough to reduce downstream flooding. All water stored above the level allowed by the rule curve at that particular time of the year would be released as quickly as possible without causing flood damages.

In this way, rule curve operation could store water that is available in May to be used later in August. This would help to offset the problem of water availability and water demand not coinciding. The portion of the rule curve allowing less water to be stored in the reservoir as early fall approaches would help to alleviate worries caused by the chance occurance of a large flood when the reservoir was full. This portion of the rule curve would also coincide with a smaller demand for water, which would require less storage in the reservoir and thus make available more empty storage for flood control. Lower allowable storage values during this time would be helpful because flood damage potential is also higher during the late summer.

CHAPTER IV

REVIEW OF ANALYSIS TECHNIQUES FOR DETERMINING OPTIMUM RESERVOIR OPERATING POLICIES

A state-of-the-art review of systems analysis techniques applied to reservoir operation in general is documented by Wurbs, Tibbets, Cabezas, and Roy (1985). The intent of the present section is to provide (1) an overview summary of the types of models used in analyzing reservoir operations and (2) an introduction to specific techniques incorporated into the analysis strategy developed by the study and outlined in subsequent chapters.

Numerous mathematical models have been reported in the literature for sizing storage capacities and establishing release policies during project planning and for supporting release decisions during real-time operations. Each particular model was developed specifically for either planning or real-time applications or may be applicable in either situation. However, the present investigation addressed the somewhat different situation of evaluating plans for seasonal reallocation of storage in existing reservoir systems. Little attention has been directed in the literature toward reevaluating existing operating policies in response to changing public needs and conditions. A comprehensive literature review revealed essentially no models developed specifically for evaluating seasonal or long-term reallocations between flood control and conservation or otherwise considering tradeoffs and interactions between flood control and conservation purposes. However, generalized models and modeling concepts can be applied meaningfully to the analysis of seasonal rule curve operations even if they were not developed specifically for that particular application. This chapter addresses modeling of reservoir operations in general but from the perspective of identifying those modeling concepts and techniques which are pertinent to the seasonal storage reallocation problem.

Types of Models

The various types of mathematical models used in analyzing reservoir operations can be categorized as (1) simulation, (2) optimization, and (3) streamflow synthesis. A broad range of types of analyses routinely applied in the planning, design, and operation of reservoir projects are included in the category of simulation modeling. The role of optimization models is to provide the capability to search through a large number of possible combinations of values for a set of decision variables to find the decision variables to find the decision policy which maximizes or minimizes a defined objective function. Streamflow synthesis methods are used to extend and supplement historical records for developing required input data for simulation and optimization models.

A simulation model is a representation of a system used to predict the behavior of the system under a given set of conditions. Simulation is the process of experimenting with a simulation model to analyze the performance of a system under varying conditions. Although simulation only serves to analyze system performance under a given set of conditions, trial-and-error runs of a simulation model can be used to search for an optimal decision policy. However, numerous simulations may be required to achieve acceptable results, and the optimum decision may never be found. Consequently, application of mathematical programming or optimization techniques, which automatically find the optimum decision policy, to reservoir operation has received much attention.

Simulation models have been proven through practical application to be a valuable aid in sizing reservoirs and establishing operating policies. During the past twenty years, a major thrust of research and the resulting literature related to reservoir operation has been to supplement simulation models with optimization techniques such as linear programming, dynamic programming, and various nonlinear programming algorithms. The academic research community in particular, and many practitioners as well, have been very enthusiastic about applying optimization techniques to reservoir operation problems. Research in this area has dominated the water resources planning and management literature. Research results, case studies, and experience in application of optimization models in actual planning and real-time operation decisions indicate a high potential for improving reservoir operations through their use. Optimization models have played a relatively minor role compared to simulation models in regard to influencing decisions made in the planning and operation of actual projects. Simulation is the "work-horse" of reservoir system analysis. Optimization techniques provide valuable supplemental analysis capabilities for a select number of specific types of problems.

Optimum sizing of storage capacities, establishing release policies, and real-time operations are complex tasks involving numerous hydrologic, economic, environmental, institutional, and political considerations. Defining system objectives, developing criteria for quantitatively measuring system performance in fulfilling the objectives, and handling interactions and conflicts between objectives is a major area of complexity. Mathematical optimization techniques require that the real system be represented in the proper mathematical format. Representing complex project objectives and performance criteria in the required format, without unrealistic simplifications, as a particularly difficult aspect of the modeling process which limits the application of optimization techniques.

Since simulation models are limited to predicting the system performance for a given decision policy, optimization models have a distinct advantage in this regard. However, simulation models have certain advantages over optimization models from a practical applications perspective. Simulation models generally permit more detailed and realistic representation of the complex hydrologic and economic characteristics of a reservoir system. Stochastic analysis methods can be combined with simulation models easier than with optimization models. The concepts inherent in simulation tend to be easier to understand and communicate than optimization modeling concepts.

Combined use of simulation and optimization models is an effective analysis strategy for certain reservoir operation problems. Preliminary screening with an optimization model may be used to develop a manageable range of alternative decision policies for further detailed analysis with a simulation model. Another approach is for an optimization model to be embedded as a component of a complex simulation model. Likewise, an optimization model can be used to compute the objective function value for any given set of decision variable values.

Although the potential for applying optimization techniques in analyzing storage reallocation plans was investigated, the evaluation strategy developed in the present study is based strictly on simulation. The reallocation decision problem is basically to determine whether conversion of storage capacity between flood control and conservation is warranted and, if so, the optimal storage capacity allocation. Capabilities are needed to assess system performance as precisely and as meaningfully as possible for a few alternative reallocation plans rather than search through a large number of possible capacity allocations. Consequently, optimization models are not particularly advantageous for this particular application.

Inadequate basic data is a major concern in analyzing reservoir operations. Hydrologic data synthesis methods are used to overcome the limitations of short-duration records and missing data. Although rainfall, evaporation, and other data may be synthetically generated, the emphasis in reservoir operation studies is usually on extending streamflow data for reservoir inflows and flows at downstream control points.

Simulation models are often used deterministically with historical period of record or critical period inflows. However, the historical period of record is typically too short to provide an adequate basis for certain types of analysis. Stochastic hydrology techniques can generate synthetic streamflow sequences, statistically similar to the historical record, for input to simulation models. The monthly Markov model is the fundamental approach most often used for streamflow synthesis. Wurbs, Tibbets, Cabezas, and Roy (1985) discuss alternative stochastic streamflow generation models as well as simulation and optimization modeling capabilities.

Simulation Models

The major types of simulation models typically used in analyzing reservoir operations can be categorized, as outlined in Figure 13, as (1) hydrologic, (2) economic, (3) water quality, and (4) sediment transport. Although water quality and sediment transport may be important in evaluating storage reallocation plans in some situations, in general hydrologic (water quantity) and economic analysis will be the primary thrust of the simulation effort.

1. Hydrologic (Water Quantity)
   - rainfall-runoff (watershed models)
   - streamflow (flood routing)
   - reservoir yield and reliability
   - system operation for flood control
   - system operation for conservation purposes

2. Economic
   - flood damages
   - benefits for conservation purposes

3. Water Quality

4. Sediment Transport

Figure 13.
Major Types of Simulation Models

Consequently, the present investigation was limited to hydrologic and economic simulation models. Hydrologic simulation models include rainfall-runoff and streamflow simulation, computation of reservoir yield and reliability, and modeling system operations for flood control and conservation purposes. Economic models typically extend hydrologic simulation to include evaluation of flood damages and benefits associated with water supply, hydropower,and possibly other conservation purposes. A specific model may contain capabilities for one or several of these types of hydrologic and economic analyses. All of the analyses are pertinent to the problem of evaluating storage reallocation plans.

Rainfall-Runoff Models

Streamflows at pertinent locations in the reservoir-stream system are fundamental input to hydrologic simulation of reservoir operations. Historical gaged streamflow data is utilized whenever feasible. Hydrologic synthesis methods are available for extending streamflow records and filling in missing data. In many cases, streamflow records are unavailable or major changes in the watershed have rendered the historical data no longer representative of present and projected future streamflow conditions. Rainfall data combined with rainfall-runoff, or watershed, modeling are then used to develop the required streamflow data. Rainfall-runoff modeling is most often used for developing single-event flood hydrographs but can also be used to develop long-term continuous streamflow sequences. The HEC-1 Flood Hydrograph Package is an example of a single-event rainfall-runoff model which has been widely used to develop flood hydrographs for reservoir design and operation studies (3). The Streamflow Synthesis and Reservoir Regulation (SSARR) Model is a continuous rainfall-runoff model developed specifically for reservoir design and operation studies. Viessman, Knapp, Lewis, and Harbaugh (1977) provide an overview of rainfall-runoff simulation and describe a number of readily available generalized computer models. In general, rainfall-runoff modeling could play an important role in analyzing season rule curve operating plans. However, the case study analysis was based on measured streamflow data without needing to use rainfall-runoff modeling.

Streamflow Models

Streamflow modeling is an integral part of simulating reservoir flood control operations. The term streamflow model is used here to mean flood routing, water surface profile computations, and related flood wave analysis methods. Flood routing is the computation of the magnitude (discharge and/or stage) and celerity, as a function of time and location, of a flood wave propagating through a river or reservoir. Reviews of the current state-of-the-art of flood routing are provided by Fread (1982) and Wurbs (1985). Although two- and three-dimensional models have been developed, the present state-of-the-art of simulating flows in rivers and reservoirs, from a routine practical applications perspective, is one-dimensional modeling. One-dimensional flood routing models can be categorized as hydraulic, hydrologic, or purely empirical.

Hydraulic routing is based on the two one-dimensional equations of unsteady flow, commonly called the St. Venant equations, which express the physical laws of conservation of mass and momentum. Due to the mathematical complexity of the theoretical equations, for many years significant simplifications were necessary in order to obtain solutions. During the last two decades, solution of the complete St. Venant equations has become practical using numerical methods and high speed computers. A flood routing method based on the complete St. Venant equations is called a dynamic wave model, or dynamic routing. The Operational Dynamic Wave Model (DWOPER) developed by the National Weather Service is probably the most widely used of the available generalized dynamic routing models (4). A variety of simplified hydraulic routing techniques have been developed by omitting or linearizing certain terms in the St. Venant equations or making other simplifying assumptions.

Hydrologic routing models are based on a relationship between storage and discharge combined with the storage form of the conservation of mass equation

$$I - O = \frac{dS}{dt}$$

where I is inflow, O is outflow, and dS/dt is change in storage with respect to time. The difference between the various hydrologic routing techniques is the form of the relationship between storage and/or outflow. Hydrologic channel routing methods include Muskingum, working R&D, variable storage coefficient, modified Puls, and their variations.

Reservoir routing is commonly performed using the modified Puls method which is based on the assumption that storage is dependent only on outflow. This conservation of mass equation is written in finite difference form and rearranged to give the following equation

$$\frac{2S_2}{\Delta t} + O_2 = I_1 + I_2 + \frac{2S_1}{\Delta t} - O_1$$

where the subscripts 1 and 2 refer to the beginning and end of the routing interval delta t. The equation is solved step-by-step for the left-hand side, with the right-hand side of the equation known at each step of the computations. A relationship between the left-hand side of the equation and outflow must be developed from a known storage versus outflow relationship. A level reservoir water surface is assumed.

Some flood-routing methods are based strictly on intuition and observations of past floods. Lag method and gage relations are examples of purely empirical methods.

Hydraulic routing methods compute both discharge and stages as a function of time and location. However, hydrologic and empirical routing methods are limited essentially to computing a discharge hydrograph from a known hydrograph an upstream location. Water surface profile computations are then used to compute stages corresponding typically to peak discharges. Water surface profile computations are based on an iterative solution of the one-dimensional energy equation. The standard step method is usually used. The HEC-1 Flood Hydrograph Package and HEC-2 Water Surface Profiles computer programs are probably the most widely used generalized models for hydrologic routing and water surface profile computations (3). These models are used in various applications including reservoir studies.

Hydrologic routing in combination with water surface profile computations has been the traditional approach to streamflow modeling for many years. Dynamic routing is more complex but also more accurate. Dam breach flood wave analysis requirements of recent federal and state dam safety programs have provided the impetus for developing greatly expanded dynamic routing capabilities during the past decade (Wurbs 1985). Precise simulation of the effects of storage reallocation plans on major flood event stages upstream and downstream of a dam is another potential application of dynamic routing models. However, this research topic was not pursued in the present investigation. The flood routing required in the case study analysis was performed using traditional hydrologic routing methods available in the generalized computer program adopted for the study.

Reservoir Yield and Reliability

The relationship between storage capacity, yield, and reliability is a fundamental and extremely important aspect of planning, design, and operation of a reservoir for conservation purposes. Yield is the amount of water which can be supplied from a reservoir in a specified period of time. Traditional analyses have been based on the concept of dependable or firm yield, which is the maximum rate of withdrawal which can be maintained continuously assuming the period of record historical inflows. Thus, analysis of the complex uncertainties involved in providing various levels of water supply are simplified to stating the constant yield which could be provided by a given storage capacity if future inflows reproduce the historical period of record. Reservoir inflows, as well as all other hydrologic phenomena, are stochastic in nature. Therefore, it is not possible to guarantee any yield with certainty. Reservoir reliability is an expression of the likelihood or probability of meeting given yield levels. The concept of reservoir reliability expands the concept of firm yield to provide a more meaningful basis for dealing with the uncertainties inherent in the random nature of hydrologic variables.

The yield provided by a given storage capacity is computed based on a mass balance of reservoir inflows, releases or withdrawals, evaporation and other losses, and change in storage. McMahon and Mein (13) provide a comprehensive review of methods for analyzing reservoir capacity versus yield relationships. Reservoir reliability is the probability that a specified demand will be met in a given future time period. Reliability is the complement of the risk of failure or probability that the demand will not be met. Firm yield and reservoir reliability are major components of the evaluation strategy outlined in the coming chapters.

System Operation for Flood Control

Simulation of flood control operations is another major modeling task addressed by the present study. A model can include the capability to compute reservoir release rates for each time interval during the simulation period based on specified operating rules. Various forms of operating rules may be incorporated into a model. For example, when the water level is in the flood control pool, reservoir releases are typically based on emptying the pool as quickly as possible without contributing to downstream flooding. Allowable nondamaging discharges are specified at downstream control points. Reservoir inflows and incremental local inflows at the downstream control points are provided as input to the model. For each control point the model compares the discharge assuming no reservoir releases to the allowable discharge. If the allowable discharge is larger, reservoir releases are made. Since the releases at the reservoir must be routed to the downstream control points to reflect attenuation time, an iterative solution is required to determine the release rate which will maintain the allowable flow levels at the control points. Additional release criteria incorporated into the model includes balancing the storage levels in multiple reservoirs releasing to the same control point and limiting the rate of change of the release rate.

A generalized model developed by the Southwestern Division (SWD) of the U.S. Army Corps of Engineers is routinely used to model reservoir operations for Corps of Engineers projects in Texas and other states in the Southwestern Division. The SWD model simulates the daily sequential regulation of a multipurpose reservoir system including the computations discussed above (6). As discussed later in this report, the similar HEC-5 Simulation of Flood Control and Conservation Systems computer program was used in the present study.

System Operation for Conservation

Simulation of reservoir operations for conservation purposes typically involves computing releases to meet water supply and hydroelectric power demands. Reservoir storage levels, releases, and flows at pertinent locations are computed for each time interval during simulation. The simulation is essentially an accounting procedure for tracking the movement of water through the system. Input data includes reservoir characteristics, reservoir inflows and incremental lateral inflows at downstream control points, evaporation rates, and target demands. Diversions and return flows could occur at a reservoir or at downstream control points. Minimum instream flows may be required for fish and wildlife habitat or other purposes. HEC-5, discussed later in this chapter, allows diversions and instream flows to be designated as required or desired with respect to the amount of water in storage. Required demands are met as long as the reservoir storage level is above the top of the inactive pool. Desired demands are met only if the reservoir storage level is above the top of buffer pool.

Whereas flood control simulation requires a relatively short (an hour to a day) routing interval to track hydrograph peaks, simulation of conservation operations are typically based on a longer routing interval (up to a month). Flood routing techniques are not used. A simulation may be performed with historical period of record, critical period, or synthetically generated streamflows.

Computer models for simulating conservation operations include: the MIT Simulation Model (15), HEC-3 Reservoir System Simulation for Conservation (7), Potomac River Interactive Simulation Model (14), and several models developed by the Texas Water Development Board (1, 17, 10, 11, 12).

Economic Evaluation

Economic evaluation consists of estimating and comparing the benefits and costs, expressed in dollars, which would result from alternative plans of action. Fundamental economic evaluation procedures incorporated into simulation models used to analyze reservoir operations are outlined below.

Flood Damage Evaluation

Economic evaluation of flood control plans have traditionally been based on the concept of average annual damages. The inundation reduction benefit is defined as the difference in average annual damages without and with a proposed plan. Computing average annual damages using the damage-frequency method described below has been an integral part of the economic evaluation procedures followed by the Corps of Engineers and other federal agencies in planning flood control improvements for many years. The method is incorporated into several generalized computer programs and is a major component of the procedure presented in following chapters.

Average annual damage computations are based on the statistical concept of expected value. Expected or average annual damage is computed as the integral of the damage versus exceedance probability function. Exceedance frequency versus peak discharge, discharge versus stage, and stage versus damage relationships are combined to develop the damage versus exceedance frequency function. A fundamental assumption of the procedure is that damages can be estimated as a function of peak discharge or stage. Additional analyses are required to show how damages change with variations in flow velocity, duration, and sediment content.

The magnitude of a flood threat can be quantified in various ways. Discharges, stages, and damages at specified .locations can be estimated for historical storms (such as the most severe flood of record), statistical floods (such as the 50-year and 100-year recurrence interval floods), and/or hypothetical floods (such as the standard project flood). Expected or average annual damage is actually a frequency weighted sum of damage for the full range of damaging flood events and can be viewed as what might be expected to occur, on the average, in any present or future. Additional meaningful information, including discharges, stages, and damages associated with a range of storm magnitudes, are generated in the process of computing average annual damages.

A river system is divided into reaches of analysis purposes. Average annual damages are computed for each reach and summed to get a total. Each reach is represented by an index location. The functional relationships developed are developed for each index location and represent the variables for the entire reach.

Since watershed and floodplain conditions change over time due to urbanization and other factors, average annual damages are computed assuming conditions expected to occur at a particular point in time. The computations can be repeated for a discrete number of future points in time. The average annual damages computed for alternative future years can be converted to an equivalent value using discounting techniques and an appropriate discount rate and period of analysis.

The basic functional relationships used in computing expected annual damages are illustrated in Figure 14. The discharge-frequency, stage-discharge, and stage-damage relationships are computed from field data. The damage-frequency relationship is derived from the other three functions. Expected annual damage is computed by numerical integration of the damage-frequency function.

The peak discharge versus exceedance frequency relationship describes the probabilistic nature of flood flows and is developed using standard hydrologic engineering techniques. *Exceedance frequency* or exceedance probability is the probability that a given discharge level will be equalled or exceeded in any year. The exceedance frequency is the reciprocal of the recurrence interval. Discharge-frequency functions are commonly computed either from a statistical analysis of gaged streamflow data or through rainfall-runoff modeling.

Stage versus discharge is a basic hydraulic relationship that relates stage or water surface elevation to discharge and is commonly referred to as a rating curve. It is usually developed from water surface profile computations. A stage at an index location corresponds to a water surface profile along a river reach.

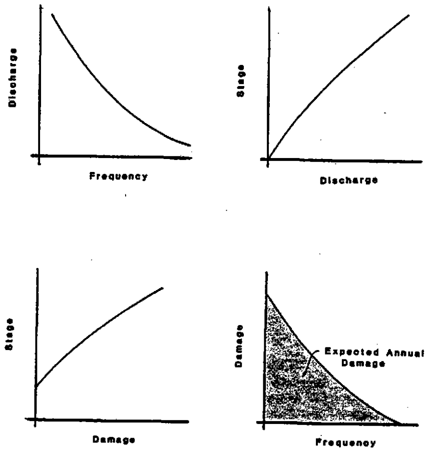


Figure 14.
Computation of Expected Annual Damages

The stage versus damage relationship represents the damage, in dollars, which would occur along a river reach if flood waters reach various levels. Three alternative approaches which have been taken in developing stage versus damage relationships involve using: (1) historical flood damage data for the study area; (2) synthetic data for the study area; or (3) generalized local, regional, or national inundation depth versus percent damage functions.

A historical stage-damage curve can be developed if post-flood damage surveys have been made for several major floods which have occured in the floodplain in the past. Damages, with price-level corrections for inflation, are plotted against stage for the historical floods. Although numerous post-flood surveys have been made at various locations, adequate historical data is still not available for most floodplains. Consequently, synthetically developed damage data or generalized depth versus percent damage functions must be used for most studies. Generalized relationships between inundation depth and damage as a percent of market value have been developed for different types of damageable property. An inventory of property located in the floodplain is combined with the generalized relationships to obtain the required stage-damage function for an index location. Synthetic damage data can be developed based on estimates of damages which would be sustained by specific activities as a result of various depths of inundation.

The effects of alternative flood damage reduction measures are reflected in the computation of the basic functional relationships. Watershed management, reservoirs, and diversions modify the frequency-discharge relationships at downstream locations. Levees, flood walls, and channel improvements change the discharge-stage function. Nonstructural measures are reflected in the stage-damage function. Any change in these three basic relationships results in a corresponding change in the frequency-damage function and thus in expected annual damages.

In order to model the effects of structural flood control improvements, a series of flood hydrographs representing a broad range of magnitudes must be routed through the stream system. Each flood provides one point on the basic relationships. Hydrographs are included on each tributary at location upstream of *all* damage areas and damage reduction measures. Additional hydrographs are included at downstream locations to reflect incremental lateral inflows. The locations of the inflow hydrographs are determined based on engineering judgement considering the watershed and stream configuration and the location of damage areas and damage reduction measures. Since each flood will create one point on the frequency-damage curve which is to be numerically integrated to obtain expected annual damages, an adequate number and range of magnitude of floods are needed to properly define the frequency-damage function at each of the index damage locations.

Benefits and Losses for Conservation Purposes

Benefits for hydroelectric power can be computed by a reservoir system simulation model based on inputed primary and secondary energy values in dollars and the purchase cost for obtaining energy from an alternative source in case of a shortage in primary energy. Firm energy demands and the associated benefits are provided as input data. Secondary energy is energy in excess of firm energy which is produced by routing releases for other purposes through the turbines. Shortages are computed whenever the firm energy demands cannot be met. Cost data is provided as input for assigning dollar losses to shortages. Both the MIT Simulation Model and HEC-5 have routines for this type hydropower economic analysis.

HEC-5 has no options for computing dollar benefits for water supply. The MIT Simulation Model allows water supply benefits and also shortage versus loss functions to be provided as input data. Economic costs associated with not meeting water supply demands are determined by the model by relating computed water shortages to the inputed shortage versus loss function.

HEC-5 Simulation of Flood Control and Conservation Systems

HEC-5 was selected for use in the present study because (1) both flood control and conservation operations can be modeled, (2) pool levels can be varied seasonally, and (3) the program is well documented and readily available. The case study computations were performed either manually or using HEC-5. HEC-5 was used to simulate both flood control and conservation operations, compute expect annual flood damages, and develop firm yield versus storage relationships.

The decision to use this model was based on the need to analyze both conservation and flood control functions of multipurpose reservoirs. Many models are available which consider the aspects of conservation usage. Several are available which deal with flood control operations. Although there are a few models developed that deal with both conservation and flood control purposes, HEC-5 is the most documented, available, and accepted model for use in the study of multipurpose reservoir operations.

The "HEC-5 Simulation of Flood Control and Conservation Systems" computer program was developed and continues to be maintained by the Hydrologic Engineering Center (HEC) of the U.S. Army Corps. An initial version of the model released in 1973 has subsequently been significantly expanded.

The April 1982 version used in the present study has since been superceded by a July 1985 version. The users manual (21) provides detailed instructions for using the generalized computer program. Feldman (3) describes HEC-5 as well as several other water resources system simulation models available from the HEC.

HEC-5 simulates the operation of multipurpose, multireservoir systems. The reservoir system consists of a number of reservoirs and control points. Water demands for municipal, industrial, and/or agricultural water use, hydropower, or instream flow maintenance are specified at the reservoirs or at downstream control points. Flood control storage is operated based on flows at downstream control points. The model operates the system of reservoirs in order to best meet specified flood control and conservation requirements.

HEC-5 may be used to determine reservoir storage requirements and/or operational strategies for various water control needs. The model is also used to assist in determining reservoir releases during real-time flood control operations. Capabilities are provided for computing expected annual flood damages and hydropower benefits. A program option is also provided to determine the firm yield versus storage capacity relationship for a reservoir.

Since the program has no rainfall-runoff modeling capabilities, streamflows must be furnished as input data. The simulation may be performed using any one-hour or larger time interval. The time interval may vary during a simulation. For example, conservation operation is typically modeled with monthly flows, switching to daily or hourly flows for modeling operations during flood events. Flood routing methods incorporated in the program are Modified Puls, Muskingum, progressive average-lag, successive average lag, and working R&D. The reservoir rule curve can vary monthly. Storage in each reservoir is discreetized into levels or pools for operational control purposes. The model uses a set of operational priorities for dealing with conflicts between multiple purpose objectives and to balance storage between reservoirs.

HEC-5 is a very flexible computer model with many capabilities which make it adaptable to a variety of reservoir operation problems. The task of investigating the effect of seasonally varying operating controls for a reservoir with opposing objectives (conservation and flood control) required a computer model with many capabilities. These capabilities, or options, were necessary to compute the reservoir firm and secondary yields for seasonal and non-seasonal operating rules, expected annual damages for both types of operating plans, simulate the reservoir operation with both type plans to show flooding effects on downstream control points, use both period-of-record and critical period analysis. be able to handle seasonal demands, determine reservoir elevations for various floods, optimize storage for a given reservoir size and inflow record, and a host of other assignments. To be more specific, the following paragraphs discuss the capabilities of HEC-5 which rendered it so desirable in the case study.

HEC-5 allows the user to specify up to 15 different index levels of storage for a reservoir. This was necessary for the case study since four different levels were used: sediment, buffer, conservation, and flood control.

A study of the effects of seasonally varying operating rules would not have been possible with HEC-5 had it not been for its capability of accepting monthly changing reservoir levels. In the case study, the top of the conservation pool was changed, being down in the late fall and winter, and rising for spring and summer.

The program allows the user to specify the initial storage when computations are to begin. Beginning storage values could be very important when addressing critical drawdown periods or large floods.

Inflow forecasting capability in hours can be specified by the user if it is desirable that reservoir releases be made prior to a potentially damaging flood. The Waco Lake regulation manual specifies that releases be made if forecasts indicate that the reservoir would otherwise rise above the top of the flood control pool. This option within the program allows the computer operation of the reservoir to more closely model what actually occurs at the reservoir prior to a flooding situation.

HEC-5 has the capability of releasing water to control flooding at designated control points downstream of the dam. This capability was also desirable for the case study since Waco Lake is used in conjunction with Lake Whitney to control damaging floods at several locations downstream on the Brazos River. A damaging flow can be designated as the point above which damage occurs. This gives the program criteria to consider when making flood control releases.

Giving a maximum desired flow at each control point allows the model to operate the reservoir for flood control purposes. The program also allows input of a minimum desired as well as a minimum required flow for conservation use criteria at various downstream control points.

Literally a multitude of user defined output is available upon request. Time period data, control point data, annual data, output error check, maximum and minimum event summaries, reservoir data by period, and hydrologic efficiencies are some of the optional output choices available to the user. The user also has choice of whether or not to have hydrographs plotted, and a choice of which ones to plot.

Some of the available options for user defined output that were used in the case study included diversions, reservoir inflow, reservoir outflow, end of period storage, evaporation, elevation, diversion, shortages, percent of flood control used, and allowed top of conservation pool for seasonal operation rules as set by the user.

An option that proved useful during the system flood control simulation was the ability to compute natural flows from recorded flows (which reflected reservoir releases in addition to local flows as a part of the total recorded flow at any downstream point).

Also useful during the system flood control simulation was the ability to use flow data of different time increments. This allowed hourly intervals to be used during intense floods, daily intervals for lesser floods, and monthly intervals during normal or low flow periods, all within the same computer job run. This eliminated having to run the program separately for each flow series, and then trying to add them together to arrive at some meaningful representation of elevations and flows.

HEC-5 has an optional capability for computing expected annual damages from a series of historical flood events which reflects the seasonal timing of the floods. This is advantageous over the pattern hydrograph approach also provided as an option in HEC-5. The pattern hydrograph approach to computing expected annual damages is the only option contained in the widely used HEC-1 Flood Hydrograph Package. A pattern inflow hydrograph characteristic of those expected to enter the reservoir is given the model as input. Next, ratios are input which are used to multiply the ordinates of the pattern hydrograph. This multiplication raises or lowers the ordinates of the pattern hydrograph to give additional inflow hydrographs for use in comparing the effects these additional hydrographs have on the reservoir. This may not take as much time to input as does all the recorded hydrographs used in HEC-5, but it also does not reflect seasonal timing of the flood events.

If the user desires, reservoir shortages can be transferred from one routing set to another. For example, in the case study it was necessary to route flows with hourly increments followed by daily or monthly increments, vice versa, etc. This option allowed flexibility in the amount of work required. Separate runs for each series of flow data were not required. One run for flows of all different time intervals was all that was necessary since storages could be transferred from one series of routings to the next.

One or more reservoirs can be operated as a system by the model.

Both evaporation and water demand are allowed to vary according to the season of the year. Seasonally varying demand and evaporation greatly influence the amount of storage that would be available at different parts of the year for either water supply or flood control.

Optimization is a major feature of HEC-5 that is available to the user. The model can optimize the yield of up to six reservoirs, although only one was needed in the case study. The program is capable of optimizing reservoir yield for a given storage as well as optimizing storage for a given yield. This option can determine, from a given set of 12 monthly varying demands, the relative ratios or percentages of yearly demand which is required each month. After being supplied a storage value for which the yield is desired, the optimization feature can simulate the operation of the system for the given set of monthly demands. The minimum storage is determined, and if the value is within a specified stop criteria the program will not "optimize". However, should the given withdrawal of water not cause the reservoir to empty, or should it cause the reservoir to go dry more than once (during the set of given inflows) then the program optimizes by multiplying each monthly demand by a number to increase or decrease the total withdrawal, depending on whether the total yearly demand needs to increase or decrease. Multiplying each monthly demand by the same number preserves the ratio of that particular monthly demand to that of the yearly total. This insures that whatever yield is determined will reflect the same proportion of monthly demands to yearly total as was given by the original set of monthly water demands.

HEC-5 has the ability to sort through a given period of recorded inflows and determine the critical period for reservoir yield calculations. This option is designed to limit the amount of time required for the computer to determine required storage or maximum withdrawal for yield studies. It eliminates consideration of all non-critical periods of inflow, which is especially useful with long periods of historical data and and/or during use of the optimization routines which repetitively use the same series of flow data. Decreased computational time translates into decreased costs to the user.

As with normal simulation use of the program, the optimization portion of HEC-5 has the capability of selecting only the critical period of inflows for use in the optimization routines. This is meant to save computational time.

If it is necessary or desirable to end computations before using all the given inflow data, HEC-5 has an option available to terminate computations at a specified point before the end of the set is read.

Releases from the reservoir could be specified at any or all time periods. This was important in the case study system simulation when recorded releases were specified and routed to downstream control points for comparison of how *control point river levels compared to those resulting from seasonal regulation.*

Overall, HEC-5 is a very flexible, useable computer model with a large number of options. Many of these options were not exercised for this particular study, but are nonetheless available for further reservoir operation investigations.

CHAPTER V

CASE STUDY

The hydrologic and economic impacts of adopting a seasonal rule operation for Waco Lake was investigated as a case study. Waco Lake was selected for the case study because (1) its physical and hydrologic characteristics and operating procedures are representative of reservoirs in Texas and (2) a permanent reallocation of flood control storage capacity to water supply was recently proposed, and (3) the availability of necessary data.

This chapter presents the reasoning, the background, and the methods used in order to quantitatively study the feasibility of seasonal rule curve operation for Waco Lake. Although the flood control and yield-storage results of the case study are also included, discussion of these results will be primarily limited to Chapter 6.

In choosing Waco Lake as a case study, it was hoped that the problems associated with, the feasibility of, and the desirability of seasonal reservoir operation would be demonstrated and would be somewhat characteristic of other reservoir situations in the state to which seasonal operating rules might be applied. Waco Lake is located in Central Texas where the existing conditions should be fairly representative of many areas of the state. Existing conditions include seasonal weather patterns, reservoir size, reservoir usage, seasonal demands, constant top of conservation pool operation, etc.

Description of the Case Study Reservoir

The Waco Dam and Reservoir project was authorized by the Flood Control Act of 1954. Construction was initiated in 1956, and deliberate impoundment began in February 1965. The dam and reservoir are located entirely within the corporate limits of the City of Waco in central Texas. The dam is on the Bosque River 4.6 miles above its confluence with the Brazos River. At the top of conservation pool, the reservoir inundates the confluences of the four major tributaries of the Bosque River: North Bosque, Hog Creek, Middle Bosque, and South Bosque. The reservoir has a drainage area of 1,670 square miles. The water surface area at top of conservation pool is 7,270 acres.

Waco Dam is 24,620 feet long with a maximum height of 140 feet. The dam is an earthen embankment except for a 1,034 foot concrete gravity spillway section. The spillway is controlled by fourteen 40-feet x 35-feet tainter gates. he outlet works consists of a 20-foot diameter conduit controlled by Broome-type tractor sluice gates. Pertinent elevations in feet above mean sea level are as follows: streambed, 370 feet; top of conservation pool, 455 feet; spillway crest, 465 feet; top of tainter gates, 500 feet; maximum design water surface, 505 feet; and top of dam, 510 feet.

Project purposes are flood control, municipal and industrial water supply, and recreation. Flood control, conservation, and sediment reserve capacities are 553,300 acre-feet, 104,100 acre-feet, and 69,000 acre-feet. The 69,000 acre-feet of sediment reserve was available at the time of initial impoundment to provide for 50 years of sedimention. The Fort Worth District of the U.S. Army Corps of Engineers constructed, owns, and operates the project. Releases from the conservation pool are make at the discretion of the local project sponsors. The city of Waco and the Brazos River Authority (BRA) have contracted with the Corps of Engineers for 12.6 percent and 87.4 percent, respectively, of the conservation storage. The BRA has contracted with the city of Waco to supply the city water from BRA's 87.4 percent share of the conservation pool. Thus, all of the conservation storage in Waco Lake is committed for providing municipal and industrial water supply for the city of Waco and its suburbs.

The Corps of Engineers operates the project. Water supply releases are make as requested by the city of Waco to meet its demands. Normally no flood-control releases are make if the reservoir level is at or below the top of conservation pool, elevation 455.0 However, if flood forecasts indicate that the inflow volume will exceed the available conservation storage, flood control releases may be made if downstream conditions permit. Whenever runoff-producing rainfall occurs or a flood is in progress on the Bosque and Brazos Rivers and the reservoir is in the flood control pool, all of the gates are closed. The gates remain closed until the flow on the Bosque and Brazos Rivers has crested and receded to 50,000 cfs on the Bosque River at the Waco gage and 60,000 cfs on the Brazos River at the Waco and Richmond gages. The flood control pool is emptied as quickly as possible without exceeding these allowable downstream flow rates unless the schedule shown in Figure 7 indicates a larger release. The Figure 7 schedule controls during extreme flood events.

Waco Lake is a component of an eleven reservoir system operated by the Corps of Engineers to control flooding in the Brazos River Basin. The reservoirs are operated to maintain allowable discharges at a number of downstream control points, several of which are common to two or more reservoirs. In making releases to common control points, system operation is based on balancing the percentage full of the flood control pools in each reservoir. Waco Reservoir is operated primarily in conjunction with Whitney Reservoir which is located on the Brazos River 18 miles upstream of the Bosque River confluence. The Brazos River Basin has a drainage of 45,570 square miles of which 1,670 square miles are above Waco Dam. The Richmond gage, which serves as the most downstream control point for Waco and the other reservoirs, is over 200 miles downstream of Waco Dam.

The water supply study area consists of the city of Waco and nearby cities of Woodway, Hewitt, Robinson, and Bellmead. The city of Waco supplies water to about 32,000 municipal and industrial customers and accounts for approximately 90 percent of the water use in the study area. Waco reservoir and groundwater are currently the source of supply for the study area. However, groundwater availability is limited and rapidly declining. The cities of Woodway, Hewitt, Robinson, and Bellmead rely primarily on groundwater but are expected to need an alternate source by 1990. The present study is based on the premise that all five cities rely soley on Waco Reservoir.

The data required to perform the evaluation were obtained primarily from documents and unpublished files provided by the Fort Worth District (FWD) office of the Corps of Engineers. The Waco Lake Regulation Manual was the source for much of the data. This data included physical characteristics of the reservoir, operating procedures, monthly streamflows at the damsite for the period 1907-1970, average monthly net evaporation rates, reservoir inflow unit hydrograph, and probable maximum flood inflow hydrograph. Likewise, information required for Whitney Lake was obtained from the Whitney Lake Regulation Manual. Hydrologic records for the two reservoirs, including daily inflows, were furnished by the FWD Reservoir Control Section. U.S. Geologic

Survey streamflow records provided daily flows at six downstream gaging stations. Channel routing coefficients were taken from previous FWD studies. Discharge versus damage curves were also provided by the FWD from unpublished files. Present and projected future water demands were available from the FWD Waco Lake Reallocation Study. Water use data were also obtained from the City of Waco. This data was necessary to study the availability of water throughout the year, to investigate how flood damage potential varies throughout the year, for the computation of EAD for various plans of reservoir regulation, and for many other considerations necessary to study the feasibility of seasonal reservoir operation.

Fort Worth District Reallocation Study

In March of 1979, the Brazos River Authority, in cooperation with the City of Waco, requested that the Fort Worth District (FWD) investigate the feasibility of increasing the conservation storage capacity in Waco Reservoir to provide a greater dependable water supply yield. A subsequent study by the FWD resulted in a recommendation that 47,500 acre-feet or 8.6 percent of the flood control capacity be permanently reallocated to water supply. The reallocation would raise the top of conservation pool from elevation 455.0 feet above mean sea level to about 462.0 feet. The dependable yield of the reservoir would be increased from 54.9 mgd to about 70.0 mgd. A loss of 47,500 acre-feet of flood control capacity was estimated to reduce protection from a 100-year to about an 80-year recurrence interval design flood (21). Seasonal rule curve operation was not investigated to any significant extent in the Corps of Engineers reallocation study.

The proposed reallocation was approved by the Office of the Chief of Engineers in April 1983. The Chief of Engineers, located in Washington, D.C., has the discretionary authority to approve reallocations of not greater than 15 percent of the total storage capacity allocated to all authorized federal purposes or 50,000 acre-feet, whichever is less. Larger storage capacity reallocations in federal projects would require Congressional approval.

A contract between the Brazos River Authority (BRA) and the federal government for the Waco Reservoir reallocation was executed in September of 1984. The contract provides for the BRA to reimburse the cost for relocating recreation facilities plus the allocated value of the water supply storage. The next step in the process is for BRA to provide funds in an escrow account. The FWD will then relocate the recreation facilities as required and impound water in accordance with the rainsed top of conservation pool elevation.

The FWD reallocation study report includes: an analysis of present and projected future water demands; formulation of alternative strategies for meeting the water needs; selection of the storage reallocation plan; environmental impact assessment; cost estimate for implementing the recommended plan; allocation of costs between the federal government and Brazos River Authority; and a draft contract between the federal government and Brazos River Authority for use of the additional conservation storage and repayment of associated costs (21). The effects on reservoir performance of the proposed capacity reallocation were evaluated primarily in terms of firm yield and the recurrence interval of the design flood which could be contained by the flood control pool.

As can be seen by the Corps' reallocation study, the positive effect of increasing the top of the conservation pool is increased yield from Waco Lake. This is offset somewhat by the decrease in flood protection for the City of Waco which is directly downstream of Waco Lake, as well as all other points downstream to which Waco Lake is operated for the reduction of flood damages. This type of tradeoff is necessary when the elevation delineating the bottom of the flood control pool is raised, to be kept constant all year.

Development of a Modeling Strategy

An overall approach and detailed techniques were developed to evaluate whether or not seasonal deviations in the operating policy of an existing reservoir are worthwhile. The approach consists of formulating alternative operating plans and simulating their performance. The measures of system performance includes water supply yield and reduction in expected annual flood damages. The procedure quantifies the tradeoffs between providing water supply and reducing flood damages. Unlike traditional evaluation procedures, the proposed modeling strategy reflects seasonal variations in flood threat, damage potential, water demand, and water availability. The Corps of Engineers computer program "HEC-5 Simulation of Flood Control and Conservation Systems" was used for the hydrologic and economic simulations.

Yield Study

One of the steps taken to evaluate the feasibility of seasonal operation was the determination of firm yield versus storage capacity (and elevation) relationships for Waco Lake using the entire period of monthly inflow to the reservoir. These relationships were developed to show how the firm yield increases as the top of the conservation pool is raised. Initially, none of these firm yield determinations were based on seasonally changing the top of the conservation pool. Later on in the yield study, after the yield-storage relationships had been established for constant top of conservation pool elevations, the firm yield corresponding to seasonally varied top of conservation pools were determined.

Constant Pool

Five different curves were developed for the portion of the yield study which corresponds to constant top of conservation pool operation. Each curve was based on the same period of recorded inflows, storage vs. elevation relationships and other reservoir characteristics. However, each one differed from the other four by one or more of the following items: sedimentation, evaporation, and water demand. The firm yield versus capacity relationship developed by the Corps of Engineers (21) is included as a sixth curve in Figures 15 and 16 for purposes of comparison. These figures present the results of the yield study for constant top of conservation pool elevations.

The evaporation rates used in curves one through five represent average monthly evaporation from Waco Lake based on average monthly pan evaporation recorded at the damsite. Curves three and five subtracted out the average monthly precipitation at the damsite for computation of net monthly evaporation used in the yield studies. However, the Corps of Engineers used recorded monthly evaporation and rainfall (not average values) to compute net monthly evaporation for curve 6 instead of the average values used in curves one through five. Table 15 shows how each curve of yield versus storage (or elevation) differed from the others in the areas of sedimentation, evaporation, and water demand; the key parameters of the yield study. Ratios of monthly demand to yearly demand were estimated from actual monthly water use as recorded by the City of Waco. Table 16 shows these ratios.

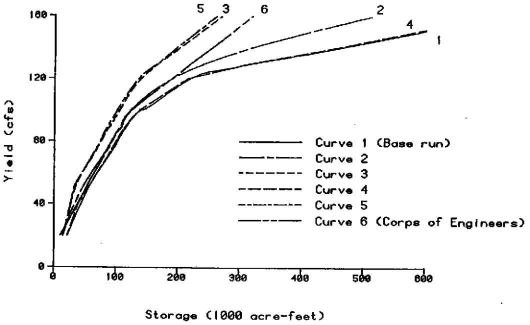


Figure 15.

Yield-Storage Relationship for Waco Lake

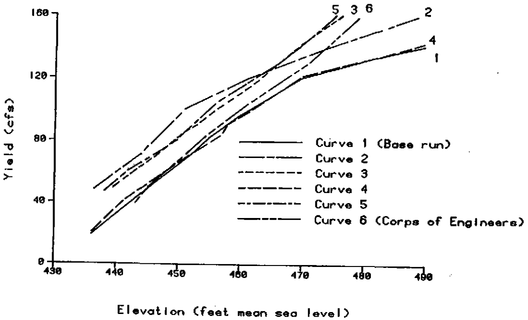


Figure 16.
Elevation-Storage Relationship for Waco Lake

Table 15.
Yield Study Curve Conditions

Curve	Sediment Conditions	Evaporation : Precipitation	Water Demand
1	50 - Year	Evap. Only	Seasonal
2	None	Evap. Only	Seasonal
3	50 - Year	Precip. Only	Seasonal
4	50 - Year	Evap. Only	Constant
5	50 - Year	Precip. Only	Constant
6	50 - Year	Some of Both	Constant

Table 16.

Average Monthly Demands by the City of Waco

Month	Monthly Water Use as a Percentage of Annual Water Use
January	6.58
February	6.16
March	6.41
April	6.96
May	7.93
June	9.55
July	11.51
August	11.68
September	10.29
October	8.53
November	7.27
December	7.13

In an effort to maintain consistency for the sake of comparison, the same ratios were used in each simulation. Firm yields are expressed in terms of average annual demand. Annual demand was multiplied by each one of the 12 monthly ratios expressed as a fraction to obtain the demand for that month that was in proportion to the amount actually used by Waco during that month. Therefore the same percentage of use per month of the year propagated throughout the yield study, regardless of the total demand or amount of storage being investigated.

Curve number one represented the most conservative combination of conditions. This was referred to as the "base run". Fifty year sediment deposition taking up volume in the conservation pool, evaporation from the reservoir surface without consideration of precipitation which fell directly on the reservoir surface, and seasonal demand as opposed to constant demand represented the most conservative conditions of the six different combinations used for the yield study.

Curves three and five used monthly evaporation as well as all of the precipitation falling directly on the reservoir when computing the total loss (and in some cases gain) of water from the reservoir surface. Curve six used only a portion of the precipitation falling on the reservoir. The reason that rainfall is typically included in net evaporation is to reflect the rainfall that reaches the reservoir that would have been abstracted prior to reaching the stream before the reservoir was built.

All six of the curves did not consider seepage in the yield calculations. It is very difficult to measure seepage throughout the entire reservoir, or to know whether there is a net inflow due to high water tables, net losses due to permeable bottom soils, or net losses due to fractured rock formations within the reservoir. In addition, no attempts at measuring seepage were made in the years since the reservoir was constructed. Seepage has been recognized as a possible major factor in the amount of water available from a reservoir. It is possible that incoming sediment has a "grouting" or sealing effect on the reservoir bottom and/or sides precluding some of the seepage that might otherwise take place. However, more work needs to be done in this area to be able to quantify and predict net water movements due to seepage.

To determine the various points on the six graphs, the simulation model was run to determine the yield which corresponded to each of the different top of conservation storage levels for each of the five combinations of conditions shown in Table 15 The sixth set of conditions had already been simulated by the Corps of Engineers.

HEC-5 input for these runs included starting time for inflows, definition of inactive (sediment storage), buffer, conservation, and flood control pools; user defined output indications, optimization requirements, starting storage levels, reservoir characteristics (area, elevation, storage), requirements, inflow, the aforementioned ratios of monthly demand to yearly demand for seasonal usage, and evaporation data.

The data listed in the previous paragraph was used as input for HEC-5 during the yield versus storage analysis portion of the case study. It was used for both the constant and seasonal top of conservation pool elevation. The simulation was run on a trial-and-error basis: a yield was selected for which the required storage was desired. The proper combination of conditions (see Table 15) for whatever curve was being investigated was input with the rest of the pertinent data. A top of conservation pool elevation was input to the model and the simulation was run. If the storage pool elevation which was used in the simulation was deemed the minimum amount necessary to maintain that demand throughout the period of recorded inflows, it thus became a point for the curve representing the yield-storage (or elevation) relation for the particular set of conditions being investigated. If however, the minimum storage went far negative, then it was determined that the top of conservation storage pool elevation that was input to the model was inadequate to maintain the given demand, and was increased for the next simulation run of the model. On the other hand, if the minimum storage stayed positive and did not approach zero, then the top of conservation pool given the model during that run was deemed larger than really necessary to maintain that continuous release from the reservoir, and was subsequently decreased for the next simulation run of the model. This process was repeated for each point on the curve until near-zero minimum storage was attained.

Discussion concerning the results of the yield study for the constant top of conservation pool operating plan is contained in the next chapter.

Seasonal Pool

The second major portion of the yield-storage study concerned the firm yield available from seasonal operating plans as compared to plans which kept the elevation delineating the top of conservation storage constant all year.

This study was very similar to the yield-storage study done for constant operating policies. The same trial-and-error approach was used as previously defined. Much of the same input data was used. However, there were two major differences involved between the yield study of the constant plans and that of the seasonal policies.

One difference was the fact that only curve one (base run) conditions were used for the seasonal yield study: 50 year sediment conditions, evaporation without precipitation, and seasonal demand. None of the other combinations of conditions were used. The other difference, of course, was that the elevation delineating the top of the conservation pool varied with the time of year for the seasonal plan.

However, it was not much trouble to continue with the yield study for the seasonal operating policies because of the ease at which HEC-5 was changed to reflect seasonal operating policies as opposed to constant.

As previously mentioned in Chapter 4, HEC-5 is a very flexible model. The only changes necessary to continue this yield-storage study for seasonal operation, were to change one card and add four others. This added to the simulation runs of the model the seasonal dimension changing the elevation delineating the top of the conservation pool.

The results of the yield study for the seasonally varying operating plan are shown in Table 17. To aid in comparing these results with those from the constant policy yield study, those results have been duplicated below.

As can be seen from the above table, seasonal operation gives almost precisely the same firm yield as does constant operation. This fact is supported by efforts described in the next few pages and discussed in the next chapter.

Reservoir Reliability

Another major portion of the yield study was an investigation of reservoir reliability. This task was undertaken in an effort to quantify the ability of seasonal and non-seasonal operating policies to deliver various amounts of water. The results of this portion of the study indicate, based on the simulation

Table 17.

Firm Yield for Constant and Seasonal Rule Curves

Type of Operation	Top of Conservation Pool (feet)		Firm Yield (cfs)
	Nov-Mar	Apr-Oct	
constant	455	455	81
constant	462	462	102
constant	465	465	108
constant	475	475	127
seasonal	455	462	102
seasonal	455	465	108
seasonal	455	475	126

of the recorded inflows, the percentage of time in which a certain operating policy was able (or unable) to meet different release rates. The same results show the percentage of time in which the different operating policies had minimum annual storages less than or equal to various reservoir conservation storage levels for different release values.

The method used in the reliability analysis portion of the yield study involved determining the minimum monthly storage for each year of the period of record (75 years). This was done for each combination of operating plan and average yearly release. Each yearly minimum storage was then written down and the entire 75 values were consecutively ordered with the largest minimum yearly storage as number one and the smallest at 75. The equation below used this ranking (1 through 75) to express the percentage of years that the reservoir minimum storage was less than or equal to the level of storage associated with that ranking.

$$(75\text{-Ranking} + 1)/75$$

Since there was a percentage associated with each of the 75 minimum annual storages, and 75 of these percentages associated with each operating plan, it might be easier to compare the percent of years that the reservoir minimum storage was less than or equal to different storage levels through means of graphs. Figures 17, 18, and 19 show the storage exceedance frequency relation for drafts of 102, 108, and 127 cfs respectively. These drafts correspond to constant top of conservation plans with pool levels of 462, 465, and 475 feet. The values of the y-axis represent the minimum annual storage. The x-axis represents the exceedance frequency in percent.

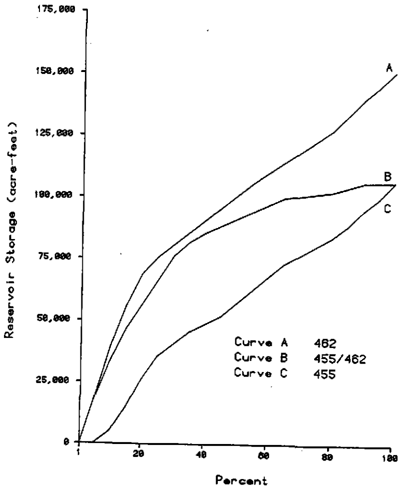


Figure 17.
Storage Exceedance Frequency Relation-Draft = 102 cfs

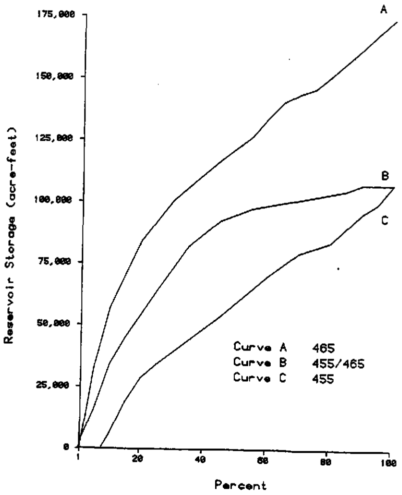


Figure 18.
Storage Exceedance Frequency Relation-Draft = 108 cfs

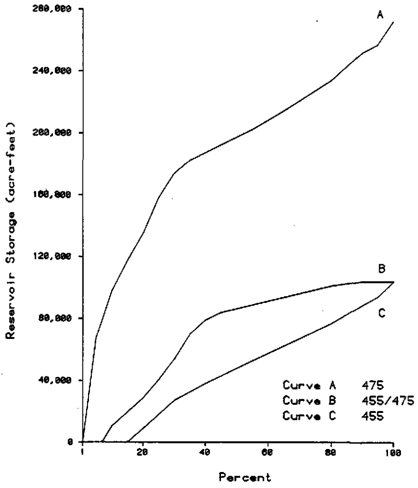


Figure 19.
Storage Exceedance Frequency Relation-Draft = 127 cfs

However, the failure rates are a little easier to compare and are therefore provided in Table 18 (The failures are correct at 1.33 instead of 1 percent because there were not 100 years of data, only 75). This table provides a more complete numerical review of the performance (reliability or failure) of various plans at meeting various levels of demands.

Notice that the percentage of time that a certain plan failed to meet different demands either stayed the same or increased as demand rose. For example, the 455 constant plan had about 7 percent failures at 102 cfs demand, 8 at 108, and 15 at 127. The 455/465 seasonal plan had about 1 percent failure rate for 102 and 108 cfs of demand, and increased to about 7 for a demand of 127 cfs. This finding turned out as expected: after a certain point of demand (the firm yield associated with that plan), failure of an operating plan to certain demands will increase as the magnitude of the demand increases.

As mentioned earlier,seasonal operation shows the capability of providing almost the same release through the critical period as does constant operation. This finding is backed up by the reliability study showing that seasonal operation can deliver essentially the same firm yield as constant regulation with almost the same degree of reliability. This finding, with related explanations and discussions is covered in Chapter 6.

Flood Control

The next step undertaken in the investigation of seasonal reservoir operation dealt with damages due to flooding. Whereas reservoir yield dealt with amounts of water over a long period of time, flooding deals with large amounts of water which arrive in a short period of time, often 3 days or less. Sometimes the period of flooding is very short, measured in a few hours in

Table 18.

Percentage of Years Various Operating Policies Were Unable to Deliver Various Demands

Operating Plan	Constant Yearly Release (cfs)	Percent Failure
455	102	6.67
455/462	102	1.33
462	102	1.33
455	108	8.00
455/465	108	1.33
465	108	1.33
455	127	14.67
455/475	127	6.67
475	127	1.33

some cases.

To compare the benefits of flood control due to seasonal reservoir operation it was necessary to compute the flood damages resulting from the constant and seasonal operating plans. These damages were calculated for all downstream points to which the reservoirs were operated with the objective of reducing flooding at these points. These points are referred to as "control points" in the rest of this report.

Waco Lake is operated conjunctively with Lake Whitney to decrease flood damages downstream on the Brazos River. Lake Whitney is a major impoundment on the Brazos River north of Waco. Although Waco Lake is on the Bosque River, not directly on the Brazos, it is operated to control the flows from the Bosque which empty empty into the Brazos 4.6 miles from the dam site.

Expected Annual Damages

A comparison of present-day versus seasonal operating policies as well as alternative constant levels requires the computation of flood damages caused by each operating policy. A common means of expressing flood damage is in terms of Expected Annual Damages (EAD). The damage reduction (benefit) accrued due to the implementation of seasonal reservoir operation is determined by computing the difference between damage values occuring in a river basin with and without seasonal measures. Damage is assumed to be a function of peak discharge or stage and does not usually depend upon the duration of flooding. A flood event is assumed to have a fixed exceedance frequency that is a unique function of maximum discharge. Total damage is determined by summing the damage computed for individual damages reaches within the river basin. The damage in each reach is calculated as the sum of damage for individual land use categories such as agricultural, commercial, industrial, etc. (20).

One of the first major steps in determining EAD was the calculation of a discharge versus exceedance frequency relation for each of the downstream control points. This was accomplished by selecting the instantaneous flood flow peaks for each year from the United States Geological Survey streamflow records for each of the control points. The mean and standard deviation were computed for these annual peak discharge series. The Gumbel distribution of extreme values was used to develop an exceedence frequency versus peak discharge relationship for each control point.

However, not all of the peak flow rates were available or usable in their present form. For example, instantaneous peak flow rates were available from 1951 to 1982 for the Brazos River at Waco. However, Waco Lake was not constructed in its present form until 1965. (The former Waco Lake had been constructed years earlier with its dam being about one-half mile upstream of the present dam. It was inundated in 1965 after completion of the present reservoir). This left the first fourteen years of data not showing the effects of flow regulation by Waco Lake as it is presently being carried out.

Computation of exceedance probabilities with data from before and after the present reservoir was considered unacceptable. Therefore the first fourteen years of flow data at Waco on the Brazos were not used. Although using the first fourteen along with the last eighteen years of data would have given a longer period of record (which is very desirable when making decisions based on historical records), having regulated and non-regulated flows within the same record would have greatly lessened confidence in the outcome of any computations which were based on that period of record.

Realizing the importance of having a long period of record upon which to base computations, efforts were made to modify the flow records at Waco on the Brazos in an attempt to make the first fourteen years of flow data reflect present operating policies at Waco Lake.

Flow data from the Clifton gage on the Bosque River upstream of Waco and the Whitney gage on the Brazos were used in this effort. The flows from these two stations were combined and plotted against the post-1965 flows on the Brazos. This was done to see if a pattern or relationship emerged which could be utilized to alter the pre-1965 flows to better reflect present operating policy. However, this effort did not prove to be beneficial in this regard. It did serve as a basis on which to decide to use only the last eighteen years of available data.

Another major step in the computation of expected annual damages due to flooding was the computation of a discharge versus damage function for each downstream control point. Fortunately, this relationship had already been developed by the Corps of Engineers Fort Worth District Office. These points included the Brazos River at Waco, High Bank, Valley Junction, Washington-on-the-Brazos, Hempstead, and Richmond. The damage discharge relations at these points included non-agricultural and two agricultural categories.

The third major step in the computation of expected annual damages, after determination of discharge frequency at the downstream control points and determination of a discharge damage relation at these points, was a simulation of the two-reservoir system. This simulation was carried out with HEC-5 using the 35 years of recorded flow data available since the construction of Lake Whitney. Waco Lake enters the computations in 1965 representing the completion date of its construction. Simulation of the operation of Lake Whitney (and Waco Lake after 1965) was carried out by routing the recorded reservoir discharges to the downstream control points. The resulting hydrograph was then subtracted from the recorded hydrographs at those points to determine the hydrographs of lateral inflow (which result from the streamflow contribution of areas not controlled by the dam). The routing coefficients used in this simulation came directly from Corps of Engineers data.

The system simulation was run again with different tops of conservation pool levels, for both constant and seasonal operating plans. These levels were 462, 465, and 470. This was done to show the increase in EAD that should result from an increase in the top of conservation pool with its corresponding decrease in flood control pool volume. The seasonal operating policies were 455/462, 455/470, and 462/470. Each simulation started with a full conservation pool, and routed the 35 years of recorded inflow data through the reservoirs past each control point to determine the resulting hydrographs.

All inflows which caused the reservoir to go past the top of the conservation pool were released as quickly as possible (as long as damaging flows were not already going on downstream) without exceeding the non-damaging channel capacity at the control points. To the resulting hydrographs were added the previously determined hydrographs of lateral inflow that occured at that control point and date. Adding the routed and the lateral inflow hydrographs together gave the total hydrograph that should have occured at that point if the identical rainfall conditions had occured with the proposed higher top of conservation pool levels.

The total hydrograph peaks resulting from routing the recorded flows from the reservoirs to the control points resulting from the present and proposed top of conservation pool levels were taken to the damage-discharge relation at that point. Each discharge was also assigned an exceedance probability from the previously determined discharge-exceedance probability relationship. The peak flow rate of each of the 7 major floods of record for the 35 years since Lake Whitney went into operation was assigned a discharge frequency which corresponded to the same flood which occured during present operation with the top of conservation pool at 455. For example, if a particular storm caused a peak discharge at Waco of 39,000 cfs when the top of conservation pool was set at 455, it would be assigned an exceedance probability of 0.10. If the same storm caused a peak flow of 46,000 cfs for the top of conservation pool at 465, the same exceedance probability would be assigned to that discharge. The same exceedance probability was assigned to the peak discharge of each top of conservation pool resulting from the same storm. This is due to the fact that the cause of the flow rate at any given point is dependent upon the storm over the reservoir's watershed. These storms would have occured regardless of the elevation set as the top of the conservation pool.

After the discharges were assigned a probability, the damage and its associated exceedance probability were plotted as a point on a graph of damage versus exceedance probability. This procedure was repeated for each top of conservation pool level. EAD in turn, was determined by calculating the area under the damage-exceedance probability curve. EAD was determined for each different top of conservation pool by this method. Table 19 shows how the expected annual damage changes for various operating policies.

Although Lake Whitney became operational in 1956, Waco Lake did not become operational until 1965. Therefore, flows before 1965 did not reflect regulation by Waco Lake, only by Whitney.

Table 19.

Expected Annual Damages

Top of Conservation Pool Elevation November - March	April - October	Expected Annual Damages ($1,000)
455	455	4,685
462	462	4,795
470	470	5,005
455	470	3,213
462	470	3,875
455	462	4,184

Reservoir Effects on Statistical and Hypothetical Inflow Hydrographs

The expected annual damage simulation studies were supplemented with an analysis of the effects of various top of conservation pool elevations on a series of hydrographs routed through the reservoir. Statistical inflow hydrographs associated with a range of return intervals and the probable maximum flood were used in the analysis. Although this information was not used in the expected annual damage computations and does not reflect seasonal operating policies, it does significantly contribute toward meaningfully displaying the hydrologic impacts of storage reallocations.

The following simplified approach was followed in developing statistical inflow hydrographs. A 46-year annual series of peak discharges was assembled from several sources and fitted to a log-Pearson Type III probability distribution to develop a peak discharge versus exceedance frequency function. The reservoir inflow unit hydrograph was obtained from the Waco Lake Regulation Manual. For a given frequency, the ratio of peak discharge divided by unit hydrograph peak discharge was computed. This ratio was then multiplied by the unit hydrograph ordinates to obtain the reservoir inflow hydrograph associated with the specified exceedence frequency.

The inflow hydrographs were routed through the reservoir manually using the regulation schedule shown in Figure 7 to determine the release rate for a given reservoir water surface elevation and inflow rate. The water surface was assumed to be at the top of conservation pool at the beginning of each flood. The spillway and outlet works gates were assumed to remain closed until releases were indicated by the Figure 7 regulation schedule. Thus, the computed peak reservoir water surface elevations and outflows are independent of downstream flooding conditions. The probable maximum flood taken from the

Regulation Manual was routed through the reservoir by the same procedure. The results of a series of routings for several assumed top of conservation pool elevations are presented in Tables 20 and 21.

The flood control storage capacity was also quantified in terms of the exceedence frequency or recurrence interval of a design flood which just fills the flood control pool without overflowing. This is the return interval of the reservoir inflow hydrograph which has a total runoff volume equal to the flood control storage capacity of the reservoir. The design recurrence interval, as thus defined, is tabulated in Table 22 for a range of alternative storage allocations.

As illustrated in Table 20, a storage capacity reallocation has little effect on peak outflows for smaller floods which are contained without exceeding the flood control capacity and on extreme events approaching the probable maximum flood (PMF). Table 20 shows that raising the top of conservation pool elevation from 455 to 470 feet increases the peak outflow for the 200-year flood from 98,000 cfs to 224,000 cfs. As indicated in Table 22, with the top of conservation pool at elevation 455 feet, the 109-year flood just fills the flood control pool. Raising the top of conservation pool to elevation 470 feet reduces the flood control capacity from a 109-year to a 57-year recurrence interval design flood.

Table 20.

Peak Outflows for Statistical and PMF Hydrographs

Return Interval (years)	Peak Inflow (1000 cfs)	Peak Outflow (1,000 cfs) for Top of Conservation Pool Elevation (feet)		
		455	462	470
50	160	0	0	0
75	187	0	0	30
100	208	0	18	74
125	226	14	44	111
150	241	36	100	148
200	264	98	130	224
250	284	100	182	248
300	300	174	198	260
400	327	188	252	280
PMF	623	570	572	574

Table 21.

Peak Water Surface Elevation for Statistical and PMF Hydrographs

Return Interval (years)	Peak Water Surface Elevation for Top of Conservation Pool Elevation (feet)		
	455	462	470
25	485	488	492
50	492	494	498
75	494	497	500
100	497	500	501
125	500	501	501
150	500	501	501
200	501	501	501
250	501	501	501
300	501	501	501
400	501	501	501
PMF	504	504	504

Table 22.

Non-Discharging Design Intervals

Top of Conservation Pool Elevation (feet)	Water Supply Capacity (acre-feet)	Flood Control Capacity (acre-feet)	Design Return Interval (years)
447	62,000	595,400	113
451	80,000	577,400	111
455	104,100	553,300	109
462	147,500	509,900	84
470	220,000	437,400	57

CHAPTER VI

DISCUSSION

One purpose of this chapter is to go into detail concerning the results and findings of both yield and flood control aspects of the case study. The other is to expound upon the ramifications or influences that these results will possibly have upon those individuals which are affected by multipurpose reservoirs in Texas. This portion of the discussion will not be centered as such upon the methods or findings per se. These were covered in previous chapters. Instead, the attention will be focused on what these findings mean, and explanations as to why the findings turned out as they did.

Discussion of Case Study Results

Yield Study

Addressed in previous chapters and sections of this text have been the firm yield computations and the associated results. The Case Study (Chapter 5) issued the specifics of this portion of the research, and gave the resulting findings, as they applied to various constant and seasonal operating policies using different top of conservation pool elevations for Waco Lake.

The beginning of a discussion concerning the results of the yield study probably should center around the yield versus storage and yield versus elevation curves previously presented, Figures 15 and 16. As can be seen in these figures, the curve with the most conservative assumptions (base run) gave the lowest yield for each top of conservation storage or elevation value. Curve four however, was almost as conservative. The reader may refer back to Table 15 to recall that the conditions for curves 1 and 4 are identical except that 4 has no seasonal demand as does curve 1. This may lead to the conclusion that seasonal demands are not much more detrimental to the overall yield than non-seasonal demands that have the same yearly total demands for the same set of circumstances (location, inflow, evaporation, etc). which are particular to Waco Lake. Curves 1 and 4 seem to take this statement a step farther by saying that the magnitude of the yearly total demand does not adversely affect the relationship between the two curves. In other words, as the yield increases, the two curves maintain their position with respect to each other.

Curves three and five tend to support this conclusion also. Curve three with non-seasonal demands shows slightly higher yields for the same amount of storage used to construct curve five with seasonal water demand. Throughout the two curves, there is only a three to five percent difference in the required storage for any one yield.

Including the precipitation which fell directly on the reservoir surface seemed to be an important factor influencing yield. Curves three and five included all precipitation whereas six only included a portion of the precipitation. Curves 1, 2, and 4 included none. It can be seen that curves 3 and 5 with the added precipitation and six with some added precipitation give higher yields, especially for the higher pool elevations, than do curves 1, 2, and 4.

Perhaps this is due to the fact that as the storage and elevation increase, so does the surface area and elevation, as is shown in Figure 16. Area increases exponentially with an linear increase in elevation. Since evaporation is tied directly to surface area, as the storage and elevation increase, then so does the evaporation. That is why the curves 3 and 5 move so far away from curves 1, 2, and 4 at higher storage and elevation values. Precipitation somewhat makes up for the loss of water due to evaporation, and therefore curves 3 and 5 can sustain a higher yield than can curves 1, 2, and 4.

Although curves 1 and 3 through 6 maintained the same relative positions with respect to each other on the elevation versus yield graph as they did in the storage versus yield graph, curve 2 did not. This is because all of the other curves except 2 had the same storage relationships: 50 years of sediment deposition had reduced the amount of available storage and increased the elevation required for all values of storage. Curve 2 did not account for the sediment deposition and had more volume at the same elevation than did the curves. Otherwise, curve 2 would appear out of place with respect to the other curves if it were not for this fact.

The amounts of sediment in the reservoir which were used to determine the yield versus storage relationships were obtained from the Waco Lake Reservoir Regulation Manual (19). The manual calls for the deposition of 48,400 acre-feet of sediment in the conservation pool, with 14,400 below 427 feet elevation, and 34,000 between 427 and 455. In addition to the 48,400 acre-feet of sediment storage in the 152,500 acre-foot conservation pool, there is a volume of 20,600 acre-feet of sediment that is expected to be deposited in the 553,360 acre-foot flood control pool.

A curve of initial and fifty year elevation versus storage relations was provided by the Corps of Engineers in the Waco Lake manual. These curves were reproduced in Figure 20. As the study turned out, it did not seem to matter that curves one through five used average monthly evaporation and rainfall data, and that the curve produced by the Corps of Engineers used actual instead average data. Curve 6 fell right in between the other five curves.

Curves one and five seemed to be the extremes, giving the least and most yield respectively. The Corps of Engineers' curve plotted in the middle of

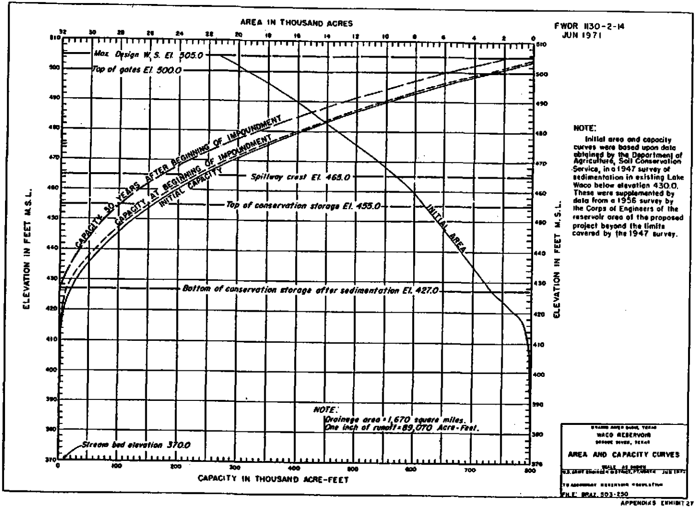


Figure 20.
Waco Lake Area and Capacity Curves

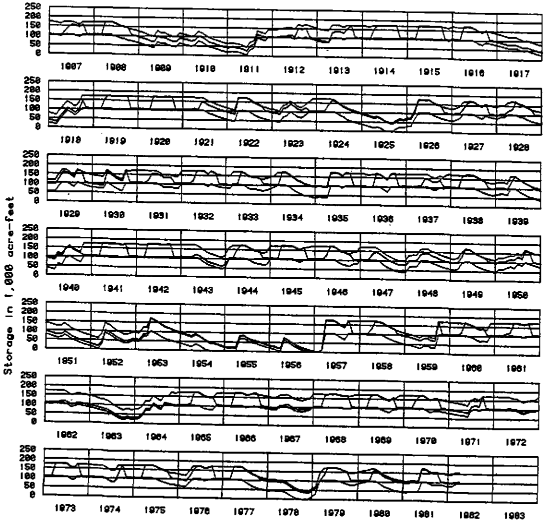
these two extremes. Curves four and one plotted closely with six for the first 120,000 acre-feet of required conservation storage. For the first 120,000 acre-feet of required storage there seems to be no more than a 20 percent difference between the six curves. However, past about 120,000 acre-feet curves 3 and 5 show seem to separate quickly from 1, 2, and 4 with 6 staying in the middle.

Keep in mind that Figures 15 and 16 were developed for constant top of conservation pools only. However, the findings of the yield study for seasonally varied top of conservation pools and the reliability study seem to say that the yield-storage and yield-elevation curves that worked for constant pool regulation will work just as well for seasonal pool regulation.

This unexpected finding is supported when a definition of firm yield is closely studied in terms of the critical period of inflow for Waco Lake. Firm yield is the greatest amount of water that can be withdrawn from a reservoir such that the sum of the inflows and outflows (mass balance) due to releases, evaporation, inflows, etc., causes the reservoir storage to just reach zero during the worst period of record. The firm yield is the rate of water which can be continuously withdrawn from a reservoir without making it go dry but once. The critical period extends from the time of a full conservation pool, through the point of declining and eventual zero storage, up to the next point in time when the reservoir conservation pool is full again.

In computing firm yield based on historical streamflow records, the value obtained for firm yield is controlled by a critical drought period. Figures 21 and 22 compare the 455/465-feet seasonal rule curve with constant top of conservation pool elevations of 455 and 465 feet.

An average draft of 108 cfs, varying monthly in accordance with Table 16, was included in each of three simulations. The 108 cfs average draft is



Simulations for a draft of 108 cfs (69.8 mgd) and alternative top of conservation pool elevations of 455, 455/465 seasonal rule curve, and 465 feet.

Figure 21.
1907 to 1981 Reservoir Simulation-Draft = 108 cfs

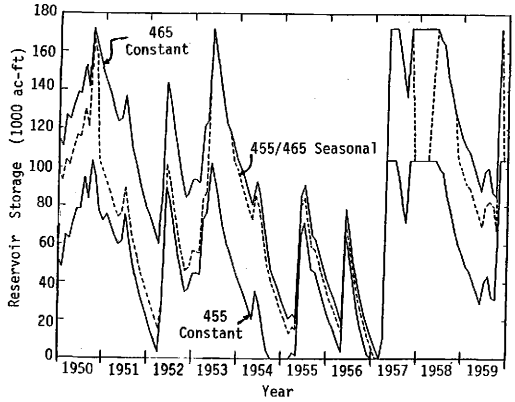


Figure 22.

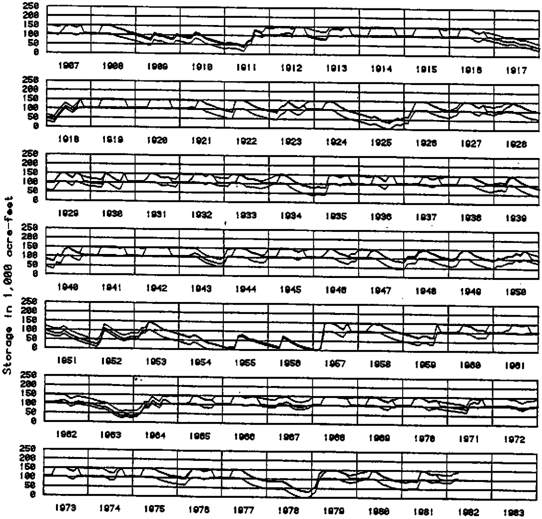
1950 to 1959 Reservoir Simulation-Draft = 108 cfs

equal to the firm yield of both the 465-feet and 455/465-feet operating policies. Figure 21 is a plot of reservoir storage levels versus time computed for the entire period of record. A more detailed plot of storage levels during a series of years encompassing the critical period is presented in Figure 22.

As shown graphically in Figure 22 with either the permanent 465-feet or seasonal 455/465 seasonal operating policies, the reservoir is full, with 172,500 acre-feet of water in storage, at the end of May, 1953. The 465-feet and 455/465 feet operating policies result in spillages of 1,026 and 438 acre-feet, respectively, during the month of May. The critical period drawdown begins in June 1953 with the reservoir empty in February of 1957 and full again in April of 1957. In November of 1953 the seasonal rule curve necessitated spilling 143 acre-feet to lower the water surface to elevation 455 feet. Thus, the reservoir storage levels are different for the 465-feet and 455/465 operating policies during the critical drawdown period. However, the difference is so small that the computed firm yields are the same. The reservoir is essentially refilled during the one month of April 1957, with a little refilling during March.

Storage levels resulting from operating at a permanent top of conservation pool elevation of 455 feet and draft of 108 cfs is shown in Figure 21. The reservoir failed to meet demands due to being empty several months in 1925, 1954, 1955, 1957, and 1978.

Raising the top of conservation pool from elevation 455 feet to 462 feet seasonally or permanently results in precisely identical increases in firm yield. The seasonal rule curve firm yield is identical to the permanent reallocation as long as long as the pool is raised no later than early May and lowered no earlier than late September. Figure 23 is similar to Figure 21 except the maximum conservation pool elevation is 462 feet instead 465. However, the same



Simulation for a draft of 102 cfs (65.9 mgd) and alternative top of conservation pool elevations of 455, 455/462 seasonal rule curve, and 462 feet.

Figure 23.
1907 to 1981 Reservoir Simulation-Draft = 102 cfs

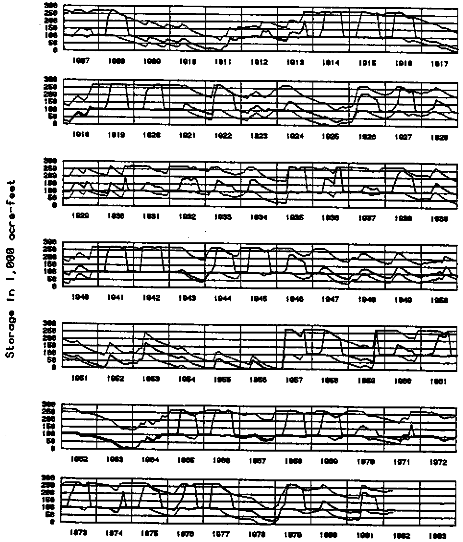
conclusions can be drawn from it as from the other graph.

Simulations of the 455/462-feet and 462-feet alternatives using a draft of 102 cfs results in the same May of 1953 to April of 1957 critical period discussed in previous paragraphs. With either the 462 constant or the 455/462 seasonal operating policies, the reservoir is full at the end of May, 1953 with a storage of 148,000 acre-feet. The 462 and the 455/462 result in spillages of 1,298 and 979 acre-feet, respectively, in May.

The critical period drawdown begins in June, 1953 with the reservoir being essentially empty in February of 1957 and full again in April of 1957. The storage level drops below elevation 455 feet (104,100 acre-feet) during September of 1953 and does not reach this level again until April of 1957. The reservoir is essentially refilled during March. From May of 1953 to past April of 1957, the reservoir storage levels are identical for either the seasonal rule curve or permanent reallocation. Thus, the firm yields are identical.

A similar comparison of a 455/475-feet rule curve operation with a 475-feet permanent reallocation indicates the firm yields are almost the same. The seasonal rule curve consisted of raising the top of conservation pool to 475 feet from April to October. A draft of 127 cfs was used in simulating the two alternative top of conservation policies. See Figure 24.

The permanent 475-feet policy results in a full reservoir (270,000 acre-feet) in June of 1946 which begins to empty in July and does not fill again until April of 1957. The reservoir is essentially empty in February of 1957. For the 455/475 seasonal policy, the reservoir is full in July of 1945, begins to empty in August and is full again in April of 1957. The reservoir is empty in March of 1952, February and April of 1955, and October of 1956 through February of 1957. The 455/475 seasonal policy also results in the reservoir being



Simulations for a draft of 127 cfs (82.1 mgd) and alternative top of conservation pool elevations of 455, 455/475 seasonal rule curve, and 475 feet.

Figure 24.
1907 to 1981 Reservoir Simulation-Draft = 127 cfs

essentially empty during one month, while the seasonal rule curve results in the reservoir emptying several times during the hypothetical period of record simulation. However, the computed firm yields are practically the same.

As can be seen by the previous discussion concerning the effects of seasonal and constant reservoir operation on the yield during the critical period, there is not many "carry-over" benefits where water availability is concerned associated with either type of plan. In other words, all water that enters the reservoir which causes it to fill above the top of the conservation pool, whether the conservation pool be seasonal or constant, is released downstream in order to lower the flood control pool to the top of the conservation pool. All of the water that rises above the top of the conservation pool is released and is no longer available for use during the critical period. None of this water is "carried over" for beneficial use at a later date.

This fact is brought out upon further examination of Figures 21 and 22. Plenty of inflow was available in 1953 to fill the seasonal 455/465 and the constant 465 conservation pools. In fact, more water was available than was necessary, and had to be released. This release was of no value, as far as conservation usage was concerned, during the critical period.

However, when critical period is not being considered, the 465 constant plan "carries over" more water year to year than does the 455/465 seasonal plan of operation. The upper *line* of *Figure 21 shows* that the constant plan has significantly more water in storage during non-critical time periods for the months of November through March. This extra water is then available for use over and above that which would be available from the seasonal plan during the same period. The arguments set forth in the last three paragraphs easily apply to the 455/462 and 462 plans as they do to the 455/465 and 465, as was discussed.

Although seasonal rule curve operations can achieve essentially the same increases in firm yield as corresponding permanent reallocations, pool levels will tend to be lower under noncritical or more normal *conditions* of rainfall and streamflow. Permanent reallocation of flood control space for use as conservation storage allows more water to be available during November through March. These are the months during which the seasonal pool would be lowered for flood control purposes. Therefore, although seasonal control offers the same yield as constant control (due to control by the critical period), it does not offer additional water during the winter months as does the constant plan.

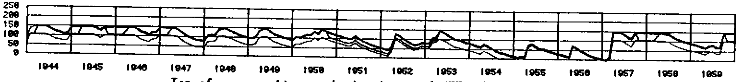
For example, Table 18 shows that although 455/465 seasonal operation fails to deliver 108 cfs only one time just as does 465 constant, that it fails to have 150,000 acre-feet of storage 18 times in 100 years more often than does the constant pool. It fails 46, 10, and 5 times in 100 years more often than the constant pool to attain 100, 50, and 20 thousand acre-feet of storage. The same type of statements can be made for all the seasonal operating plans when compared to permanent reallocation of flood control to conservation storage.

Another fact in support of this statement lies within Figures 21 and 22. Notice how the seasonal plan and the constant reallocation plan both meet at the origin. Both have the same minimum storage value for which they fail to maintain only once. However, it quickly becomes apparent that the reallocation curve for constant operation is above the seasonal curve. This shows that for higher storage values the seasonal curve fails to attain these values more often than does the constant plan after reallocation.

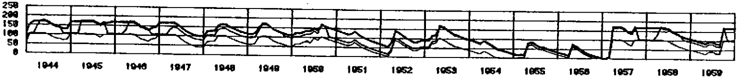
The reader may wonder why Figure 23 shows that the seasonal 455/475 pool is far less reliable at all storage values than the permanent constant reallocation plan for 475 feet when the 455/462 and 455/465 seasonal plans were only slightly less reliable at all storage values compared to their respective permanent constant reallocation plans. A reference to Figure 25 will show that the critical period for the 475 constant plan lasted about eleven years while the 455/475 had a longer critical period of about 13 years. The critical periods of the 455/462 and 455/465 seasonal plans stretched from about May of 1953 to April of 1957, nearly four years. The critical periods for these seasonal plans are much shorter than that of the 455/475 plan. There appears to be a connection between the amount of inflow and the length of the critical period for various seasonal operating plans.

It seems as though there was ample inflow during the critical periods of the 455/462 and 455/465 seasonal plans to keep their reservoir levels the same as their respective constant reallocation plans during the critical period. The inflow of April and May of 1953 was sufficient to fill the seasonal conservation pools of these two plans before the critical period began.

However, the same inflow that filled up these two seasonal pools was not enough to fill the seasonal pool of the 455/475 plan. Further reason for the lack of inflow deals with the size of the seasonal conservation pools. The differences in storage between the tops of the March and April conservation pools for the 455/462 and 455/465 seasonal plans is 43,900 and 67,900 acre-feet respectively. The difference for the 455/475 plan is 165,900 acre-feet. It is less of a mystery why the 455/475 pool did not fill up as did the other two seasonal plans when it is considered that the difference in the amount of seasonal storage to be filled for the 455/475 is 3.8 and 2.4 times that of the 455/462 and 455/465 plans, respectively. A 278 and 144 percent increase in the amount of inflow is required to fill the 455/475 seasonal pool above that required for the



Top of conservation pool elevations of 455, 455/462 seasonal rule curve, and 462 feet.



Top of conservation pool elevations of 455, 455/465 seasonal rule curve, and 465 feet.

Top of conservation pool elevation of 455 feet.

Top of conservation pool elevations of 455, 455/475 seasonal rule curve, and 475 feet.

Critical period comparisons for drafts of 102 cfs (65.9 mgd), 108 cfs (69.8 mgd), 127 cfs (82.1 mgd), and 81 cfs (52.4 mgd), from top to bottom respectively.

Figure 25.

Critical Period Comparisons

other two seasonal plans. Lack of sufficient inflow to fill the seasonal pool as the other two pools were filled accounts for the smaller reliability of the 455/475 seasonal operating plan.

Table 18 is based on the number of times the reservoir storage dropped below different values per 100 years of operation. These values are actually converted from the number of times the reservoir went below these values for a simulation based on the 75 years of recorded data. To obtain the number of times the reservoir went below a certain level in 75 years instead of 100, divide the figures of the table by 1.33. These numbers can be taken to the graphs of minimum annual storage frequency (Figures 17, 18, and 19) to see that the numbers on the graph actually correspond to the chart, and vice versa.

Flood Control

Previous sections and chapters of this thesis have outlined the general method of computing the Expected Annual Damages along a stretch of river due to flooding. The Case Study (Chapter 5) gave the specific results of the EAD computations for six different operating policies as applied to Waco Lake.

These EAD figures were based on the damages resulting from different streamflows caused by the different policies. On the average, each policy caused a different flow at the same control point for the same storm. The EAD figures were based on seven recorded storms which occured between 1957 and 1981. The only storm which caused the same flow at the same control point for every different plan was the storm of April, 1957. This storm completely filled the conservation and flood control portions of the reservoir regardless of the plan of operation.

As a reminder, the only discharges that were actually recorded were those resulting from the present-day operation (top of conservation pool constant at elevation of 455 feet) after the dam was built in 1965. Lake Whitney was already in operation at that time. The 1957 flood resulting from the top of conservation pool at 455 feet all year and the flows resulting from the other five operating policies were taken from the HEC-5 simulation of the Waco Lake and Lake Whitney system. As the reader will recall, the recorded releases from the dams were routed to the control points and subtracted from the recorded flows there to obtain the lateral inflow hydrographs. Next, the recorded inflows to the reservoirs were routed through the lakes with the new operating policies and the resulting releases routed downstream to the control points, and added to their respective lateral inflow hydrographs to obtain the total flow at that point for each different policy. Hydrographs of lateral inflow remained constant regardless of the operating policy. Lateral inflows are a direct result of rain falling on areas which contribute to flow at the control point but are downstream of the dam and are therefore unaffected by the dam or whatever reservoir policy happens to be in effect. The peak flow of each hydrograph at each point was used to determine the EAD for each plan of operation.

Up until this point, all calculations and results had followed standard procedures for computation of EAD. However, the magnitudes of the EAD for each plan did not vary as was expected. It was assumed that the damage should be lowest for the 455 constant and highest at the 470 constant; with 455/462, 455/470, and 462 constant somewhere in between the two extremes. However, the EAD came out in the following order (lowest to highest), magnitude, and change of magnitude, as shown in Table 23. The operating policies which called for a constant top of conservation pool had relationships

Table 23.

Order, Magnitude, and Change in EAD for Various Operating Policies

Order	Top of Conservation Pool Elevation November - March	Top of Conservation Pool Elevation April - October	Expected Annual Damages ($1,000)	Change in Expected Annual Damages
1	455	470	3,213	
				662
2	462	470	3,875	
				309
3	455	462	4,184	
				501
4	455	455	4,685	
				109
5	462	462	4,794	
				211
6	470	470	5,005	

with respect to each other as was expected. The 455 constant policy was expected to have the lowest of three because it should have the greatest amount of available empty flood control space to store a potentially damaging flood. (A reference to Table 24 will show less conservation storage available at 455 than 462 or 470 feet and more space available for flood control).

It was for this very reason that it was assumed that the 455 constant should have the lowest EAD of all the other plans, including the seasonal plans. The same argument was thought to apply to the seasonal plans, and it had the same conflicting results. Among the seasonal plans it was assumed that the 455/462 plan should have lowest of the seasonal plans because of the extra flood storage space that it had compared to the other plans. However, as explained below, due to imperfect streamflow forecasting, the constant operating policies resulted in releases which contribute to downstream flooding. The downstream control point is 96 to 130 hours travel time below the dam. Only 24 hours of foresight was used in the model, which is realistic.

Multiple peaked hydrographs for some of the storms at the downstream control points would recede to below nondamaging discharge levels, triggering releases from the reservoir. In the several days required for the reservoir release to reach the downstream control point, the lateral inflow hydrograph would have risen again such that the combined flows resulted in flood damages.

An incidental benefit of the seasonal rule curve operating policies was the prevention of the situation just described. Most of the historical floods used in the simulation occured during late spring and early summer, after the designated top of conservation pool was raised but before the resulting additional conservation capacity filled with water. A significant portion of the normal releases of flood water were not made after downstream discharges

Table 24.

Conservation Storage Capacities

Top of Conservation Pool Elevation (feet msl)	Conservation Storage Capacity (acre-feet)
455	104,100
462	148,000
465	172,000
475	270,000

receded below damaging levels because water was being stored in the seasonal conservation pool instead of the flood control pool. Thus, premature reservoir releases, which would contribute to flooding several days later, were prevented or reduced with a resultant decrease in expected annual damages.

This was the case for 3 of the 7 recorded storms where the associated discharges (and thus damages) at the two control points farthest from Waco Lake were not in the order of magnitude corresponding to their respective operating policies as was expected. In fact, the 455/470 discharge resulting from the 1977 flood showed no damage at one control point (Hempstead) whereas the other five policies showed damage. For these key storms, the discharges at the two farthest points were lower for all three of the seasonal plans than for the three constant policies.

Tables 25 and 26 show the flows obtained from the simulation of Waco-Whitney reservoir system at Richmond and Hempstead for these storms.

The damaging discharge is 84,000 cfs and 60,000 cfs at Hempstead and Richmond respectively. This shows that the June of 1981 storm caused no damage at Hempstead. It caused damage at Richmond for all policies except the 455/470 seasonal operating plan. To aid in the explanation, Figure 26 has been included. It shows the flows at Richmond and releases from Waco Lake for the flood of April, 1966 resulting from the 462 constant and the 462/470 seasonal operating plans.

The plotting of the flows and releases begin at time zero, the beginning of the storm. At this point the conservation pool of the 462 constant plan is about full at 148,000 acre-feet. The seasonal pool has a little more water in it, starting the storm at 150,057 acre-feet of storage in the conservation pool. The seasonal plan of operation allows the storage to begin to increase in April.

Table 25.

Flowrates at Hempstead

Date	Operating Policy 455	455/462	462	462/470	455/470	470
April 1966	95,990	89,082	96,295	86,763	86,758	98,079
April 1977	121,269	116,638	121,600	113,079	108,142	124,146
June 1981	73,647	73,513	73,526	69,343	64,710	72,077

Table 26.

Flowrates at Richmond

Date	Operating Policy 455	455/462	462	462/470	455/470	470
April 1966	102,336	96,114	102,639	92,141	92,103	104,003
April 1977	117,897	112,020	118,260	108,917	105,276	120,639
June 1981	71,515	71,425	71,436	67,454	62,327	69,992

Flows that cause the reservoir to rise above 455 are released by the constant plan, but are stored by the seasonal plan. This explains the slightly higher storage for the seasonal plan. (Remember that since the storm starts in April, the first month that the seasonal pool is allowed to rise, that the seasonal policy can have up to 218,000 acre-feet of storage, as opposed to the 148,000 for the constant plan). Sixteen hours later at point A, the 462 constant policy begins to make releases because water has since risen above the top of the constant conservation pool into the flood control pool.

However, the seasonal policy uses the same inflows to fill its seasonal pool instead of triggering releases as does the constant policy. Point A to point B in Figure 26 encompasses about 84 hours. All during this time, the constant policy is making releases to lower the reservoir level back down to the top of the conservation pool at elevation 462 feet and 148,000 acre-feet of storage. However, at point B in time is where the seasonal policy begins its first releases, over 80 hours after the constant began its initial releases.

As can be seen from the plot of the releases only, the constant plan releases water much sooner, at a higher rate, with a resulting higher volume than does the seasonal policy. At point C, both initial release periods stop. There the two policies have been able to lowered reservoir levels down to the top of their respective conservation pools. Another interesting fact to note is that the releases have ceased about 24 hours in advance of the time when the flows at Richmond began to exceed the damaging threshold of 60,000 cfs. Whether by plan or by fate, 24 hours happened to be the amount of foresight given to the computer model to forecast future flows at downstream points.

Flows at Richmond rise, crest, and begin to fall between points C and D. Point D is one time step past the point at which the storage in the reservoir

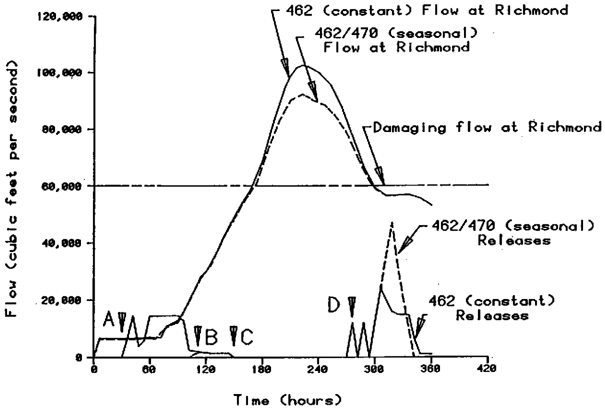


Figure 26.
Waco Lake Releases and Flows at Richmond for April 1966 Flood

peaks and begins to recede for both plans. Then the same release is made for each plan. No releases are made in the next time step because damaging flows are still in progress at Richmond. This process is repeated one more time until the point at which flows at Richmond drop below 60,000 cfs for both plans. At that point releases continue for both plans until both flood control pools are empty leaving full conservation pools.

It is evident from the graph that the constant 462 policy had a higher discharge at Richmond than did the seasonal plan. Also evident, as previously noted, was the higher volume and earlier release time of the constant level policy. The extra release volume from the constant plan contributes to the lateral inflows below Waco Lake causing damaging flows at Richmond in excess of what might have occured if the seasonal plan had been in operation.

It is also possible that the approximately 80 hours between points A and B in which the constant plan releases and the seasonal plan does not release (but instead only stores inflows) allows the lateral inflows to rise, peak, and recede before the peak of the releases from the reservoir reach the control point.

A point that is crucial to the explanation of why damage is lower for the seasonal as opposed to the constant plan is the fact that the seasonal plan makes a semi-permanent (8 months) trap of the water coming into the reservoir that causes the water level to rise above 462. It makes no releases at all until the water has risen above the 470 mark. The 70,000 acre-feet of storage between 462 and 470 is not released downstream to inadvertently add to lateral inflows already in progress and therefore does not contribute to flooding as seriously as would the constant plan. This water is stored for conservation use in the coming months. Whatever remains in the seasonal pool at the end of

October will be released to lower the reservoir back to the level of the winter pool at elevation 462).

Just as important in this explanation is the fact that the seasonal pool only releases water to get the level back down to 470, not 462 as does the plan for the constant pool. For the April of 1966 flood, this fact keeps about 60,000 acre-feet of water from being released if the seasonal plan is used instead of the constant. The seasonal pool releases about 5000 acre-feet compared to the 65,000 acre-feet released several days earlier by the constant plan.

Therefore, for the April of 1966 flood, the earlier and greater amount of water released by the constant plan in order to maintain the lower top of conservation pool level of 462 feet caused higher flows and therefore higher damages at Richmond than did the later plan. This fact of "incidental flood control" (trapping a portion of the storm inflow in the conservation pool), allowed the damage for this particular storm to be lower for the seasonal instead of the constant operating plan. The circumstances surrounding the floods of April 1977 and June 1981 behaved in much the same manner as did the flood of April 1966 which was described in detail previously. Incidental flood control took place for the seasonal policies allowing their damages to be lower than the constant plans.

However, this set of circumstances (seasonal pool almost empty ready to be filled at the time the storm started) did not occur for all the storms. For some of the storms the seasonal pool was much higher at the beginning of the storm and was not able to provide the incidental flood control like it did for the April 1966 flood. For these storms the damage was higher for the seasonal pool instead of the constant pool, as was expected at onset of this investigation.

On the other hand, since three of the seven damaging floods showed this benefit at the two farthest control points (which had the highest flows and highest damages) it was enough to lower the EAD of the seasonal plans of operation below that of the constant plans. The explanation as to the reason for lower EAD for seasonal control policies as opposed to constant policies leads to a further explanation as to why the 455/470 policy had the lowest EAD, the 462/470 was next, followed by the 455/462. (As was shown by Table 23, all of these policies were lower in EAD than the constant policies).

After understanding the reason why the 462/470 policy had a lower EAD than did the 462 constant policy, it is fairly evident why the EAD of the seasonal policies are ordered as they are. The benefits coming from incidental flood control rise in relation to the increase in storage available between the March top of conservation pool and the April seasonal top of conservation pool. Table 27 shows the EAD, storage at each level, and storage increase between the March and April allowable conservation pools.

As can be seen from the table, the EAD increases as the amount of storage between the March and April top of conservation pools decreases. This is because less volume is available to trap incoming flood flows for conservation purposes. With less volume available to trap these inflows, more water must be released to maintain the lower top of conservation pool, as was pointed out for the April, 1966 flood as pertaining to 462 constant and 462/470 seasonal plans of operation. These additional flows add to lateral inflows and contribute to higher flows and therefore higher damages at the control points. These reasons case EAD to rise as the amount of available storage for trapping flood water inflows decreases.

Table 27.

Change of Available Storage Space and Associated Expected Annual Damages

Plan	Available Storage (acre-feet) March	Available Storage (acre-feet) April	Change in Storage (acre-feet)	Expected Annual Damages ($1,000)
455/470	104,100	218,000	113,900	3,213
462/470	148,000	218,000	70,000	3,875
455/462	104,100	148,000	43,900	4,184

Having only 24 hours of foresight available to forecast flows at Richmond, coupled with the approximately 200 mile trip the released water has to flow to get there, seriously affects the ability of Waco Lake to give adequate flood control protection at Richmond, especially for the constant non-seasonal regulation plans. The Corps of Engineers (19) projects a 96 to 130 hour travel time for water released from Waco Lake to reach Richmond. Even 24 hours of foresight is stretching the present capabilities of streamflow forecasting. A large release could easily be made when no damages were occuring at Richmond and by the time they reached Richmond four days later contribute to flooding occuring four days later that forecasters had no way of predicting when release decisions were being formulated.

The added liabilities of extremely long distance and associated travel time from Waco Lake to Hempstead and Richmond, limited forecasting abilities, and lack of control (by Waco Lake) over the flow of the Brazos above its confluence with the Bosque tend to limit its effectiveness in controlling flood damages at Richmond or Hempstead (approximately 150 miles from Waco Lake).

Possible Case Study Ramifications

Yield Study

The most obvious ramification of the yield-storage portion of the case study seems to be the fact that the same amount of water can be withdrawn continuously during the critical period from a reservoir using seasonal control of the elevation delineating the top of the conservation pool as could be withdrawn from a reservoir with permanent reallocation. For example, the study showed that a seasonal 455/462 and 455/465 operating plan would have the same firm yield as would be expected if the present top of conservation pool elevation (455 feet) were changed permanently to 462 feet and 465 feet respectively.

One of the problems with this finding is that it was based on past records. It is usually a safe assumption that the characteristics of a long period of record will approximate what will happen in the future. However, should the future not happen as generally expected, problems might occur. Specifically, if a worse period of drought occurs than is on record, the critical period may change, A change in the inflows may cause the critical period to be longer for the seasonal operating plans than for the constant plans.

This was the case for the 455/475 seasonal and the 475 constant plans. Inflow of April and May of 1947 which filled up the conservation pool of the 475 constant operating plan was insufficient to fill up 455/475 conservation pool. This lead to a longer critical period for the 455/475 plan as opposed to the 475 constant. Although this occurrence is less likely for the 455/462 and the 455/465 seasonal plans because of the much smaller amount of water required to fill their seasonal conservation pools as opposed to the 455/475 seasonal pool, it is a distinct possibility under the right circumstances.

Therefore, if history adversely changes in terms of inflow to Waco Lake, then seasonal operation may not be able to release the same amount of water throughout times of critical drought conditions as would be able through use of the constant reallocation plans.

Although this has a distinct possibility of happening, the chances of this occuring decrease downward from the 455/475 seasonal operating plan to the 455/465, down to the 455/462. This is due to the lower amounts of storage required for each of the seasonal conservation pools.

Another reason for decreased chance of occurance lies with the period of record inflows to Waco Lake. The yield study benefited greatly from the long period of record of reservoir inflows that were available. 75 years of recorded data added greatly to the confidence that the author was able to express in the conclusions drawn concerning the yield study. If there were less years of record available, then confidence in the conclusions would slip and the author would have to surmise that a worse drought could occur giving a chance for seasonal operating plans to have a lower firm yield than that of the constant plans. However, it is this long 75 year period of record that implies that a critical period in excess of the one already recorded has a small chance of occuring.

If a smaller period of recorded inflows were available, synthetic streamflow generation could be the next best thing to having a long period of recorded inflows. Even the conclusions drawn from the long period of record that is available for Waco Lake could be compared against conclusions drawn from a string of synthetically generated reservoir inflows.

If the conclusions drawn from the synthetically generated flows were vastly different from those drawn the actual recorded flows, then it would a warning that further analysis might be required. However, if the conclusions based on synthetic flows agreed closely with recorded flows, then the researcher could place more confidence in the original conclusions.

Another ramification of the yield study is that much less secondary yield is available from seasonal plans as compared to a permanent reallocation plan featuring a constant top of conservation pool. However, from a water supply standpoint, this does not seem like a very important consideration. After all, there is certainly more water available from the seasonal plan than if no change at all was made in the operating policy of top of conservation pool at 455 feet elevation. Secondly, the seasonal pool usually fills or comes close to filling during most years. This is especially important because the seasonal pool usually comes close to filling when the water is needed most anyway. The majority of the secondary water available from a constant plan of permanent reallocation would be available during November through March when water use is lowest. Water demand by the City of Waco is low then, and the irrigation water that is released for downstream use is not needed during these months. Possibly this secondary water could be used as estuary flow maintenance, but that does seem to have been a consideration during the planning of Waco Lake, and should probably be ignored.

Therefore, it seems that from a water supply standpoint, that seasonal control of Waco Lake can at least do no worse than not changing operating plans at all, and seems to have the potential for providing the same firm yield as would a permanent reallocation of storage for constant top of conservation pool elevation. Up front, loss of secondary yield due to seasonal operation as opposed to constant sounds bad. However, that water does not seem to be needed very much during the time period in which it would be available. The storage that it would take up during those months could be put to better use as empty flood control storage volume under seasonal operation. This last statement will be discussed further in subsequent paragraphs.

Flood Control

The findings presented up to this point concerning the flood control portions of this discussion tend to show the benefits of using seasonal control of Waco Lake as opposed to the use of constant pool level control.

This position takes on renewed importance when the ability of seasonal operation to overcome the lack of streamflow forecasting ability is considered. As it stands, only limited foresight is available to dam operators at Waco concerning flow conditions 200 miles downstream at Richmond. As previously stated, releases could be made during low flow conditions at Richmond that actually contribute to flooding that has since begun in the four to six days it takes to reach Richmond. The advantage of seasonal operation stems from the fact that the extra volume required by the seasonal conservation pool actually becomes a form of flood control pool when it is used to trap incoming flood water for use in the conservation pool. Storing this water all summer long (April through October) in the seasonal conservation pool instead of releasing it downstream as is required by a constant plan of regulation, allows seasonal regulation to overcome the lack of forecasting ability for downstream flows. With seasonal operation, it does not matter as much that one day no damages are occuring at Richmond and three days later there is. With seasonal operation, no consideration is given to releasing water until the seasonal conservation pool has filled.

When the allowable top of conservation pool is raised in April, it takes advantage of the fact that six of the seven damaging floods recorded (after reservoirs Whitney and Waco were constructed) occured in April and May (the other occured in June). Therefore, the pool level is kept lower from November through March, and in April when the floods of record historically begin occuring, the allowable maximum top of conservation pool is raised to allow greater volume to be available to trap the incoming flows flows.

Consequently, the limitation of only 24 hours of streamflow forecasting ability for a station which requires four to six days of travel time for releases to reach, is softened considerably by use of a seasonal operating plan. Knowing streamflows four to six days in advance is not as important with seasonal operation as opposed to constant operation. Emphasis under seasonal control is being placed on filling the seasonal conservation pool with inflow that would have been released and possibly contribute to flooding if a constant level operating plan had instead been in effect. Therefore, emphasis is shifted from releasing water in the flood control pool to using that same water to fill the seasonal conservation pool.

One ramification of the above argument is that it implies other reservoirs with limitations similar to Waco Lake could possibly benefit from seasonal operation also. If other reservoirs are required to control flows at points that have a longer travel time than their flow forecasting ability time, and/or are not directly located on the main river on which their top of conservation pool elevation may be beneficial in terms of flood control.

Let it be stated that the pretense of this report is not to say that seasonal operation should work for every multipurpose reservoir. On the contrary, seasonal operation should probably be avoided for some reservoirs. If for no other reason, the cost of reallocating boat ramps, picnic areas, etc., may not be justified by the small decrease in EAD caused by a switch to seasonal operation instead of constant operation.

Seasonal operation may increase the amount of water in the conservation pool and reduce the amount of available empty storage left in the flood control pool to curtail downstream damages.to curtail downstream damages. This statement leads up to a very real concern for those individuals still considering the use of seasonal reservoir operation: What would happen if seasonal control was implemented and a flood occured unexpectedly in the offseason when the seasonal pool was full?

This is a very real possibility that must be considered. Even with Waco Lake, with seven of seven recorded floods (since dam construction) occuring in April, May, and June; there still remains the possibility of receiving a potentially damaging flood any time of the year. As shown in the chapter on seasonality factors, floods have occured in every month of the year in Texas, and at Waco Lake dam site.

If history (in the form of reservoir inflows) repeated itself, then there would be no problems with seasonal reservoir operation. However, this is not the case. The unexpected inflows should be expected and analyzed as to their effect upon the food control capabilities of the reservoir.

Studies already reported upon in this thesis should be useful in determining this effect. These studies included the determination of non-damaging frequency of a flood that would just fill up the flood control pool for different tops of conservation pools with no releases. Also included was a study to determine the peak release and water surface elevation corresponding to different combinations of return period storms and top of conservation pool levels.

These two studies showed what would happen if a flood should occur during a time when the conservation pool was above the present top of 455 feet. Table 22 shows that seasonal operation with a maximum top of conservation pool level at 462, 470, or 480 would still be able to store at least an 84, 57, or 27 year storm, respectively, without any releases at all, with the conservation pool completely full. This would mean completely filling the flood control pool. It also shows the releases required if a storm with a return period in excess of these figures occured.

Tables 21 and 22 demonstrated the integrity of the dam for all three top of conservation pools. Even the probable maximum flood does not cause the water level to rise above the maximum design water surface. Therefore, with Waco Lake there should be no fear of dam failure due to overtopping. Even the PMF does not cause the water level to exceed the maximum design water surface, even if a seasonal plan is implemented with a top of conservation pool elevation at 480 feet.

However, if a large storm occurs unexpectedly (July or August for example) and the seasonal pool is full, then the releases can be much higher than if the constant pool operation had been in effect. This coin has two sides though. If the remaining flood control space is inadequate due to hold the entire food without making releases, then the previous statement can be entirely true. The flip side of this coin is that should the flood occur when the seasonal pool is full, that there may still remain enough flood control storage to entirely contain the storm runoff without making any releases.

As was seen in the discussion concerning the April of 1966 storm and the 462 constant and 462/470 seasonal policies, it was the releases made in the name of flood control that actually contributed to flooding at the downstream control points. As was alluded to previously, if four to six days of forecasting ability had been available, these releases could have been rescheduled for later and stored in the flood control pool until such time as damaging flows had ceased.

Unfortunately, this was not the case. This shows that releasing water to empty the flood control pool so that the reservoir will be ready to catch the next big storm and thereby reduce the damage expected to be caused by that storm, the operators actually contribute to damages.

Therefore, the only way that seasonal operation of reservoirs will be detrimental as far as flood control is concerned, is when a flood arrives that causes the operator to make releases that are greater than that which would expected of a constant operating plan. The fact that the flood control pool would be in danger of being filled would necessitate releases being made. The fact that the pool would be higher for seasonal as opposed to constant operation at the storm beginning would cause higher releases if releases had to be made because the flood control pool was nearly full. As the reader will recall from the 462 constant and 462/470 discussion concerning the April 1966 storm, the seasonal operating plan can start the storm with a higher conservation pool level, but make smaller releases and cause smaller releases thereby causing less damage if the flood control pool does not come close to filling, as was the case.

The problem with the seasonal operation is therefore caused when the seasonal pool is full and a storm occurs which comes close to filling the flood control pool. That is when it will have flows and damages in excess of the constant plan. Otherwise, if the storm does not cause the flood control pool to come near to full capacity, then the seasonal operating policy actually has lower releases and lower flows than does the constant operating policy.

As mentioned earlier, three of the seven damaging floods of record since construction of the dams showed the benefit of seasonal reservoir operation over constant regulation. Three of the remaining four floods, May of 1965, June of 1966, and May of 1975 did not cause the flood control pool to come close to filling for the seasonal plan. Even the seventh flood, April of 1957, which did cause the flood control pool of all of the seasonal plans to fill, also caused the flood control pool of the constant operating plans to fill. Therefore not one of the damaging floods recorded since 1957 on the Brazos would have shown higher damages for seasonal operation as opposed to constant operation due to the filling of the flood control pool because the seasonal pool was full at the onset of the storm.

This consideration of floods occuring while the conservation pool is in the higher seasonal pool as opposed to the present lower plan could be partially analyzed through use of the reliability portion of the yield study. For Waco Lake, Table 18, Figures 17, 18, and 19 and to some extent Figures 21, 23, and 24 show the amount of time that various seasonal plans have a certain amount of storage as compared to constant plans. These figures and table were based on 75 years of inflow data and should be helpful in determining how often the reservoir is, or is not, at a certain level for different plans. Use of sound engineering judgement concerning what amount of flooding risk is acceptable will determine whether or not the risk of a large occuring after a seasonal pool fills is worth the added benefit of additional firm yield.

As far as flood control operations are concerned, seasonal reservoir operation can be beneficial for many multipurpose reservoir projects in general and Waco Lake in specific. Even if the probable maximum flood occurs the maximum design water surface will not be exceeded. Table 20 showed less than a 1 percent difference in the discharge from Waco Lake due to the occurance of the PMF between starting elevations of 455 and 470. This leads to the conclusion that if the pool is a few feet higher because of seasonal operation, then the PMF should cause no more problems than would be caused with constant regulation.

Yes, it is true that a flood could occur after the seasonal pool has filled that had sufficient volume to fill the flood control pool and cause releases in excess of that which might have been expected from a lower constant top of conservation pool elevation. This would cause a higher damage associated with seasonal operation than by the constant for that particular storm. This event would be likely to occur over the life of the project, as is observed from previous chapters where floods have been recorded in every month of the year. However, during this project life there should be enough cases of seasonal operation making a substantial reduction in flood damages (as is the case for the floods of April of 1966, April of 1977, and June of 1981) that in the end the overall EAD will have been reduced showing a net benefit directly attributable to the seasonal control of the top of the conservation pool elevation.

For the sake of objectivity, some of the shortcomings of the preceding analysis should be brought forth and explained, with steps laid out for the necessary corrections. The first deficiency that comes to mind is the EAD computations. It is regretful that only the flood flows into the reservoirs from 1957 until 1982 were available upon which to base the EAD figures. Although there were seven floods in the record which caused damages during that time, a longer record with a greater range of flow values would have added a greater degree of confidence to the EAD computations. A further liability that limits the confidence that can be placed in the EAD figures was the period of record of flow data at the control points downstream of Whitney and Waco Reservoirs. Although there was 35 years of flood flow data for storm runoff entering the reservoirs, there was only 18 years of flood flow data for the control points. Although this was the best data available at the time of the study, the actual figures for EAD should be viewed through a cautious eye.

As time goes by there will be an increase in the length of the flood flow record which will be very useful in comparing future EAD computations with the ones set forth in this paper. Any computation which attempts to predict future events based on past histories of similar events will increase in credibility as the length of those past histories increase. And so it is with EAD computations.

General

This portion of the discussion deals with topics which affect both the yield study and flood control aspects of the case study as they pertain to the feasibility study of seasonal multipurpose reservoir operation. The ramifications of using seasonal control to achieve both increased water supply and flood control protection are discussed in this portion also.

Perhaps the biggest or most obvious statement that could be obtained from the previous presentation of results, findings, discussions, etc., would be that seasonal operation at Waco Lake can simultaneously lower EAD below present levels expected from constant operation while increasing firm yield to that of a yearly constant permanent reallocation of flood control space to conservation storage.

As the author has consistently tried to point out it the discussion of the case study results, there are several shortcomings that tend to decrease the confidence one could place in certain aspects of the previous statement. Since most of the case study was based on simulation of the reservoir for various control policies due to recorded inflows, the length of the record of inflows becomes very important. The record used for the yield study was fairly long-which is good. However, the longer the record the more confidence that can be based on the results based on that record. If future inflows are not consistent with those recorded in the past, the yield study may not be as meaningful as it now is.

On the other hand, the results contained in this thesis are based on the best records possible. Even based on these somewhat less-than-desirable flow record lengths, it appears that a cautious application of seasonal operation to Waco Lake could prove to be very beneficial, both in terms of firm yield and flood control. An increase in the elevation delineating the top of the top conservation pool of only seven feet seasonally in April would raise it to the level (462 feet) recommended by the Corps of Engineers in their reallocation study of Waco Lake (21). However, with seasonal operation the firm yield would remain the same as would be expected from the constant reallocation plan, and would show lower EAD figures from flooding. Since the EAD figures are somewhat suspect due to a small period of record (less than 40 years), seasonal operating plan would would at least be no worse than the constant reallocation plan. The main attraction of the seasonal plan over the constant is the fact that the seasonal pool would be lower in November through March in the event a flood occurs during those months.

A secondary benefit arising from seasonal operation (or permanent constant reallocation of flood control storage to conservation) will be a larger amount of water in the reservoir. Although one of the two major thrusts of this paper has been water quantity instead of water quality, larger amounts of water tend to dilute pollutants instead of allowing them to concentrate, as might be the case if nothing was done as far as storage reallocation is considered. This might be of interest to those whose water supply is a reservoir. With conservation use in mind only, more water at higher quality could be considered as nothing less than a benefit.

CHAPTER VII

SUMMARY AND CONCLUSIONS

In Texas, as elsewhere, population and economic growth and depleting groundwater reserves are resulting in intensified demands on surface water resources. Management strategies for optimizing the beneficial use of limited reservoir storage capacity are becoming increasingly more important. The study reported here investigated the feasibility of adopting rule curve operations, which reallocate storage capacity between flood control and conservation uses as a function of the time of the year. Seasonal rule curve operation is concluded to offer significant potential for improving reservoir operations in the state in those situations in which needs for flood control and conservation purposes are severely taxing the available storage capacity.

Many factors affecting reservoir operation are seasonal in nature and should be considered in establishing a seasonal rule curve. Risk of flooding, flood damage susceptibility, water supply demands, and streamflow availability in Texas vary greatly during the year. However, the time periods when flood control and conservation storage capacity are most needed significantly overlap. Consequently, seasonal rule curve operation requires tradeoffs between project purposes.

In the case study, seasonal rule curve operation was found to be very effective compared to a permanent reallocation of storage capacity. Firm yield could be increased as much by raising the top of conservation pool during a portion of the year as by raising it to the same level permanently. Firm yield could be significantly increased with minimal loss of flood protection. A seasonal encroachment into the flood control pool for water supply purposes can actually result in incidental flood control benefits in situations in which imperfect streamflow forecasting can result in releases which contribute to flooding.

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