River Basin Planning---
A Simulation Approach

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RIVER BASIN PLANNING: A SIMULATION APPROACH

by

A. N. Halter and S. F. Miller

Part I

Introduction

During recent years a great deal of emphasis has been placed on comprehensive river basin planning and development. Federal agencies have been charged to consider:

"(1) The needs and possibilities for all significant resource uses and purposes of development, including, but not limited to domestic, municipal, agricultural, and industrial uses of water; water quality control; navigation in relation to the nation's transportation system; hydroelectric power; flood protection control or prevention; land and beach stabilization; drainage, including salinity control; watershed protection and management; forest and mineral production; grazing and cropland improvement; outdoor recreation, as well as sport and commercial fish and wildlife protection and enhancement; preservation of unique areas of natural beauty, historical and scientific interest; and (2) all relevant means (including nonstructural as well as structural measures) singly, in combination, or in alternative combinations reflecting different choice patterns for providing such uses and purposes."[1]

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[1]United States Congress, Senate, Policies, Standards, and Procedures in the Formulation, Evaluation, and Review of Plans for Use and Development of Water and Related Land Resources, Eighty-seventh Congress, Second Session, 1962, Document No. 97, p. 3.

In preparing such a plan all viewpoints--national, state, and local--are to be fully considered. The unit of planning shall be a river basin: "Planning use of water and related land resources, therefore, shall be undertaken by river basins, groups of closely related river basins, or other regions, and shall take full cognizance of the relationships of all resources, including the interrelationships between surface and ground water resources."[2]

This requirement places renewed emphasis on the concept of systems analysis. In the past lack of funds, adequate tools, and computational problems have restricted federal agencies from examining over three or four different designs. Of the different designs considered, generally one plan is recommended for authorization.

In order to undertake truly comprehensive planning, not only tremendous amounts of technical information must be available but also the planner or decision maker needs to have the knowledge and techniques to make efficient use of the information once it is available. It is with this last problem, the availability of techniques capable of evaluating alternatives, that this study deals.

Objectives of the Study

The general objective of this study is to test the applicability of simulation in evaluating water resource development projects and to test alternative resource management policies for an actual river basin system. A secondary objective is to test the applicability of

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[2] _Ibid._, p. 3.

DYNAMO simulation language for river basin modeling.[3]

The authors will demonstrate that simulation can be a useful tool to decision makers in tracing the consequence of management decisions before their implementation. Thus, part of the uncertainty due to finite comprehension of the operation of an entire system can be removed. The authors will also show that DYNAMO is a well suited simulation language for river basin problems.

The remainder of this report is divided into three sections. Part II reviews existing techniques in river basin planning and introduces the simulation approach used in this study. Part III describes the Calapooia River basin system and the project design problem. Part IV summarizes the results obtained from the computer runs of the model.

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[3]Alexander L. Pugh, III, DYNAMO User's Manual. Cambridge: The M.I.T. Press, 1963.

Part II

River Basin Planning Techniques

The literature abounds with many articles, books, journals, etc., directed to the subject of benefit-cost analysis. This has been and now is the basic tool for river basin planning and development. Theoretically, the benefit-cost ratios and net benefits are computed for several alternative size projects for an intended site. The project with the highest net benefits is submitted to Congress for authorization. However, time and funds limit this approach to only a few (very few) alternative sizes. In the case of multiple projects to be located within one river basin, the interrelations and feedbacks between alternative structures and operating procedures become tremendously complex. Without an appropriate technique to analyze these relations truly realistic benefit-cost ratios and net benefits are impossible to achieve.

In recent years two approaches or techniques have been developed to cope with design problems. The first, we will call analytic models and the second simulation models. Both rely heavily on the development of high speed, large capacity, modern computers. Both are representations of reality, consisting of quantitative inputs and outputs connected by arithmetic relationships.

Analytic Models

An analytic model is a set of equations intended to be solved to obtain an optimal value for the design variables (capacity of reservoir, capacity of channel, capacity of hydroelectric plant, etc.) using standard methods of calculus or some phase of activity analysis.

Generally, in order to solve the system of equations the model must be a drastic simplication of the real situation being described. Because of this limitation, analytic models for the most part have been concerned with only a part of a river basin system, or with one or more of a river's important features. Recent studies by Thomann, et. al.,[4] Burt,[5] and Bather[6] are examples of this partial analysis. Thomann concerns himself with water quality management. He uses mathematical programming to obtain static water quality improvements at minimum cost. Burt uses dynamic programming to derive approximate decision rules for the optimal allocation over time of a single fixed or only partially renewable resource. His specific application is with ground water. Bather attempts to solve a set of differential equations to find an optimal release rate for a finite dam. The release rate is selected to maximize a quadratic utility function.

Castle[7] used linear programming to evaluate changes in irrigation structures, cropping patterns, and intensities of water application for an assumed irrigation area typical of many of the intermountain

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[4] R. V. Thomann and Matthew J. Sobel, "Estuarine Water Quality Management and Forecasting." Journal of Sanitary Engineering Division, Proceedings of the American Society of Civil Engineers. Vol. 90 (October, 1964), pp. 9-36.

[5] Oscar R. Burt, "Optimal Resource Use Overtime with an Application to Ground Water," Management Science, Vol. 11 (September 1964), pp. 80-93.

[6] J. A. Bather, "The Optimal Regulation of Dams in Continuous Time," The SIAM Journal, Vol. 11 (March, 1963), pp. 33-63.

[7] Emery N. Castle, "Activity Analysis in Water Planning," Economics and Public Policy in Water Resource Development, ed. Stephen C. Smith and Emery N. Castle (Ames: Iowa State University Press, 1964), pp. 171-185.

areas of the West. His orientation is to the efficiency criterion of water use and investment. In another article, Castle[8] explains the use of linear programming in three simple resource situations. The first deals with structure capacity and water usage, the second with interdependent structures and the third with alternative use relationships-that is, whether a transformation function between uses showing varying degrees of competition, is supplementary or complementary among uses.

Pavelis[9] using conventional benefit-cost analysis and linear programming attacked the problem of small watershed planning. His results showed what maximum net benefits were for a specific watershed conditional upon the following constraints: limited land areas, structure size, capitalized expenditures, and that conditional damages could not go uncompensated.

Analytical models concerned with an entire river basin have been discussed by Heady and Dorfman. Heady[10], after developing a complete mathematical formulation of a welfare model outlines a simplified programming version for river basin planning and development. However, no empirical application is provided.

Dorfman, in two works, developed empirical models using the programming framework. The first is a very simple model of a valley project.[11] The

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[8] Emery N. Castle, "Programming Structures in Watershed Development," Economics of Watershed Planning, ed. G. S. Tolley and F. E. Riggs (Ames: Iowa State University Press, 1961), pp. 167-178.

[9] George A. Pavelis, "Applying Economic Principles in Watershed Planning," Economics of Watershed Planning, ed. G. S. Tolley and F. E. Riggs (Ames: Iowa State University Press, 1961), pp. 151-164.

[10] Earl O. Heady, "Mathematical Analysis: Models for Quantitative Application in Watershed Planning," Economics of Watershed Planning, ed. G. S. Tolley and F. E. Riggs (Ames: Iowa State University Press, 1961), pp. 197-216.

[11] Robert Dorfman, "Mathematical Analysis: Design of the Simple Valley Project," Economics of Watershed Planning, ed. G. S. Tolley and F. E. Riggs (Ames: Iowa State University Press, 1961), pp. 217-229.

second, however, is a very sophisticated attempt to maximize net benefits from a hypothetical river basin.[12] However, to solve the problem, the complexities of the system had to be scaled down considerably. Therefore, the resultant optimal design is only optimal for the vastly simplified problem which was substituted for the first in the interest of solvability.

Simulation Models

The second recent innovation in analyzing design problems is simulation. Simulation is a method of modeling reality and designing systems. It involves the conceptualizing, building, and operating of a model designed to represent the complex and dynamic environment of the real life situation under consideration. Simulation is not a new scientific tool. It has been used for some time by engineers and others to design electrical circuits, military strategies, and guided missiles. Military strategists during the time of Caesar employed model battlefields to develop plans of attack. Space investigations rely heavily upon simulation models in an attempt to anticipate actual flight conditions and problems. More recently, with the development of modern computers, simulation of complex and dynamic social and economic systems has been initiated. A notable piece of work is by Holland and Gillespie.[13] They attempted to simulate the recent history of the Indian economy and to use the model for testing various development schemes. However, the simulation of the Indian economy proved to be beyond their budget and manpower constraints. Thus, only a hypothetical

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[12] Arthur Maas, et.al., Design of Water-Resource Systems, (Cambridge: Harvard University Press, 1962), pp. 494-539.

[13] E. P. Holland and Robert W. Gillespie, Experiments on a Simulated Underdeveloped Economy: Development Plans and Balance-of-Payments Policies, (Cambridge, Mass.: The M.I.T. Press, 1963).

economy was simulated and hypothetical development plans tested.

An interesting micro-economic problem was studied by Glickstein, Babb and French.[14] They attempted to illustrate how Monte Carlo methods could be used to simulate milk receipts both seasonally and daily. Various procurement policies were specified and the simulated sequence of receipts programmed under each policy.

Simulation of Water Resources Problems

Hydrologists have been developing physical models and representations of hydrology for approximately 50 years. According to Dawdy and O'Donnell[15] hydrologic models of catchment behavior can be divided into two groups:

"1. Comprehensive simulation of catchment behavior, i.e., over-all catchment models; and

"2. Complete specification of each of the elements of catchment behavior, i.e., component models."[16]

The first, over-all catchment models, treat catchment components in lumped form. Generally, "the construction of the components and the parameter of the relations are adjusted until known responses, within an acceptable tolerance, are achieved from known inputs." The component models demand a more objective description of the physical relation of the many components of catchment behavior. Hydrologists have tended to lean heavily on analog computers in the development and solution of the models.

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[14] Aaron Glickstein, E. M. Babb, C. E. French and J. H. Greene, Simulation Procedures for Production Control in an Indiana Cheese Plant, Res. Bul. 757, Purdue Agr. Exp. Station, December 1962.

[15] David R. Dawdy and Terrence O'Donnell, "Mathematical Models of Catchment Behavior," Journal of the Hydraulics Division, Proceedings of the American Society of Civil Engineers, Vol. 91 (July 1965) pp. 123-127.

[16] Ibid., pp. 123-124.

Comprehensive modeling of not only the physical environment but also the economic environment of an entire river basin started approximately 10 years ago with the Harvard Water Program. The Harvard Water Resources Program initiated in 1956 attempts to bring together the talents and expertise of the engineer, economist, and political scientist to improve the methodology of systems design. The principal output of this body to date has been the publication of Design of Water Resource Systems.[17] Conventional as well as modern sophisticated techniques of analysis for systems design are explored in this book. In addition to the programming of Dorfman, previously mentioned, simulation is tested and used.

The Harvard study simulates a simplified river basin system on an IBM 704 computer. The hypothetical river basin system involves 12 design variables consisting of reservoirs, power plants, irrigation works, target output for irrigation and energy, and specified allocations of reservoir capacity for special purposes, i.e., flood control. The hydrology of the system is based on the Clearwater River of northern Idaho. The basic time interval used for computation is one month; however, during floods it changes to six hours. The flows are routed through the reservoirs, power plants, and irrigation structures for a 32-year period according to a fixed operating procedure. It readily became apparent to the group that the 32-year record of historical stream flows (Clearwater River) was not adequate. Therefore, a technique was developed to synthesize longer stream flow periods. Later an attempt was also made to relax the inflexible operating procedure. The economic benefits of the system

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[17] Maass, et. al., op. cit.

are determined from the beneficial use and control of the water moving through the system. The model was programmed in Fortran II language.

The overriding objective of the study was to improve methodology of systems design "so as to identify the optimal design, or, if this is not practical, to evaluate readily a sufficiently large number of possible designs to justify assurance that the best of these approximates the theoretical optimum."[18] Two approaches as previously mentioned were used: the analytic approach of Dorfman with its built-in maximization routine and the simulation approach. Because there is no internal algorithm for maximization in the simulation approach, an attempt was made to sample from the many combinations of design variables in such a way as to facilitate the search for maximized net benefits. In addition, steepest ascent procedures were attempted to achieve this objective.

To develop an adequate design, Dorfman[19] suggests the employment of "analytic models and the simulation approach in tandem." First, the problem can be broken into a set of manageable mathematical relationships that can be solved for an approximation to a optimal design. Then a range of plausible variation around the tentative solution can be explored by a sequence of simulations. While interesting, one wonders if the advantages gained from the development of a simplified analytic model might be lost when the interrelationships between the simplified models are considered in an environment more closely resembling reality.

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[18] Maass, <u>et. al</u>., <u>op. cit</u>., p. 6.

[19] Robert Dorfman, "Formal Models in the Design of Water Resource Systems," Water Resources Research, Vol. I, No. 3, 1965.

In a follow-up to the original simulation, the Harvard Water Resources Group undertook the development of a computer model of the Delaware River basin.[20] This program includes a sub-program for simulation of the Lehigh River basin. The multi-purpose system involves water supply, dilution-flow, recreation, flood control, and water power. The model uses many of the concepts and approaches developed in the original simulation. The program is written in Fortran language for the IBM 7090 computer.

A comprehensive river basin simulation of the Susquehanna River Basin was developed by Battelle Memorial Institute.[21] The approach used by Battelle is somewhat different from that of the Harvard group. Battelle chose to study the economic interrelations existing in the river basin in an attempt to ascertain what influences economic growth of an area. To do this the entire basin was broken down into subregions. Each of the subregions is described by a series of equations relating the interrelations and feedbacks of three major factors: (1) demographic, (2) employment, and (3) water. The major water uses considered in the model are: (1) water quality, (2) water supply (agricultural, urban, and industry), (3) recreation, (4) flood control, and (5) electric power. The model was run over a 50-year period (1960-2010) on the IBM 7090 computer. It is written in the DYNAMO language.

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[20] Harold A. Thomas, Jr., and Robert P. Burden, Operation Research in Water Quality Management, Division of Engineering and Applied Physics, Harvard University (Cambridge: by the Division of Engineering and Applied Physics), pp. 201 59 2-39.

[21] H. R. Hamilton, et.al., A Dynamic Model of the Economy of the Susquehanna River Basin, A Progress Report prepared by Battelle Memorial Institute to Susquehanna River Basin Utility Group (Columbus: Battelle Memorial Institute, 1964).

Philosophy of Simulation Approach

J. W. Forrester in his book Industrial Dynamics[22] studied industrial organizations from a systems engineer's point of view, using the simulation approach. This approach in a river basin context is to (1) formulate a model representing the pre-project condition of the system, and (2) design and redesign the model to find the appropriate system modification--structural and nonstructural-which will lead to improved performance. To accomplish these objectives, the simulation approach progresses through several steps. The first step, formulating the pre-project model, traces the cause and effect feedback loops that link the system together. For example, the upstream flows appropriately lagged, influence the downstream channel level. Also, a serial correlation exists between mean-daily hydrologic flows which must be understood and specified. After the model is designed, a mathematical model of the conceptualized model is formulated and the model's behavior generated through time on a digital computer. The validity of the model is tested by comparing computer results with all pertinent available knowledge about the actual system. Generally, the model is revised by increments until it is an acceptable representation of the real system.

The degree to which the model corresponds with reality must be specified by the model builder or user. In the final analysis, the user must decide whether the model corresponds to reality to a sufficient degree for decision making.

When such an acceptable representation of the pre-project system

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[22] J. W. Forrester, Industrial Dynamics, New York: The M.I.T. Press and John Wiley and Sons, 1961.

is achieved, the model is expanded to include the proposed project. The behavior of the new project system is then generated on the digital computer. Comparison of the new results with the former provides a test of the proposed change. The pertinent design variables and decision rules are again changed, the model re-run, and the results compared. In this way improvement is made in increments, pyramiding from the previous knowledge gained.

There is no optimizing procedure built into the simulation approach, the philosophy being that knowledge is gained by repeated trials. A hypothesis is developed and the consequences obtained, then with this new knowledge a second hypothesis is developed and the consequences generated. In this way, knowledge grows and develops and better and more near optimal decisions can be made. For most complex social and economic systems, mathematical or optimizing methods fall far short of finding the best solution. The misleading objective of trying to find an optimum solution often results in simplifying the problem until it is devoid of practical interest. In the simulation approach the objective is to make improvements in the system; by necessity these are made in small increments.

As a means to apply the philosophy of the simulation approach as espoused by Forrester, computer specialists at the Massachusetts Institute of Technology developed a simulation language and computer program called DYNAMO.[23] DYNAMO which runs on IBM 7090 series machines, is a comparatively easy language to learn and understand. While DYNAMO was developed

[23]Halter and Dean give a comprehensive discussion of the special features of DYNAMO in: A. N. Halter and G. W. Dean, Simulation of a California Range Feedlot Operation, California Agricultural Experiment Station, Giannini Research Report No. 282, May 1965.

specifically for industrial systems, the structure is sufficiently flexible to fit many other forms and types of economic systems and problems.

Structure of a DYNAMO Model

The structure of a DYNAMO model is basically quite simple in that it consists of three interconnected components: levels, rates, and auxiliaries. Levels are accumulations of rates; decisions and exogenous factors control the rates of flow between levels; auxiliaries are intervening variables used for writing the rate equations. In a model one or more equations are written to represent each component. The interconnections between rates and levels are shown diagrammatically in Figure 1. The solid lines represent flows of materials, goods, inputs or outputs, etc., while the dotted lines represent flows of information. The valve symbol represents points of decision or exogenous influences which regulate the rates of flow between the levels. Information concerning the levels is used to make the decisions which regulate the flows. Other information concerning exogenous factors may influence the decisions and hence the rates of flow, or the factors may influence the rates of flow directly without affecting a decision. Examples of levels in a river basin problem are the water level in the reservoir and the flow level in the channel. The irrigation releases as controlled by the level in the reservoir and the irrigation need is an example of a rate in a river basin model. Auxiliaries are variables such as the percentage of irrigation return flow which influences the rate of flow between the reservoir and channel level.

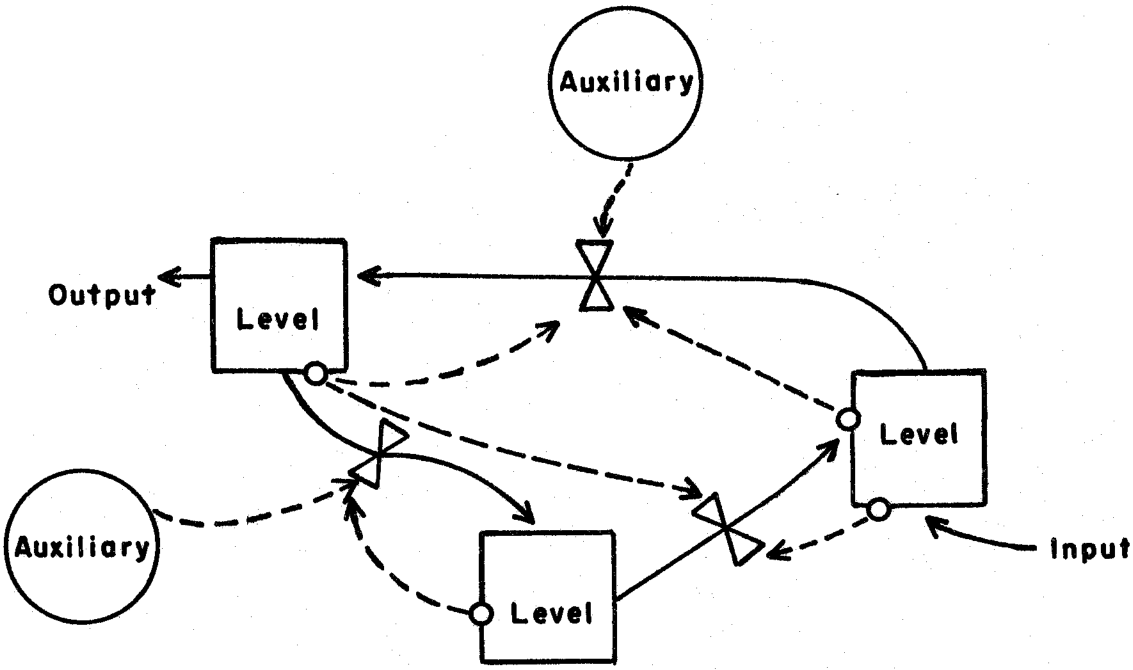


Figure 1. Interconnections between levels and rates of a DYNAMO model.

One very important aspect of the problem which is poorly shown by the diagram is the time-dependent nature of the decision variables (rates of flow per unit of time). This dependence can be illustrated by describing the time sequence of computation in terms of levels, auxiliaries, and rates. The procedure by which the computer calculates these variables is to move through time in discrete steps and calculate all the variables at each step. The procedure is graphically shown in Figure 2.

There are three time periods of importance: the present, K; the past, J; and the future, L. The length of time between calculations is denoted by DT. In other words, the past, J, is the immediately past DT from the present, K. The future, L, is the next DT or time interval from K. The level equations are first calculated from information about levels at time J and rates over the interval JK. Next auxiliaries are calculated from information about levels and other auxiliaries at time K and rates over the interval JK. Finally, rates for the forthcoming interval KL are calculated from levels and auxiliaries at time K and rates over the interval JK. After evaluation at time K of levels, rates and auxiliaries, time is indexed forward; i.e., J, K, L positions in Figure 2 move one time interval to the right. The K position is now J, L is K, and a new L is indexed. The sequence of calculations can then be repeated to obtain new values of the variables from information about the old values. The computer in this way traces the course of the model through time as the levels lead to decisions and actions that in turn affect the levels. Thus, the interaction of the variables and of their time dependency is affected.

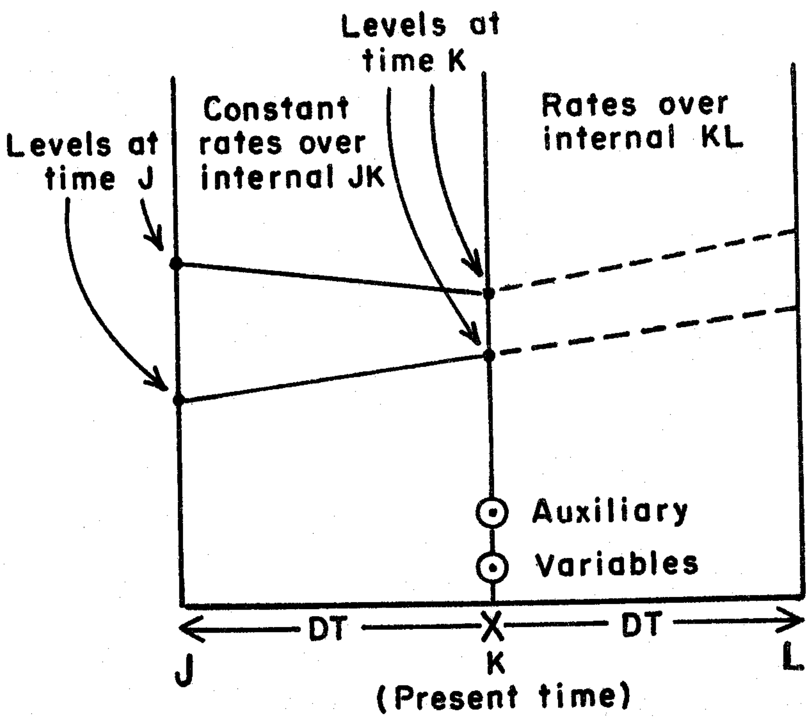


Figure 2. Calculation of levels, auxilaries, and rates through time.

A flow diagram of the Calapooia River basin model as outlined for this study is shown in Figure 3. Schematic representations of main levels, rates and auxiliaries are necessary as an aid in understanding the problem. Often the most important part of simulation modeling is obtaining a comprehensive understanding of what is actually going on in the problem under study. The flow diagram is composed of two basic segments: water movements and benefit flows. Solid lines represent movements of water; dotted lines show the dependence of decisions upon information concerning levels and auxiliaries; solid lines interspersed with dollar signs represent movement of flows of benefits.

An example will give a better understanding of the diagram. Irrigation releases indicated by a valve underneath the reservoir level are influenced by the reservoir level and the irrigation need. The irrigation use level when multiplied by the percentage return flow constitutes the return flow into the channel. And the two, irrigation releases and the return flow, certainly influence the channel level. On the other side of the diagram, channel drainage net benefits are influenced by the capital and operation, maintenance, and repair costs of channel improvement; by the channel capacity; and by the channel level during the drainage period. The channel drainage net benefits then make up part of the total net benefits for the project.

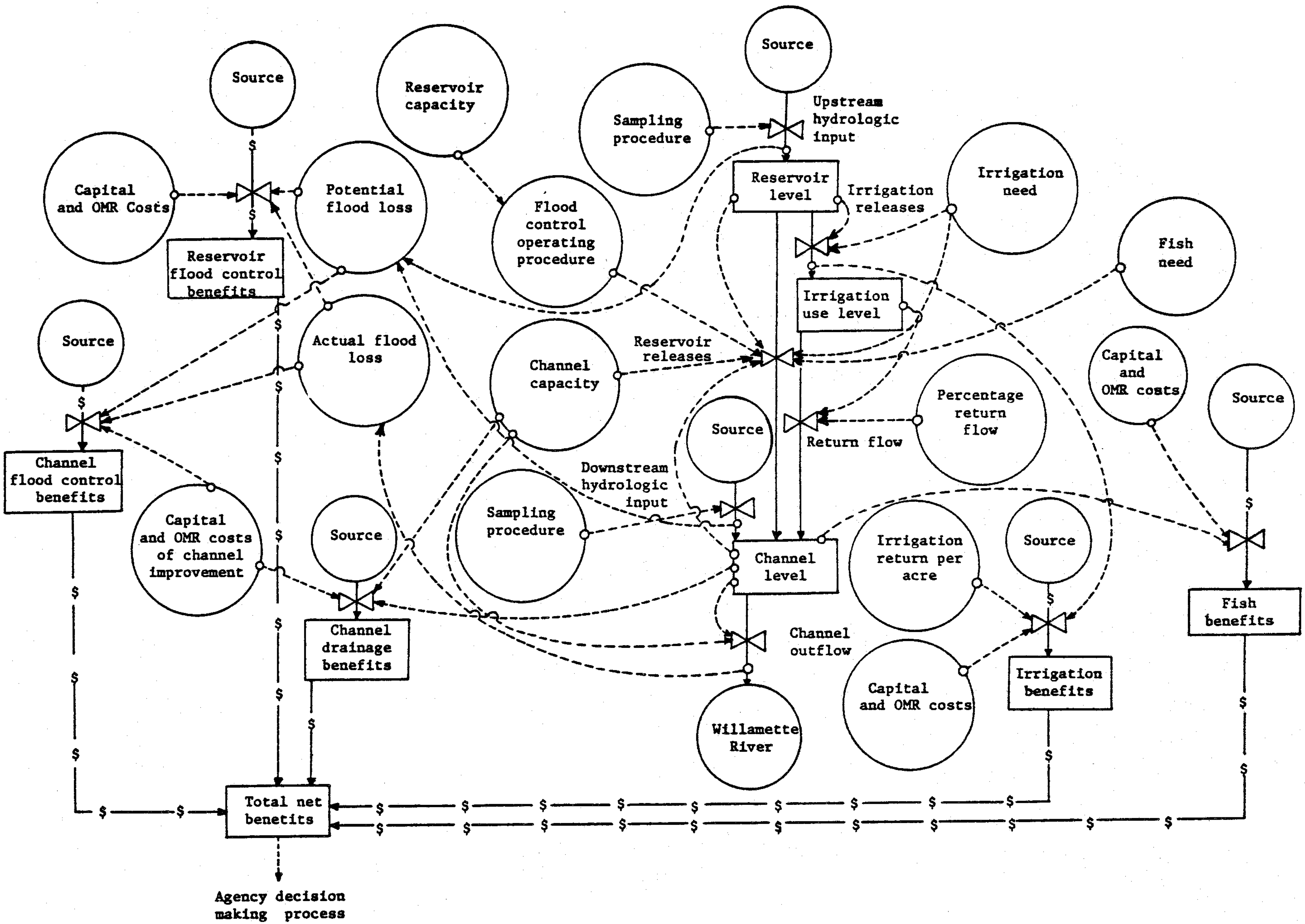


Figure 3 . Flow Diagram of Calapooia River Model

Part III

Calapooia River Basin Simulation

This section of the report provides a general description of the river basin and the beneficial water uses included in the model. First, a description of the basin is presented. Second, the hydrologic characteristics to be modeled and the results of their simulation are discussed. Finally, the beneficial water uses are discussed, together with the assumptions necessary to the basin modeling.

Selection of Study Area

The selection of the river to be studied was done in cooperation with the Corps of Engineers, U. S. Department of the Army. It was immediately apparent to the authors that considerable data concerning the physical relationships as well as the economic situation of the selected river basin would be needed. The Corps kindly offered to help in the selection of the river and to make available the data that they had. The river selected was the Calapooia River of western Oregon. The Corps had conducted a benefit-cost analysis of a proposed project on the river in 1948. Congress authorized the project but because of the lack of local cooperation in a necessary, but separate channel improvement project, the project was never funded.

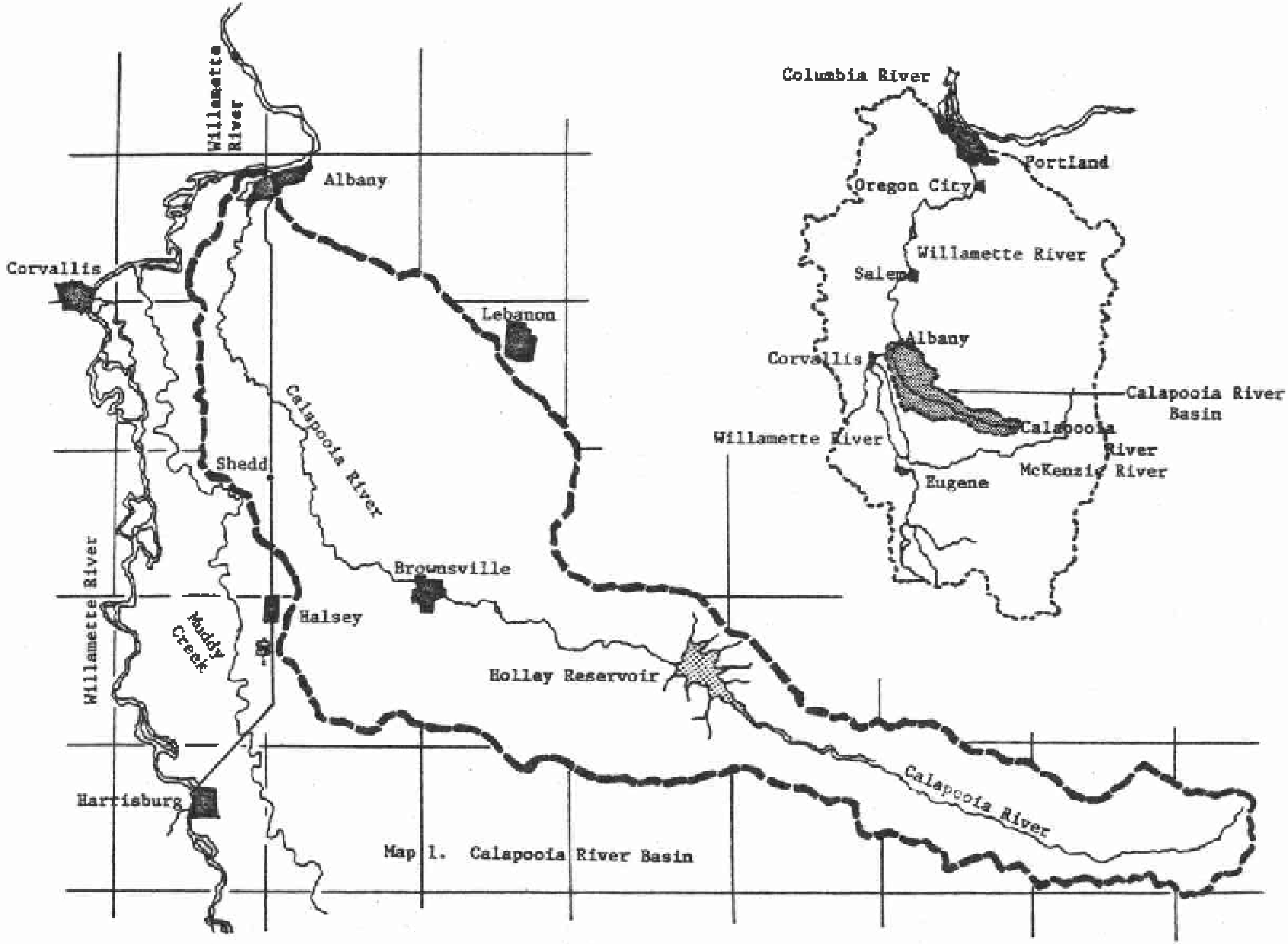
On the basis of a specific Congressional authorization, and consistent with policy outlined in Senate Document 97, the Corps of Engineers initiated a re-study of the Calapooia basin. However, at the initiation of this simulation study the Corps had not had sufficient time to complete the

the project re-study in terms of the new authorization. Thus, many of the costs and benefits used in this report are approximate, or "best guess" estimates. Care should be taken not to use this study as final in regard to the benefits and costs from the several alternative projects that will be discussed.

It was decided that all of the possible beneficial uses of water would not be evaluated. Instead, the study has been confined to benefits arising from flood control, irrigation, drainage, and fish life enhancement. It was thought that these uses would be adequate for the purpose of testing and developing simulation methodology of a river basin system.

Description of the Calapooia River Basin

The Calapooia River is located in the middle Willamette River basin of western Oregon (See Map 1). It starts in the Cascade Range on the east side of the Willamette Valley and enters the Willamette River at Albany, Oregon, approximately 76 miles from its source. It drains 371 square miles. The average gradient of the river drops from 390 feet per mile in the upper reaches to an average of five feet per mile on the valley floor. Average annual precipitation ranges from 100 inches in the high Cascades to a low of 30 inches on the valley floor. Average monthly precipitation at Albany is shown in Table 1. A cursory examination shows that the seasonal distribution is very erratic. Approximately 44 percent of the year's precipitation falls during the three month period--December, January, and February. In contrast, only two percent falls during the two summer months of July and August. Often periods of 60 to 90 days



Map 1. Calapooia River Basin

Table 1. Normal precipitation, Albany, Oregon

Month	Precipitation in inches[a]
January	6.01
February	4.94
March	4.34
April	2.31
May	1.98
June	1.48
July	.42
August	.51
September	1.47
October	4.14
November	5.98
December	7.08
TOTAL	40.66

[a]Average precipitation from 1931-1960.

pass in the summer months without measurable precipitation. Most of the precipitation comes in the form of rain. Snow rarely lasts longer than a few days on the valley floor and along the foothills of the Cascades.

The Calapooia River basin drainage area is located entirely in Linn County. It comprises approximately 16 percent of the total county area. Because much of the economic data cited are only available on a county basis, many of the references will be to county figures. The implied assumption is that the Calapooia River basin is similar to the county as a whole.

The total population of Linn County was 58,867 in 1960, or more than three percent of the total population of the state. The population density is just over 29 persons per square mile, compared to less than 20 for the state. There are presently only three urban areas (population of over 2,500) in the county. They are Albany-12,926, Lebanon-5,858, and Sweet Home-3,353. The county population has grown 8.4 percent since 1950, primarily due to the growth of the urban areas.

The economy of the county is built around the primary industries of timber and agriculture. In recent years recreation and tourism have developed into an important third industry. For example, recreational use of national forests has risen from 125,000 visits in 1956 to 480,000 in 1960. The value of all farm products sold rose 19 percent during the five-year period 1954-1959. The amount of irrigated land increased from 19,099 acres in 1954 to 23,478 acres in 1960, or 23 percent. Timber production has reached a plateau and is expected to remain fairly constant over the next 20 years. It has been estimated that by 1985 the Calapooia basin will have a demand for domestic, municipal, industrial, and irrigation water supplies of over 413,000 acre-feet; with an average annual yield of 863,000 acre-feet and proper storage for periods of low flow, this need can readily be met.[24]

The mean annual run-off of the Calapooia River at Holley Dam site is estimated to be 281,000 acre-feet. Extremes are estimated to be 156 and 61 percent of the mean. The largest flood on record at Holley occurred in December 1945 when the Calapooia had a maximum discharge of 12,400 second-feet and 14,500 second-feet at Albany. The largest flood is believed to have occurred in 1861 when the maximum discharge is estimated to have reached 15,000 second-feet at Holley and 48,000 second-feet at Albany. This is referred to as the design flood or the one-in-100-years flood. A resume of several floods on the Calapooia is given in Table 2.

Table 2. Natural maximum discharges of floods of record and historical floods, Calapooia River

Station	Discharges for floods-cubic feet per second				
	1956	1945	1927	1890[a]	1861[a]
Holly Dam site..........	10,700	12,400	9,600	13,800	15,000
Shedd..................	b	13,300	13,800	24,300	28,000
Albany.................	32,700	14,500	b	b	48,000

[a]Computed from meager records of rainfall and river stages.
[b]Not available.

[24]State Water Resources Board, Middle Willamette River Basin. Salem, June 1963. p. 87.

Simulation of Hydrologic Data

The generation of hydrologic input is the crucial phase of the river basin simulation. The time shape of the hydrologic flows, including the magnitude and duration of flood flows, determine the benefits obtainable from the intended development. The researcher in conjunction with the decision maker must decide on the extent of detail that is to be included in the model. If sufficient detail is not included there exists the danger of excluding the possibility of evaluating short-run benefits and losses from daily operating procedures. Whereas if every detail is included, for example, down to hourly stream flows, they run the risk of making the program so cumbersome and inefficient that the computer expense of running the model becomes excessive. The problem itself often dictates to some extent the compromise which must be made between the two extremes.

Frequency Functions

The general frequency function of the monthly mean-daily flows on the Calapooia is skewed. The median flow is skewed to the left of the mean (Tables 3 and 4). Thus, in any month most of the flows are less than the average for the month. Also the frequency of flows starts at a moderate level, immediately swells to the median and then gradually tails off to the right. A deliberate attempt was made to preserve the appearance of the historical frequency in the simulated hydrology. The historical and the simulated frequencey of hydrologic flows for six different months are shown in Appendix C. Figures 1-6. Similar figures are available upon request from the authors for the months of March, July, September, and November.

Table 3. Mean-daily stream flows, Calapooia River[a], Upstream hydrologic data[b]

Month	Mean	St. Dev.	5% Interval about mean	1% Interval about mean	Median	Square root of second moment about median
Jan	851	886	790-914	770-934	570	930
Feb	896	730	842-950	825-967	700	756
Mar	740	509	704-776	693-787	585	532
Apr	592	395	571-613	564-620	500	406
May	368	264	349-386	344-392	300	272
Jun	203	168	191-215	188-219	165	172
Jul	77	40	14-80	73-81	69	41
Aug	41	14	40-42	40-42	38	15
Sep	43	38	41-46	40-47	34	39
Oct	174	375	148-201	139-209	80	387
Nov	559	761	505-613	488-630	317	798
Dec	857	1054	783-931	760-955	600	1085

[a]These data are for the historical period October 1936 to September 1960.

[b]Upstream means above the dam.

Table 4. Mean-daily stream flows, Calapooia River[a], Downstream hydrologic data[b]

Month	Mean	St. Dev.	5% Interval about mean	1% Interval about mean	Median	Square root of second moment about median
Jan	1379	1740	1243-1515	1200-1559	771	1843
Feb	1141	1220	1040-1242	1009-1273	708	1294
Mar	733	822	674-793	649-818	426	878
Apr	350	440	314-385	303-396	194	466
May	163	262	142-183	136-190	86	273
Jun	44	51	40-48	38-49	33	52
Jul	10	16	9-12	9-12	11	16
Aug	0	11	-1-1	-1-1	0	11
Sep	1	24	-1-3	-2-4	0	24
Oct	85	537	43-127	30-140	55	538
Nov	555	1109	466-644	438-672	206	1162
Dec	1117	1578	993-1240	954-1279	575	1669

[a]These data are for the historical period October 1940 to September 1960.

[b]Downstream below the dam. Downstream was obtained by subtracting the upstream from the total flow at Albany.

The median, mean and other parameters of the upstream and downstream hydrology for the period of historical record are shown in Tables 3 and 4. The downstream parameters are based on a 20-year record, October 1940 to September 1960, and the upstream parameters on the 24 years from October 1936 to September 1960. In addition to the historical record, we concluded that the largest flood at Holley should have a maximum mean-daily flow of 15,000 c.f.s. (Table 2). From the historical frequency function and the one observation of an extreme value, an attempt was made to build a hydrology simulator that would generate an entire 100-year record including not only the one-in-100 years' flood, but also all the smaller floods of greater frequency which did not appear in the historical record. A 100-year record was desired because (1) Senate Document No. 97 allows this period to be used in calculation of benefits and costs, and (2) benefits and costs beyond one hundred years are effectively eliminated by discounting procedures.

In a DYNAMO model the hydrologic input is generated internally; i.e., equations are included which instruct the computer to draw numbers at random from specified distributions at specified sampling intervals. In our application to the Calapooia River we have used a combination of distributions (Figure 4).[25] For each simulated day a number is drawn from a normal distribution with the median used as the mean and other parameters as shown in Table 3 for the indicated months. This number represents upstream flows. Since there cannot be negative amounts of hydrologic input, a second number is drawn whenever the first draw is negative.

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[25] There are no doubt other ways of generating synthetic hydrologies, some of which may be more efficient than the one used here. Further research may develop alternative means.

The second number is drawn from a uniform distribution with a range from zero to the median of the frequency function. In this way, a piling-up effect (which corresponds more closely to the actual distribution of mean-daily flows) is achieved between zero and the median. Thus the skewed appearance of the frequency function is maintained.

The extreme values of a normal distribution in DYNAMO are limited to 2.4σ. Since the 100-year flow is beyond 2.4σ, it was necessary to attach another distribution to the first in tandem. A point less than 2.4σ was calculated and specified in the model. If on the original draw a number falls in this area (from the calculated point to 2.4σ, which we will call the critical range), a second number is drawn from a second uniform distribution. The second uniform distribution ranges from zero to the one-in-100 years' flow. In this way the entire spectrum of flows is developed.

Some Examples

Regular numbers. Suppose in December when the median flow is 600 c.f.s. and the second moment around the median is 1,085 c.f.s., a number is drawn from the normal distribution which is positive and between zero and the critical number, say 654 c.f.s. This number is used as the stream flow for that day.

Negative numbers. Now, suppose the number drawn is -640 c.f.s. Because there cannot be negative flow, this number is replaced by a number drawn from a uniform distribution between 0 and 600 c.f.s.

Critical numbers. When the number originally drawn is less than 2.4σ (3,203 c.f.s.) but greater than 2,997 c.f.s., the lower extremity of the critical range, it is replaced by a number drawn from a uniform distribution ranging from zero to 12,400 c.f.s., the estimated maximum mean-daily flow during the 1861 flood (see Figure 4).

Hydrograph

The daily sequence of flows arising from this method does not adequately resemble an actual hydrograph. A high mean-daily flow can be immediately followed by a low mean-daily flow, which is not realistic. To obtain information about the appearance of the actual hydrograph, the first differences between days of the historical record were calculated. No correlation was found between the magnitude of daily flows and the positive first differences; however, significant correlations were found between the magnitude or height of the hydrograph and the negative first differences. The relation between days for the negative first differences was estimated by a regression equation. The coefficients of these equations are given in Appendix Table B-1.

These relations were built into the program so that when a high flow occurred, the next day's flow could not be less than that specified by the regression equation. Thus, the smooth tapering-off appearance of the hydrograph was preserved. Should the mechanism generate a flow higher than yesterday's high, it would be used as that day's flow; thus the hydrograph would climb. A seven-day simulated hydrograph for a January week is shown in Figure 5.

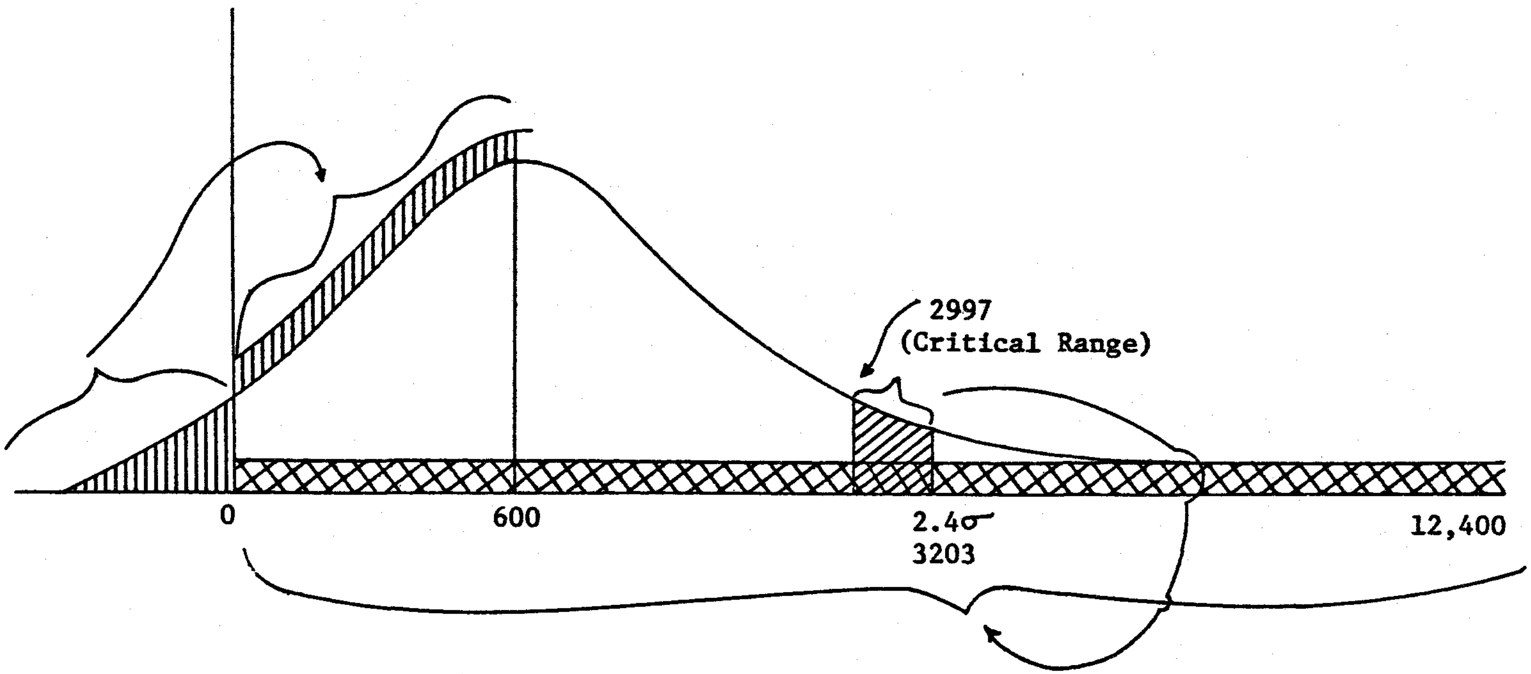

Figure 4. Simulated December frequency function, Calapooia River.

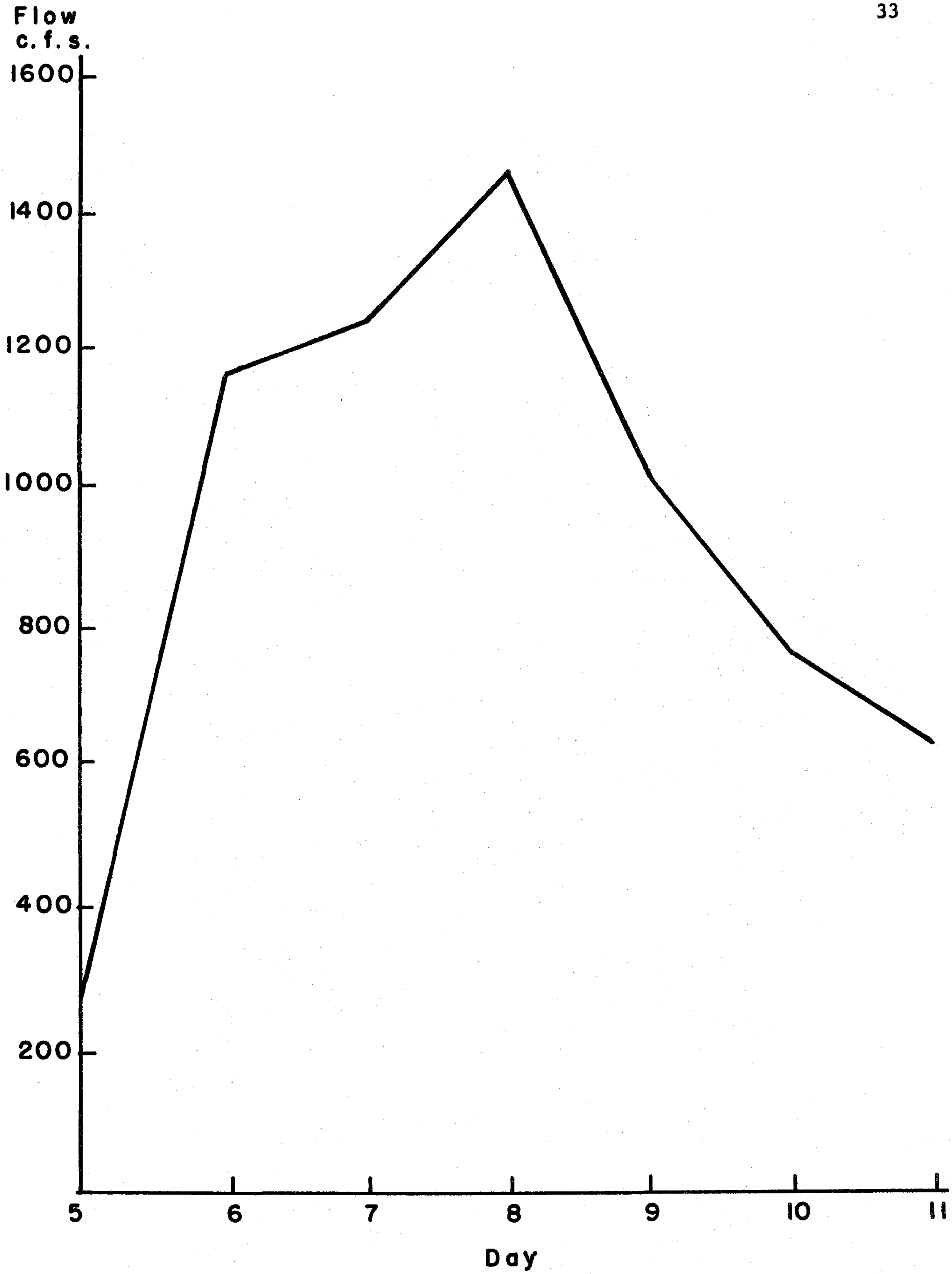

Figure 5. Seven-day simulated hydrograph, January 5-11, Calapooia River.

Upstream and Downstream Flows

In analyzing the historical record generally significant correlations were found between upstream and downstream daily flows. To obtain the downstream simulated flow the relation to the upstream flow had to be maintained. In the simulation, downstream flows were generated directly from the upstream flows through a regression equation.

The variance of downstream daily flows, especially during flood months, is large, due to the large drainage area comprising the downstream watershed. The coefficients of these equations are given in Appendix Table B-2. This variance was preserved by selecting an error term from a normal distribution with zero mean and a standard deviation equal to the standard error and adding it to the calculated flow from the regression equation. Table 5 shows how closely some characteristics of the simulated daily flows are to the historical record.

Table 5. Simulated mean-daily stream flows for Calapooia River

Upstream hydrologic data

Month	Means					
	100 Yr.	1st 20 Yr.	2nd 20 Yr.	3rd 20 Yr.	4th 20 Yr.	5th 20 Yr.
Jan	889*	835*	925*	967	887*	832*
Feb	955*	958*	965*	949*	1,000	902*
Mar	743*	773*	753*	745*	750*	696*
Apr	588*	589*	581*	623	595*	571*
May	379*	387*	413	367*	369*	362*
Jun	204*	203*	195*	216*	209*	196*
Jul	68	67	70	67	68	67
Aug	50	50	50	48	52	52
Sep	51	48	51	53	51	55
Oct	308	287	342	297	391	325
Nov	556*	580*	534*	552*	546*	567*
Dec	978	977	1,022	932*	945*	1,013

Downstream hydrologic data

Month	Means					
	100 Yr.	1st 20 Yr.	2nd 20 Yr.	3rd 20 Yr.	4th 20 Yr.	5th 20 Yr.
Jan	1,406*	1,276*	1,464*	1,545*	1,380*	1,364*
Feb	1,267*	1,268*	1,278	1,259*	1,377	1,154*
Mar	789*	821	810*	786*	795*	734*
Apr	379*	387*	362*	414	368*	364*
May	174*	182*	186*	167*	166*	167*
Jun	114	113	109	122	117	109
Jul	9*	8	9*	9*	9*	9*
Aug	4	3	3	3	4	4
Sep	5	5	5	5	5	5
Oct	219	203	239	206	207	240
Nov	502*	527*	497*	493*	497*	497*
Dec	1,209*	1,259*	1,232*	1,127*	1,171*	1,268*

* Indicates that mean falls within 1% interval about true mean from Tables 3 and 4.

Description of Projects

The proposed project authorized by Congress consisted of a dam to be constructed at Holley and channel improvement below the dam. The original design called for a dam to be constructed with an overall capacity of 97,000 acre-feet of which 90,000 acre-feet was usable and 7,000 acre-feet was dead storage. It was estimated that under the proposed plan, a flood such as occurred in 1861 could be regulated to about 16,000 c.f.s. at the mouth of the river. Natural channel capacity consists of 5,000 cubic feet per second. Thus, levees would be necessary approximately three miles above Brownsville to the confluence with the Willamette River at Albany to confine most flows to the channel.

In this study we will confine ourselves to five alternative projects: (R97C21) 97,000 a.f. reservoir and 21,000 c.f.s. channel capacity, (R97C11) 97,000 a.f. reservoir and 11,000 c.f.s. channel capacity, (R97C5) 97,000 a.f. reservoir and 5,000 c.f.s. channel capacity, (R67C21) 67,000 a.f. reservoir and 21,000 c.f.s. channel capacity, and (R67C5) 67,000 a.f. reservoir and 5,000 c.f.s. channel capacity.[26] The model, however, is completely flexible within the ranges specified and other alternative sizes of structures could be considered. These size structures were selected because the Corps of Engineers had estimated the maximum use benefits and the structure cost which can be expected from these projects.

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[26] It should be noted, however, that the Corps in envisioning the needs of all of the beneficial uses of water has estimated that a dam of 160,000 acre-feet or more may be needed.

In order to evaluate the alternative projects, functions relating costs to reservoir and channel sizes were provided by the Corps of Engineers. More specifically, points along the functions were provided. The annual costs for the three alternative reservoir and channel sizes are shown in Tables 6 and 7.

The benefits from the beneficial uses of water considered in this study were also provided by the Corps of Engineers. These benefits as estimated or provided by the Corps of Engineers specify the benefits to the water-system project if all of the use capability is met. In other words, if all of the irrigable acreage has 100 percent of its need met, then 100 percent of the maximum irrigation benefits are obtained. Generally, however, within a project competition exists among alternative water users for the available water and hence the irrigation benefit will likely be less than 100 percent. Larger projects can be built to provide additional water, but only at a cost. The larger project can only be economically justified if the increment of benefits is greater than the increment of cost.

Beneficial Uses

Each size of water project supplies its own combination of multipurpose benefits. Therefore, it is important from an economic standpoint to test alternative size project to obtain as nearly as possible the greatest net benefits. This is what is attempted in this study.

Irrigation benefits are a function of storage capacity. Flood control and fish benefits are dependent not only on reservoir size but also on channel size. Drainage, on the other hand, is related to size and depth

Table 6. Reservoir costs, Calapooia River, 1964 dollars

Reservoir Sizes (acre-feet)[a]	67,000	82,000	97,000
Capital costs			
Construction cost/acre-foot	220.00	210.00	200.00
Total construction cost	13,200,000.00	15,750,000.00	18,000,000.00
Annual cost			
Amortization and interest[b]	417,736.00	498,435.00	569,640.00
Operation, maintenance and repair[c]	79,200.00	94,500.00	108,000.00
Total annual cost	496,936.00	592,935.00	677,650.00

[a]Included in each reservoir size is 7,000 acre-feet of dead storage.

[b]Assumes a 3 percent interest rate and a life span of 100 years.

[c]Calculated at 7.5 percent of amortized costs.

Table 7. Channel costs, Calapooia River, 1964 dollars

Channel sizes (c.f.s.)	5,000	11,000	21,000
Total construction cost[a]	100,000.00[d]	1,600,000.00	8,000,000.00
Annual costs			
Amortization and interest[b]	3,164.00	50,634.00	253,173.00
Operating, maintenance, and repair[c]	316.00	5,063.00	27,317.00
Total annual cost	3,480.00	55,697.00	278,490.00

[a]As estimated by the Corps of Engineers.

[b]Assumes a 3 percent interest charge and a life span of 100 years.

[c]Calculated at 10 percent of amortized costs.

[d]While 5,000 c.f.s. is the natural channel capacity certain costs would be necessary in order to make it at all usable for reservoir discharges.

of channel. Each of these beneficial uses will be discussed in the subsections to follow.

It should be kept in mind that all benefits and cost in this report are considered as approximations. As previously stated, the Corps of Engineers has not completed its analysis of the Calapooia River basin.[27]

Fish Life

The Calapooia River presently does not provide good sport fishing. The low volume, sluggish moving flows occurring in late summer and early fall are not suitable for most game species. However, a few Chinook salmon and steelhead trout have been observed. Increased flows provided by regulated releases from the proposed reservoir during the low-water period would benefit materially fish life. An egg collection station below the dam has been proposed by the State Game Commission. Total annual costs of the structure as estimated by the Corps of Engineers would be $105,317 producing annual maximum benefits of $530,000. Table 8 gives the breakdown of the costs and maximum expected benefits. In simulating the releases for fish life, the difference between the fish requirement and the natural minimum monthly mean-daily flow was used as the amount of water needed and released.

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[27] Considerable discussion and criticism has ensued concerning the proper techniques and methods to be employed by federal agencies in deriving benefits and costs for a proposed project. Some of this criticism is justified while some is not. But it is not the purpose of this report to enter this discussion; rather it is our purpose to test simulation as a tool in making management decisions; therefore, the maximum use benefits and cost as provided by the Corps were accepted and used without modification.

Table 8. Fishlife enhancement, costs and maximum benefits, Calapooia River

Costs:	
Total construction[a]	$800,000
Annual costs	
Amortization and interest[b]	25,317
Operation, maintenance, and repair[c]	80,000
Total annual costs	$105,317
Benefits:	
Maximum annual benefits[a]	$530,000

[a]As estimated by the Corps of Engineers.

[b]Assumes a 3 percent interest charge and a life span of 100 years.

[c]Calculated at 10 percent of total construction cost.

The water requirements for the proposed fishery vary with the time of the year. The trout fishery requires moderate amounts of water through the entire year, but salmon and steelhead runs which occur from late summer on through the winter require large amounts of water during that period (see Table 9). The observed minimum channel flows also vary by months. There are periods when the natural channel flow is not adequate to meet the fish requirements. Thus, for each month a different flow must be released from the reservoir if fish life is to be preserved or maintained undiminished.

There is an average delay of one day between flows at Holley and Albany. During the delay, water flows into the channel from the unregulated drainage area below the dam. A priori one cannot predict what these unregulated flows will be; therefore, the entire difference between minimum natural flows and fishery needs are released. If one could predict this inflow, the exact amount necessary to meet the fishery need could be determined and released.

In addition, the amount of water that can be released at any specified time depends upon the level in the reservoir. Part of the need is met by the unregulated flows available in the channel. If the need cannot be met by reservoir releases and by the unregulated flows, the benefits of fish life decrease. The minimum mean-daily flow, occurring in the channel during any one year, divided by the requirement establishes the percentage of fishery requirement met.

The fishery benefits are a function of the percentage of the fishery requirement met. This function is shown in Figure 6. If

Table 9. Fishlife requirements and channel minimum flows, Calapooia River

Month	Downstream minimum flow (c.f.s.)[a]	Fishery requirement (c.f.s.)[b]	Fishery need (c.f.s.)[c]
Jan	55	140	85
Feb	90	140	50
Mar	60	140	80
Apr	25	140	115
May	0	140	140
Jun 15	0	90	90
Jun 30	0	70	70
Jul	0	50	50
Aug	0	50	50
Sep 15	0	50	50
Sep 30	0	160	160
Oct	0	160	160
Nov	0	140	140
Dec	0	140	140

[a]Estimated minimums from the 20 years of historical data available.

[b]As obtained from the Corps of Engineers.

[c]Need is the difference between requirement and minimum flow.

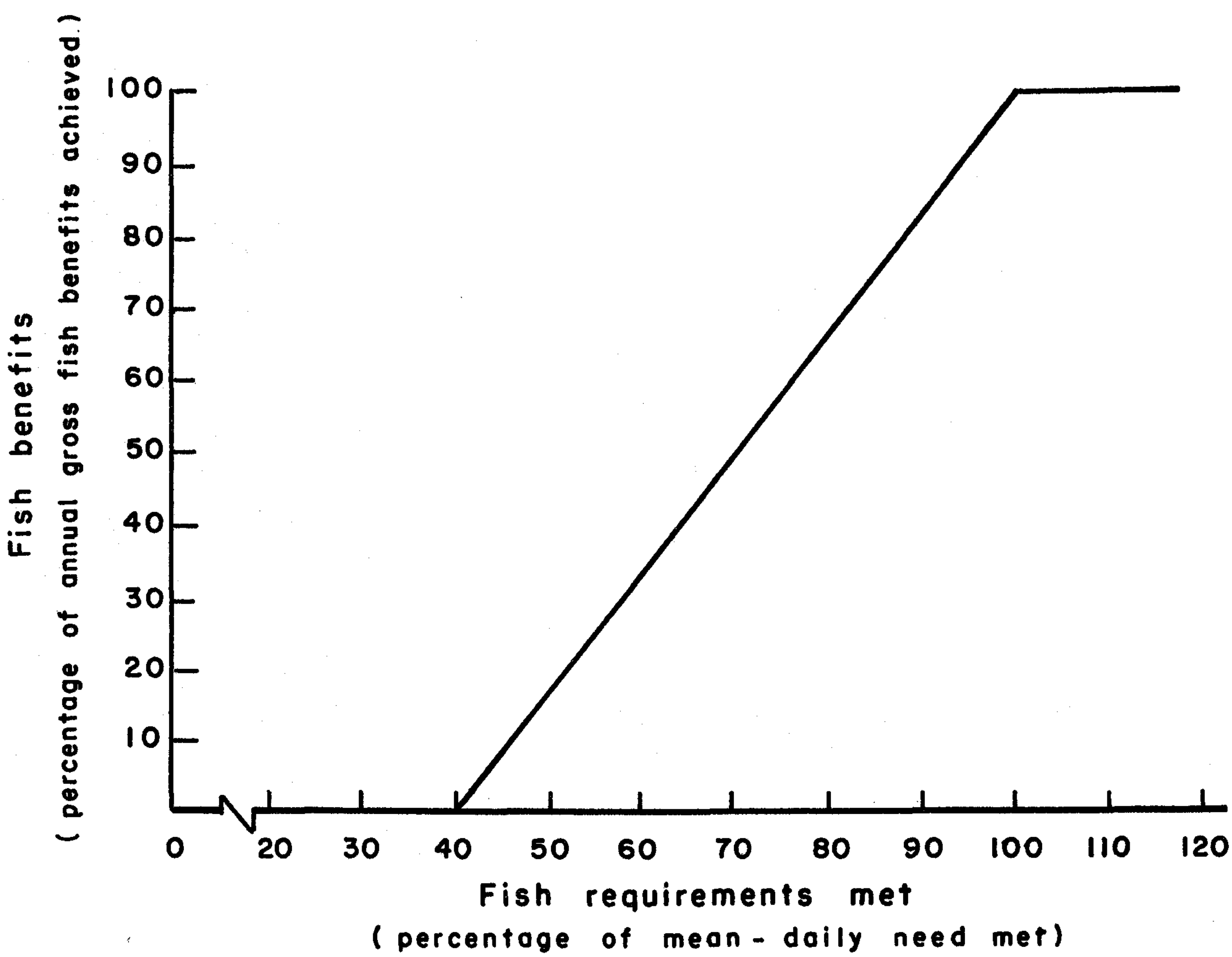

Figure 6. Fish life benefit function, Calapooia River.

the percentage of requirement met falls below .4, no fish benefits are obtained, from .4 to 1 the benefits are linearly dependent upon the percentage of requirement met and beyond 1 all the benefits are obtained. This function is arbitrary. The simulation program, however, was written so that other functions relating to fish benefits could easily be incorporated into the model.

For the purpose of this study, the fish benefits are assumed to be serially unrelated. That is, failure to achieve this year's benefits does not affect in any way next year's benefits. This assumption needs additional consideration. Low flows not only endanger the survival of this year's fish, but affect the number of fish available in subsequent years. The number of anadromous fish returning to the inland streams varies with the hatch from previous years. Associated with the level of storage, therefore, are not only the losses from the shortage years itself, but also losses extending over several additional years.

Irrigation

Irrigation in the Calapooia River basin has increased three-fold in the past 10 years. Increased irrigation acreage will continue to be developed as irrigation water is made available. However, there are certain physical and economic constraints which limit its continued growth in any specific area. The Corps of Engineers provided estimates from the Bureau of Reclamation that there are 53,400 acres of Class I, II and P lands[28] in the two proposed irrigation areas of Brownsville and Calapooia

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[28] Class I and II lands have no, or easily corrected, natural conditions that limit intensive cropping and irrigation.

that could be irrigated by water from the Holley Reservoir. The monthly irrigation need is listed in Table 10.

The total irrigation diversion required for the two areas amounts to 73,000 acre-feet; however, 3,100 acre-feet of the Calapooia area requirement can be met by the return flow from the Brownsville area. Thus, only 69,900 acre-feet of water are needed for the combined areas. The maximum annual net irrigation benefits are listed in Table 11. These are net except for a charge needed to convey the water to the farm gate. The total capital costs and annual costs for these conveyance structures are also listed in Table 11.

Irrigation shortages occur when there is not enough water available to meet the need. The water available varies with the capacity of the reservoir and also with the natural stream flows below the dam on any particular day. When irrigation shortages occur or the need is not met, the irrigation benefits decrease. The effects of these shortages on yield and subsequently on gross returns is dependent upon the intensity, timing, and duration of the shortage. However, considerable information would be necessary in order to adequately simulate these phenomena, information which presently is not available. Therefore, in the simulation model irrigation benefits are a linear function of the percentage of the need which is met. For example, if in one year the percentage of need met is 80 percent, the irrigation benefits for that year are $442,152.00 (.80 x $552,690).

The simulation model runs continuously over a 100-year period. However, one would not ordinarily expect all of the potentially irrigated acreage to be ready for irrigation upon completion of a project. Hence, the model allows for growth of irrigation needs over a 20-year period.

Table 10. Irrigation diversion and return flow for Class I, II, and P lands, Calapooia River

Month	Diversion (project) (acre-feet)	Return flow (acre-feet)
Brownsville area:		
Apr	400	200
May	1,000	300
Jun	2,400	600
Jul	4,000	700
Aug	3,500	700
Sep	700	600
Total	12,000	3,100
Calapooia area:		
Apr	1,900--- 1,700[a]	300
May	4,700--- 4,400	900
Jun	12,200---11,600	1,400
Jul	21,500---20,800	1,800
Aug	18,500---17,800	1,600
Sep	2,200--- 1,600	1,400
Total	61,000 57,900	7,400
Combined total	73,000 69,900	10,500

[a]Need less return flow from Brownsville area.

Table 11. Irrigation costs and maximum use benefits, Calapooia River, 1964 dollars

Irrigation capability (acres)[a]	53,400.00
Construction cost per acre[a]	17.44
Total construction cost	931,296.00
Annual cost:	
Amortization and interest[b]	29,472.00
Operation, maintenance, and repair[c]	2,210.00
Total annual costs	31,682.00
Benefits:	
Irrigation capability (acre)	53,400.00
Annual net benefits/acre	10.35
Total annual net benefits	552,690.00

[a]As furnished to the Corps of Engineers by the Bureau of Reclamation.

[b]Assume a 3 percent interest charge and a life span of 100 years.

[c]Calculated at 7.5 percent of amortized costs.

[d]In 1947 dollars, the irrigation benefits were $12.10 per acre. The relation between the 1947 and 1964 prices received indices indicates that 1964 prices were only 85.5 percent of 1947 prices. Thus, 1964 irrigation benefits were assumed to be only $10.35 per acre. This approach to finding 1964 irrigation benefits per acre is arbitrary, but for the purposes of this study it was deemed adequate.

At the completion of the construction, the irrigation acreage is assumed to be one-twentieth of the total acres which are to be brought into production. Each year thereafter the irrigation acreage increases by one-twentieth, until at the end of the twentieth year all 53,400 acres are irrigated. Of course, in the first 20 years the irrigation need is proportional to the acreage being irrigated, as are the irrigation benefits. Thus, irrigation benefits increase as does the irrigation acreage until the twentieth year. Thereafter, benefits are directly related to the percentage of the need met.

Drainage

The soils predominating in the Calapooia River basin require, for the most part, artificial drainage to obtain maximum production. These soils are: Amity, Dayton, and Wapato with smatterings of Chehalis, Newberg, Willamette, and Woodburn.[29] In order to drain these soils, adequate outlets for drainage are needed. The Calapooia River channel could serve such a purpose.

Drainage benefits are entirely dependent upon the level of the channel during the drainage period and the channel size (see Table 12). In the simulation model it was assumed that drainage was critical during the four-month period of March, April, May and June. The channel size influences the number of outlets and subsequently the number of acres which may discharge into the channel. The level in the channel during the drainage

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[29] Willamette, Woodburn, Amity, and Dayton are members of the Willamette catena of soils. The Willamette catena consists of old alluvial soils on the gentle sloping terraces of the basin. Chehalis, Newberg, and Wapato are recent soils deposited by water action. They are located close to existing streams of water.

Table 12. Drainage maximum annual benefits, Calapooia River, 1964 dollars

Channel size	Maximum annual benefits[a]
C.F.S.	Dollars
5,000[b]	0
11,000	200,000
21,000	500,000

[a]As estimated by the Corps of Engineers.

[b]Natural channel size.

period determines how effective the outlets will be in removing the water from the land.[30] In order to obtain the benefits for any particular drainage season, the average level within the channel is calculated in the computer program. In addition, a function which relates the average channel level to benefits obtained is also included in the model.

Figure 7 gives a graphic illustration of the drainage benefit function. The function shows that as long as the average level in the channel does not exceed 30 percent of channel capacity all of the benefits would be obtained. Thereafter, until the channel was 60 percent full, a 10 percent increase in the average level resulted in a 20 percent decrease in drainage benefits. From 70 to 100 percent full, drainage benefits decreased 10 percent as the average channel level increased 10 percent. When the channel was completely full or overflowing, no drainage benefits were forthcoming.

Flood Control

Definition of a flood. Reduction of flood damage is one of the principal aims of the Calapooia water development project. In our simulation system only the largest flow of the year is called a flood and is used to calculate flood damages. Major floods occur only during the three winter months, December, January, and February. This assumption conforms to the historical flood record. It is recognized that there

[30] On the other hand, the improvement of drainage may affect the shape of the downstream hydrograph by making possible a more rapid run-off. No account was taken of this in the model.

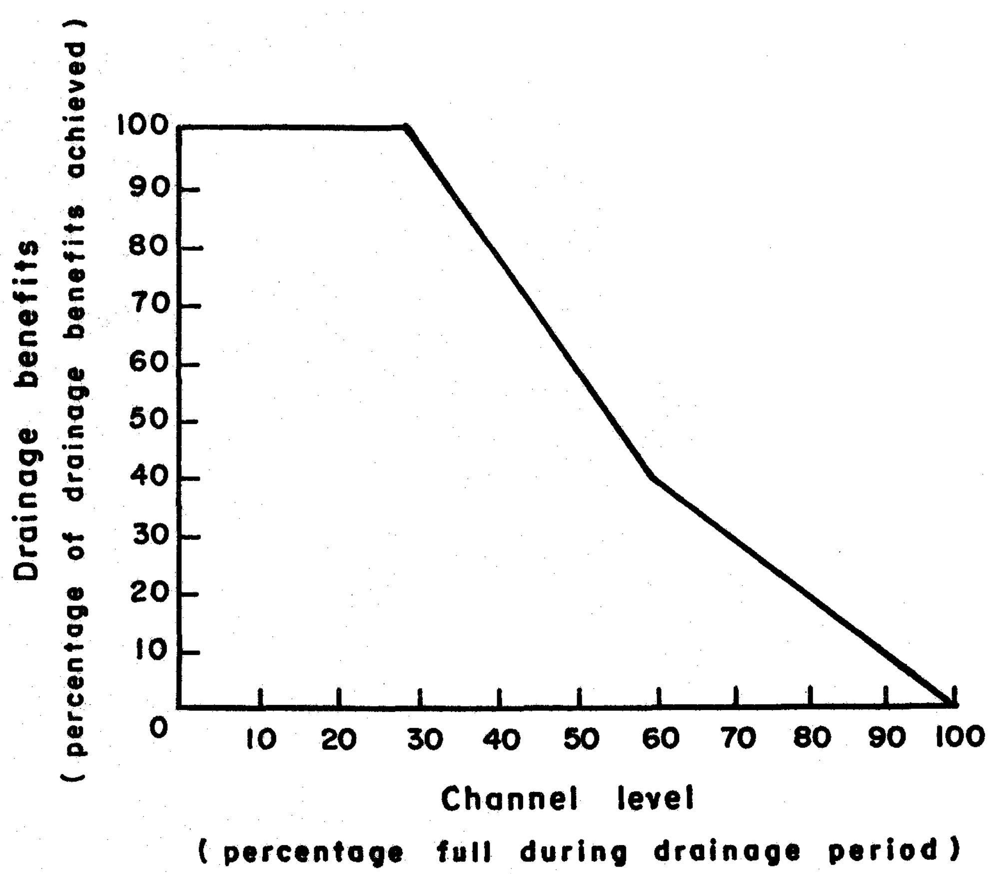

Figure 7. Function relating drainage benefits to channel level.

exists a possibility of more than one flood in any given year. Two or more floods have occurred in the past and undoubtedly will occur in the future. However, for floods of comparable magnitude most of the damages occur during the first flood. As the water overflows the banks, a block of damages results. Another flood covering the same area would cause considerably less additional damage.

Flood damage function. Flood damage can occur at points along the river between the dam at Holley and the confluence of the Calapooia with the Willamette River at Albany. However, the Corps of Engineers uses flood stage at Shedd as an indicator of the extent of flood damages for the area. In the simulation model a flood-damage function is used based upon the flood stage in feet at Shedd, Oregon. It is based on 1964 stage of economic development. The function used is shown in Figure 8.

These flood damages are assumed to be a result of instantaneous peak flow. In the simulation the flows resulting from the hydrology generator are mean-daily flows in cubic feet per second. A regression equation is used to convert these mean-daily flows into instantaneous peak flows. The instantaneous flows are converted into stage height in feet at Shedd.[31] Until the river is 10 feet at Shedd there are no flood damages; thereafter the banks are overtopped and damages result. As more water overflows the bank the area of inundation widens and damages increase according to the function.[32]

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[31] During an actual flood, the month of year, velocity of flows, and duration of the inundation may be as important or more important than the depth.

[32] For each channel capacity the conversion is different; i.e., a larger capacity requires a larger flow to obtain the same stage height of a small flow in a smaller channel.

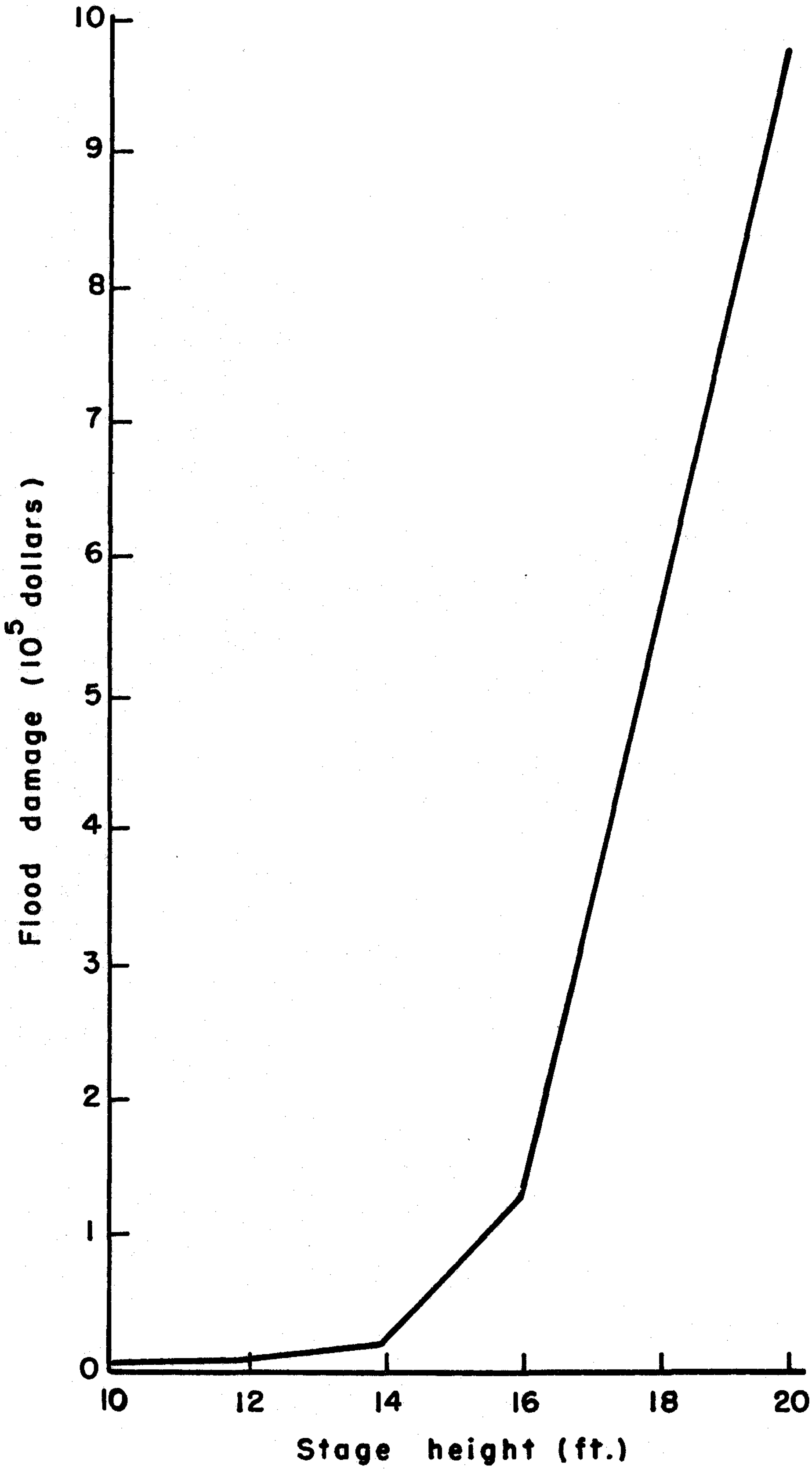

Figure 8. Function relating flood damages to stage of river at Shedd, Oregon.

Calculation of damages. Internal to the program are a number of accounting equations which keep track of both the natural flow (unregulated) and the regulated flow. The unregulated flow occurs in the absence of any man-made restrictions of the water movements. It consists of the daily downstream inflow plus the daily upstream flow lagged one day. The regulated flow at Shedd is the downstream flow plus the flows released from the reservoir appropriately lagged.

In our model the flood control benefits achieved by the dam and its operating procedure are the differences in damages associated with the regulated and unregulated flow as read from the flood damage function. The damages from the regulated flow are subtracted from the unregulated flow damages.

Evaporation and seepage. Several general assumptions related to the reservoir were made in analyzing the system. First, constant evaporation rates were assumed for the reservoir regardless of its size. Table 13 gives the monthly evaporation rates. Second, account was not taken in the model of loss of water through seepage or loss of storage through sedimentation. While it is recognized that both factors exist, no estimates were available of their magnitude. Third, while 7,000 acre-feet of water exists as dead storage, no additional water is provided to maintain a permanent pool for fish life, recreation, etc., and thus, no benefits occur to the reservoir from these sources.[33]

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[33] The model would need slight modification to incorporate these beneficial uses.

Table 13. Evaporation from the Holley Reservoir, Calapooia River

Month	Amount (acre-feet)
April	.3
May	.6
June	.7
July	.8
August	.6
September	.3
Other months	0

Operating Procedure

In order to estimate the benefits of a system over time it is necessary to construct a set of rules for storing and releasing water. The operating procedure and the size of the project are not independent. Modifying the operating procedure may be economically more important than modifying the size of the project. It is conceivable, of course, to construct a project so big that all possible beneficial uses are fulfilled, thereby simplifying the operating procedure. But, it is doubtful that such a system would be within any realistic cost constraints. The more general situation would be that proper consideration of the operating procedure would allow the size of the project to be diminished without reducing net benefits.

Basic to the operating procedure is the rule curve which specifies how full the reservoir should be at any point in time. This does not necessarily mean that the level of the reservoir will be at the level specified by the rule curve at all times. However, it is desirable to return to the rule curve as conditions permit.

In developing the rule curve shown in Figure 9, the Corps of Engineers considered only flood control and the need to have the reservoir filled by May 1 to meet the irrigation need. However, in developing an operating rule curve all beneficial uses of the project must be considered. Different rule curves for the same size of project could give considerably different net benefits. Simulation may be the only tool capable of exploring alternative rule curves.

In the simulation model, water releases for irrigation and fish needs have priority over retention of water to return to the rule curve. That is, even if the level in the reservoir is lower than the rule curve, water will be released for the beneficial uses. However, if the level in the reservoir is greater than the rule curve, water will be released to return to the rule curve in order to provide flood-control space regardless of need for use of stored water. However, the releases will be used to meet any beneficial needs which may exist at that time. If insufficient water is released to meet the needs, in returning to the rule curve, the additional will be released as in the first case.

No priorities are given to the two water uses - fish life and irrigation; the two uses are entirely independent. However, irrigation releases are modified by allocating the water in the reservoir among the days of the month on an equal-day basis. Releases for fish life are unmodified but are made as long as water is available in the reservoir for release.

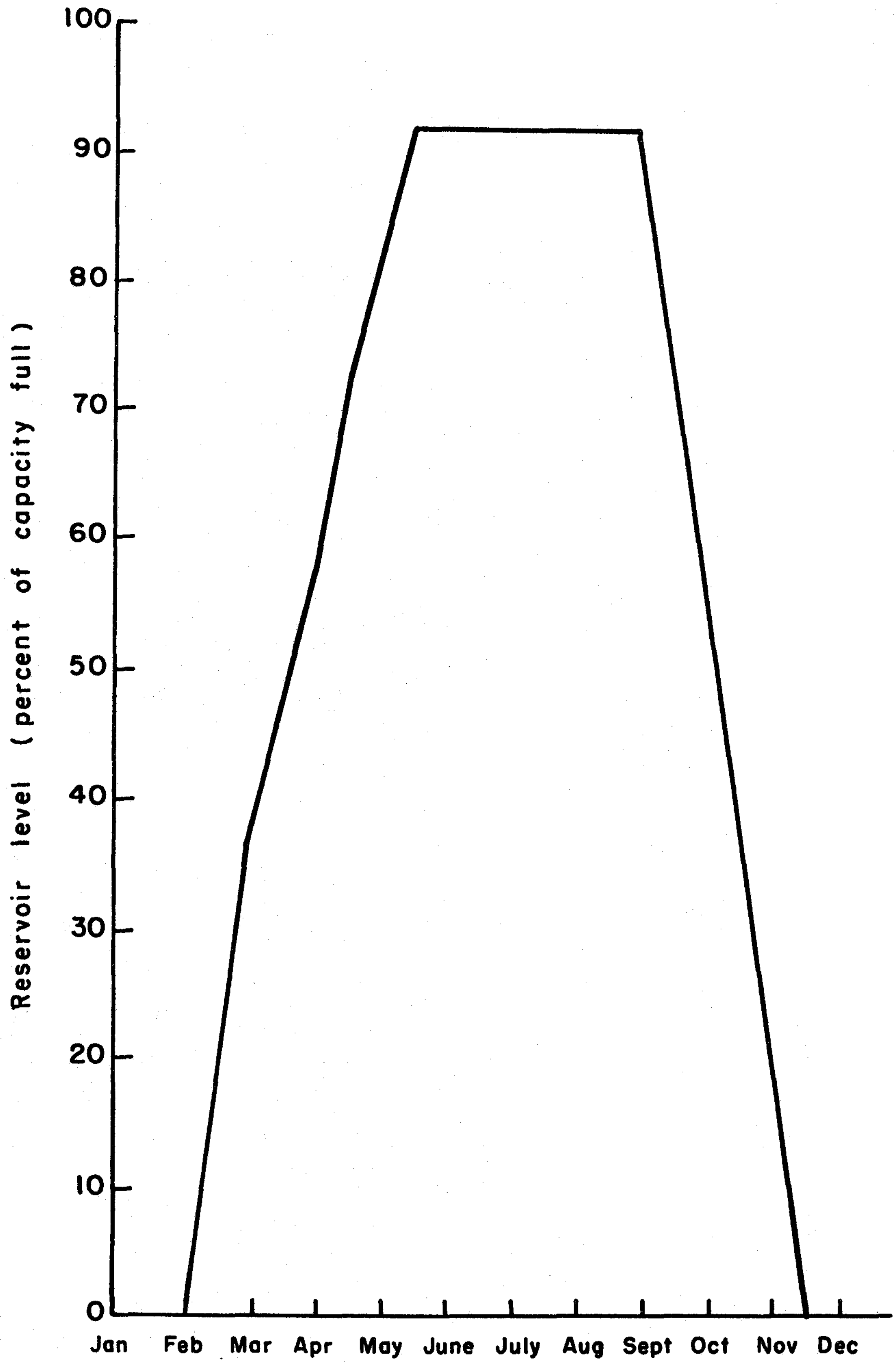

Figure 9. Rule curve for Holley Reservoir, Calapooia River.

An alternative procedure would be to attach priorities to fish life and irrigation. For example, the total amount of water available for the critical period could be estimated and allocated equally to the uses in order not to completely deplete the supply before the end of the draw-down period. If insufficient water is available to meet both the needs, water will be released for both uses until the available supply is exhausted.

Part IV

Some Results of Simulation

The model described in the preceding sections was run on the IBM 7094 computer at the University of California in Berkeley.[34] The model will simulate the benefits arising from control of natural hydrology of the Calapooia River basin over any period of time desired; for our purposes a one hundred period was selected. Changes in design variables and operating rules are reflected in the changes of the accumulated net benefit from the system. For each modification in the model the same hydrology is generated in order that the effects can be compared.

To present a summary of the voluminous data obtained from each computer run, the following will be divided into three subsections.[35] The first summarizes and compares some of the hydrologic aspects of the simulation. The second subsection summarizes the effects of several project sizes on total net benefits. The third considers changes in the management policy or operating procedure.

Hydrology: Flood Frequency and Control

In a preceding section the simulation of the hydrology was discussed. Also, some of the numerical characteristics of the simulation were given. Here we will discuss flood frequency and control. Of the beneficial water uses analyzed in this study, flood control is the only one greatly

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[34] The computer program is presented in Appendix A.

[35] Each run of the model provides many circumstantial details which no doubt would be of great value to decision makers and planners in an actual situation. Publication of such details is beyond the scope of this report.

affected by natural flows. Fish life, irrigation, and even drainage are influenced to minor degrees by unregulated flows; however, for the most part, these uses are absent or minimal in the natural condition. The height of the river at Shedd is called the river's stage height.[36] The maximum stage heights, unregulated and regulated, for each year over the 100-year simulated record for R97C21, R97C11 and R97C5 are shown in Figures 10 to 12, respectively. The black shaded area is the difference between the unregulated and regulated stage heights. The height of the unshaded area is the regulated stage height. The black shaded area plus the unshaded area is the unregulated stage height. For example, in Figure 10, the lowest unregulated stage height (13.1) occurs in year 78 of the simulated record; the highest (17.2) occurs in year 41. During year 78 the regulated stage height is 10.4 (the height of the unshaded area) and the difference (controlled area) is 2.7 feet (the black shaded area). The regulated stage height is 15.2 and the difference is 2.0 feet during year 41. As one reviews the three figures it is apparent that the size of channel is a major factor in flood control.

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[36] A table showing the numerical values of the flood stages at Shedd for unregulated and regulated flows is given in Appendix B.

It is interesting to note that the flood stages at Shedd, Oregon, are generally, but not always, the same for projects R97C21 and R67C21 and projects R97C5 and R67C5. This is not all surprising when it is noted that R97C21 and R67C21 both have channel capacities of 21,000 c.f.s., while projects R97C5 and R67C5 have channel capacities of 5,000 c.f.s. Downstream water cannot be regulated by the dam and it is the downstream flows which contribute most to the floods. During most floods the upstream flow is controlled by the reservoir regardless whether the reservoir capacity is 67,000, or 97,000 acre-feet. Only in very severe floods does water from the reservoir contribute to the flood downstream. In this situation, the reservoir size does make a difference. More water can be contained by the big reservoir.

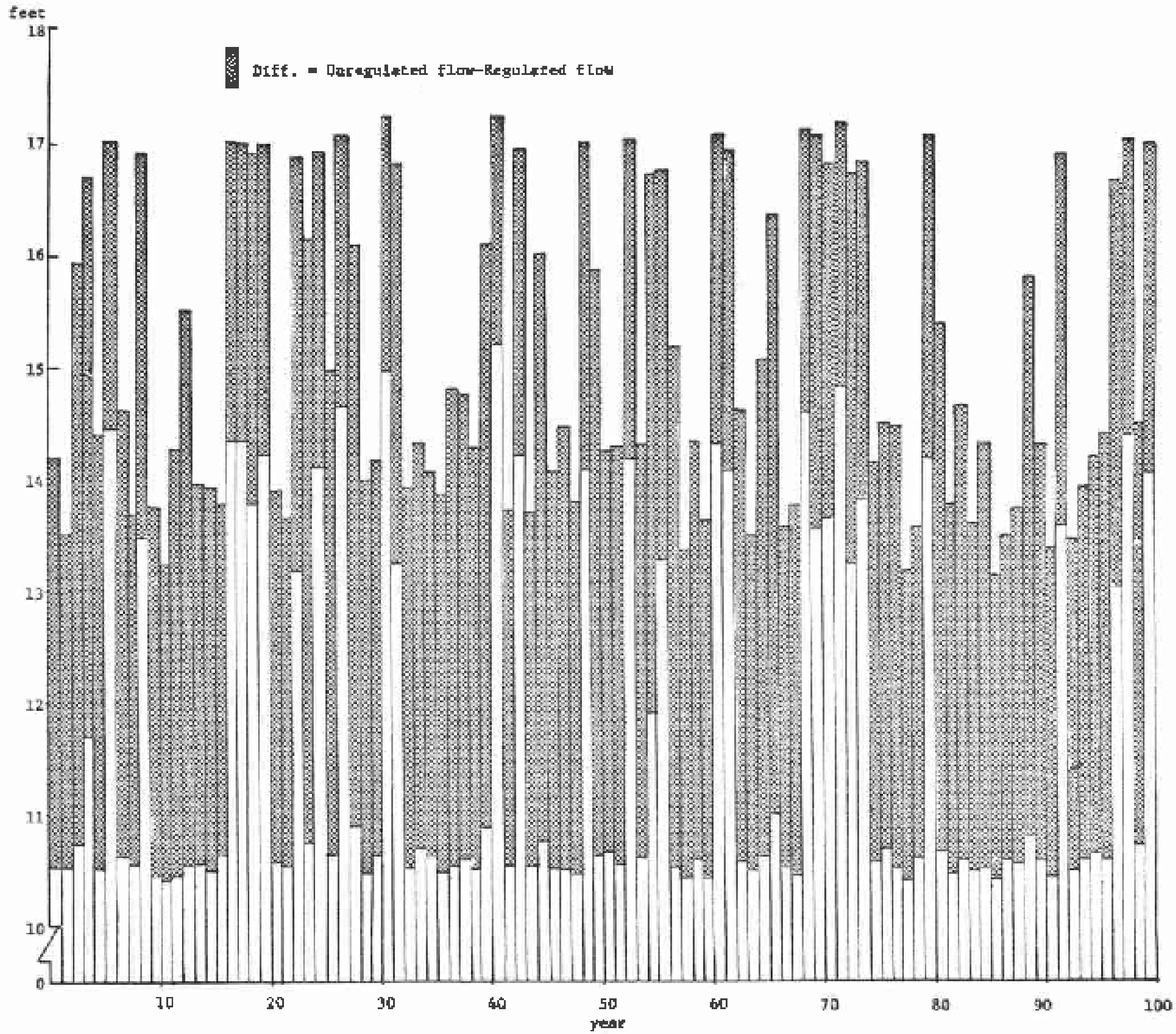

Figure 10. Simulated maximum stage heights, unregulated and regulated, for 97,000 a.f. reservoir and 21,000 c.f.s. channel.

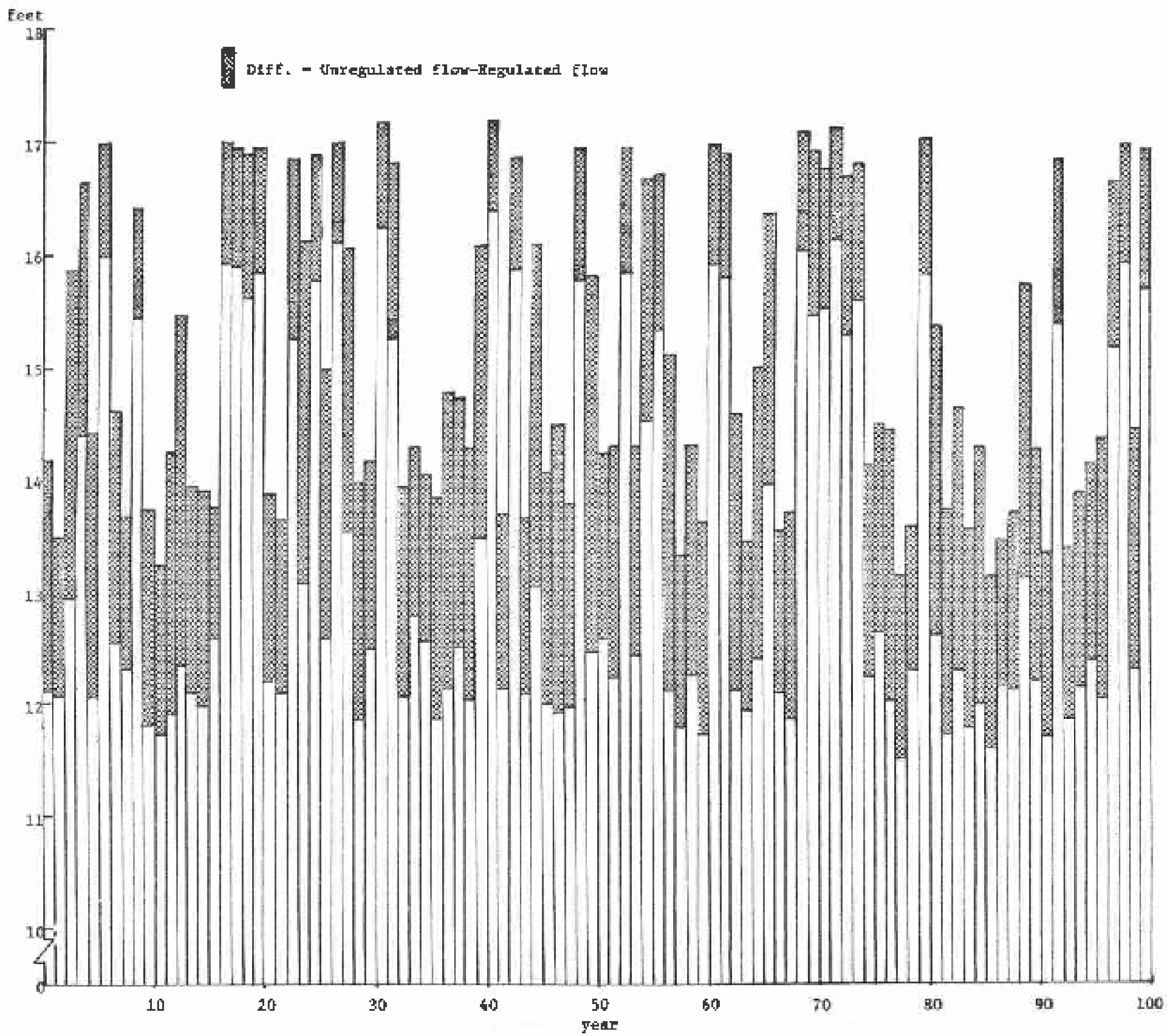

Figure 11. Simulated maximum stage heights, unregulated and regulated, for 97,000 a.f. reservoir and 11,000 channel c.f.s.

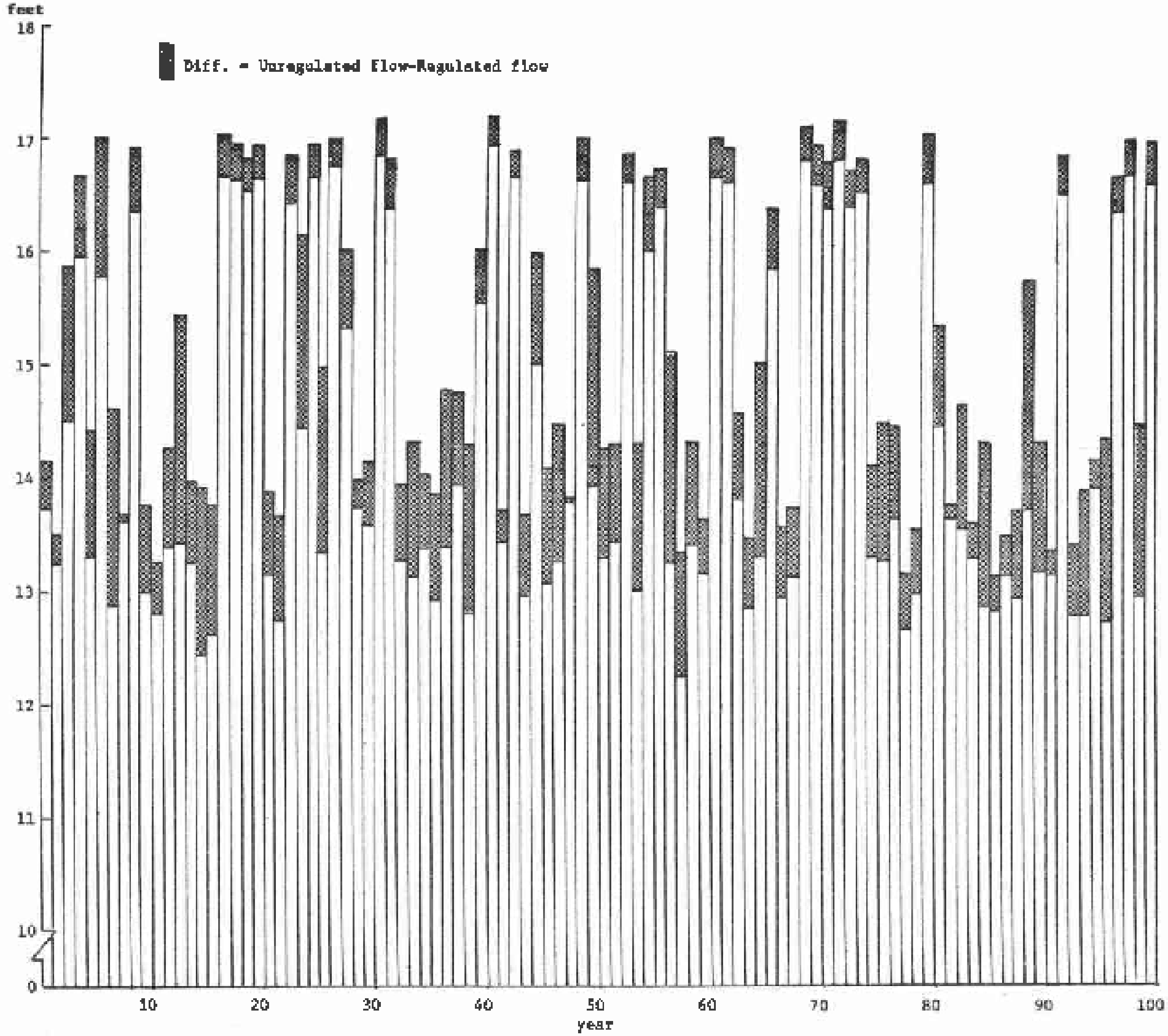

Figure 12. Simulated maximum stage heights, unregulated and regulated, for 97,000 a.f. reservoir and 5,000 c.f.s. channel.

Exceedance frequency curve

The Corps of Engineers has developed an exceedance frequency curve for unregulated flows from limited recorded data and estimated historical floods. The frequency curve gives the average recurrence interval between floods of given magnitude (see Figure 13). For example, from the Corps of Engineers' exceedance frequency curve we read that a maximum instantaneous discharge of 9,100 c.f.s. can be expected to be exceeded on the average of once in every two years. The one-in-100 years' flow is 50,000 c.f.s. That is, 50,000 c.f.s. is expected to be exceeded only once in 100 years.

A similar frequency curve from the simulated hydrology was developed. The simulated exceedance frequency curve is given in Figure 14. The regression lines for the two frequencies are almost identical. However, the variance is greater for the simulated hydrology. It should be remembered, however, that the simulated exceedance frequency is composed of all floods over the 100-year period, not just a short period of record with a few additional estimated floods. Hence, it is difficult to make a direct comparison of the two curves. Whether or not the simulated curve represents the decision makers' concept of reality and of possible and probable events of the future only he can decide. Other exceedance frequency curves can easily be generated. Ours is only intended to be illustrative.[37]

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[37] Since a simulation model for decision-making purposes must be developed for a specific decision-making framework, the planners and decision makers should be in constant contact with the model developers. Together, they must decide upon the acceptability and validity of the model and its integral parts.

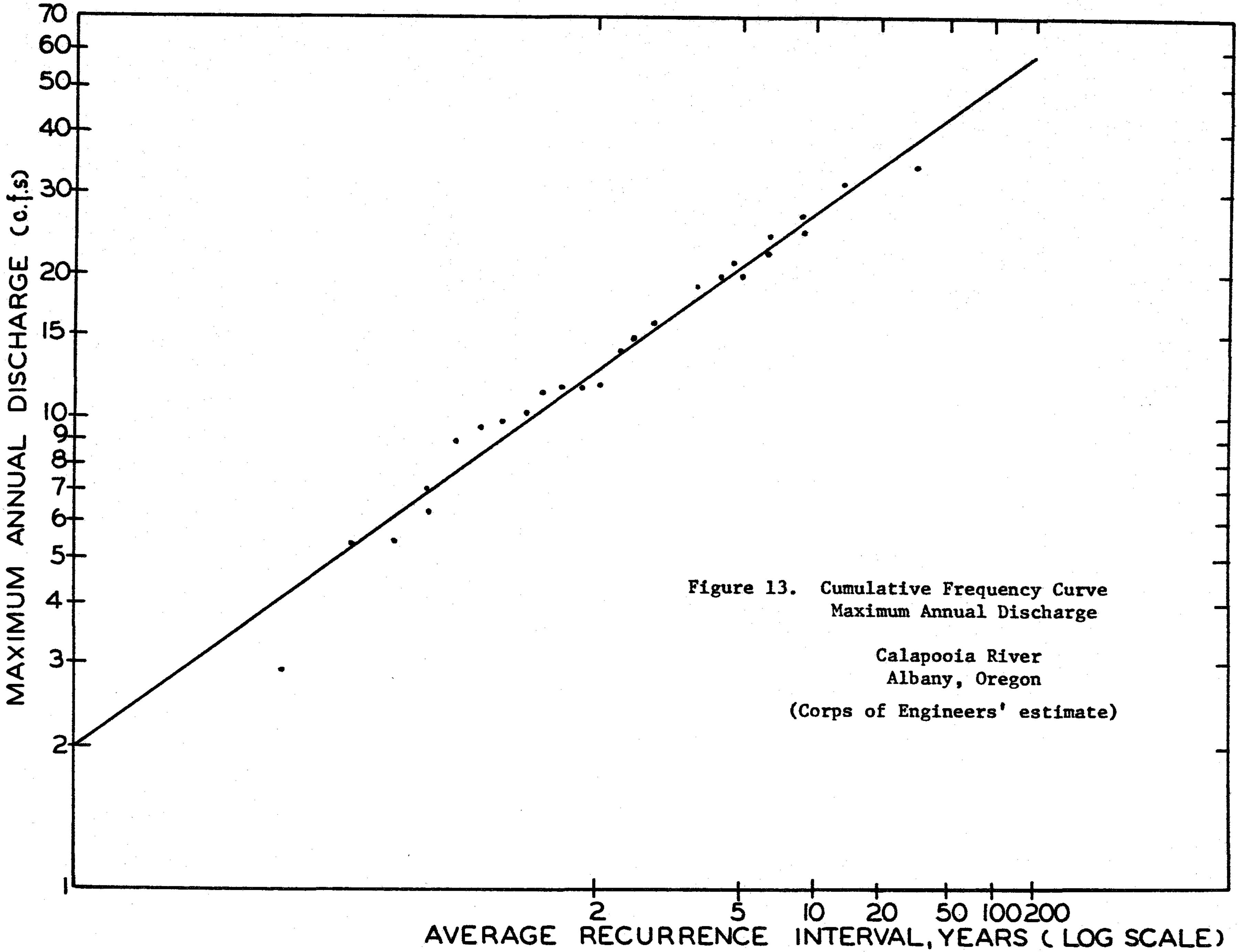

Figure 13. Cumulative Frequency Curve
Maximum Annual Discharge

Calapooia River
Albany, Oregon

(Corps of Engineers' estimate)

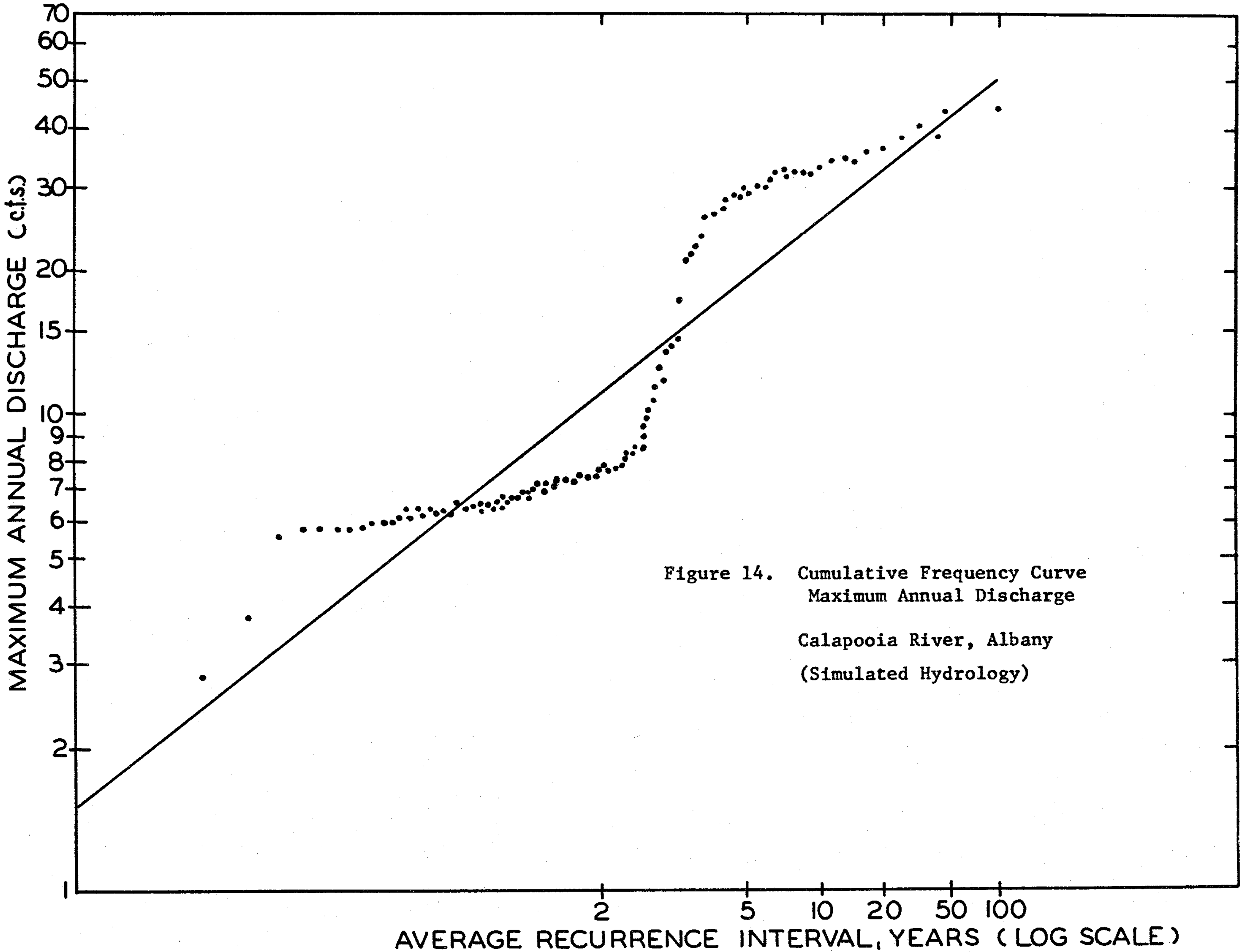

Figure 14. Cumulative Frequency Curve
Maximum Annual Discharge

Calapooia River, Albany

(Simulated Hydrology)

Alternative Projects

As previously indicated, one of the objectives of this study was to test the consequences of constructing alternative size projects. The comparative measure of effectiveness of alternative sizes is accumulated annual net benefits, that is:

$$\sum_{t=1}^{T} B_t - \sum_{t=1}^{T} C_t + K/(1+i)^t$$

Where:

B_t = benefits received annually

C_t = operating, maintenance, and repair costs incurred annually

K = fixed investment in structures

i = interest rate

T = life span of the project.

Table 14 shows the accumulated net benefits, benefit-cost ratios and a breakdown of benefits by use for the five alternative size projects using a single operating procedure.[38]

Project R97C21 is the largest in terms of cost of the five sizes tested. It also provides the highest accumulated annual net benefits and benefit-cost ratio. The second largest project in terms of accumulated total annual costs is project R67C21. It has accumulated net benefits of \$20,241,000 which is \$4,067,000 less than the accumulated net benefits

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[38] These projects were selected for analysis because they are the projects for which the Corps of Engineers had specified the maximum use benefits and construction costs.

from project R97C11. Project R67C5 is the smallest of the projects and the only one with negative accumulated net benefits or a benefit-cost ratio of less than 1.

The accumulated annual net benefits for all projects are composed of benefits from irrigation, drainage, fish life, and flood control. As shown in Table 14, irrigation benefits fluctuate only slightly with changes in channel capacity; however, increased reservoir capacity causes irrigation benefits to increase. A reservoir capacity of 67,000 acre-feet provides between 92 and 97 percent of the maximum annual irrigation benefits possible after the 20-year growth period following completion of the project. A reservoir of 97,000 acre-feet capacity provides close to 100 percent.

Fish life enhancement follows the same pattern as irrigation. The variance is small among the channel sizes with the same reservoir capacity. But, between reservoir capacities great differences exist. Because of the lowering of the reservoir to meet irrigation needs, during the autumn and early winter months, the releases from a small reservoir are not sufficient for fish life.

Benefits from drainage are influenced only by the channel capacity. Full benefits are obtained with a large channel while no benefits exist for the small channel.

Flood control benefits are divided into those attributed to the reservoir and those attributed to the channel. The natural channel size is 5,000 c.f.s.; therefore, flood control benefits from a project with a 5,000 c.f.s. channel size would be the result of the reservoir control.

Table 14. Summary of results, unmodified rule curve

Project No.	Capacity Reservoir	Capacity Channel	AANIB[a]	AAFDB[b]	AAFIB[c]	AADRB[d]	AAFCB[e]
	Acre-ft.	c.f.s.	Thousand dollars				
R97C21	97,000	21,000	49,568	4,031.4	41,070	50,000	8,191.6
R97C11	97,000	11,000	49,568	4,031.4	41,070	15,716	5,038.1
R97C5	97,000	5,000	49,570	4,031.4	42,105	0	0
R67C21	67,000	21,000	47,538	3,986.5	7,542.7	50,000	8,219.5
R67C5	67,000	5,000	47,546	3,986.5	7,606.7	0	0

	Accumulated total[f] annual benefits	Accumulated total annual costs	Benefit-cost ratio	Accumulated net benefits
	Thousand dollars	Thousand dollars		Thousand dollars
R97C21	152,861.0	114,584.0	1.33	38,277.0
R97C11	115,423.5	91,115.0	1.27	24,308.0
R97C5	95,706.4	87,083.0	1.10	8,623.0
R67C21	117,286.7	97,046.0	1.21	20,241.0
R67C11	59,139.2	69,545.0	.85	-10,406.0

[a] Accumulated annual irrigation benefits.

[b] Accumulated annual flood benefits to the reservoir.

[c] Accumulated annual fish benefit.

[d] Accumulated annual drainage benefits.

[e] Accumulated annual flood benefits to the channel.

[f] The total of the individual use benefits may not add to the accumulated total annual benefits due to rounding.

However, with a larger channel and the same reservoir size, all additional flood control benefits which arise are due to the larger channel capacity. Thus, the additional benefits are credited to the channel.[39]

The reservoir flood benefits are nearly the same between the two reservoir sizes and within each reservoir size. The channel flood benefits are also relatively unchanged between the reservoir sizes. However, within each reservoir, the benefits vary with the channel capacity. The channel flood benefits except for those from the natural channel capacity (5,000 c.f.s.) are higher than the reservoir flood benefits. This suggests that channel inprovement for flood control on the Calapooia River is just as important or more so than reservoir construction.

Changes in Rule Curve

As noted in the previous subsection, inadequate water during critical periods of the year limited the fish benefits for the alternative projects. Thus, it was thought _a priori_ that modifying the flood control rule curve to make more water available during these critical periods would increase fish benefits. Modifications of the rule would be possible without additional construction costs. Thus the question was: Will modifying the rule curve increase or decrease total net benefits to the projects?

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[39] See Appendix Table B-4 for the numerical values of flood damages. An interesting point to note is that the largest flood benefits do not always occur with the largest flood. For example, the largest unregulated flood (17.2 ft.) occurs during year 41 and causes $388,900 damage. The flood is partially controlled and flood damage reduced to $89,368 by project R97C21. Thus, the flood control benefits are $299,532. However, the largest annual flood benefits ($312,595) occur in year 80 when the unregulated flow (17.0 ft.) causes $350,070 damage and the regulated flow causes only $25,430 damage.

Three percent increase

The first modification constituted increasing the flood control rule curve by three percent. This means that the rule curve shown in Figure 9 has the same shape but the percentage of capacity full is three percent greater for each day. However, during the summer months of June, July, August, and parts of May and September the curve is unchanged. During these months the rule curve was designed to maintain only a small amount of storage for control of runoff from a late spring or summer storm. The benefits arising due to this modification are shown in Table 15.

As expected fish benefits did increase for all projects. Flood control benefits for all projects remained virtually unchanged. Irrigation benefits in the small reservoir projects also increased noticeably. Thus, benefit-cost ratios were increased for all projects. However, their ranking remained unchanged. It would thus appear that the rule curve modification was an improvement over the operating procedure of the original system.

Six percent increase

Because of the increased benefits arising from the first rule curve modification, a further increase in the rule curve was tested. The rule curve was increased by six percent. Thus, the rule curve shown in Figure 9 is six percent greater than the rule curve for the original model for each day except for the months noted previously. Table 16 summarizes the results of this modification.

Table 15. Summary of results, rule curve modified by three percent

Project No.	Capacity: Reservoir	Capacity: Channel	AANIB[a]	AAFDB[b]	AAFIB[c]	AADRB[d]	AAFCB[e]
	Acre-ft.	c.f.s.	Thousand dollars				
R97C21	97,000	21,000	49,579	4,028.2	52,470	50,000	8,194.8
R97C11	97,000	11,000	49,579	4,028.2	52,470	15,716	5,041.4
R97C5	97,000	5,000	49,581	4,028.2	49,631	0	0
R67C21	67,000	21,000	47,888	3,983.8	9,836.9	50,000	8,222.3
R67C5	67,000	5,000	47,891	3,983.7	9,836.9	0	0

	Accumulated total annual benefits[f]	Accumulated total annual costs	Benefit-cost ratio	Accumulated net benefits
	Thousand dollars	Thousand Dollars		Thousand dollars
R97C21	164,272.0	114,584.0	1.43	49,689
R97C11	126,834.6	91,115.0	1.39	35,720
R97C5	103,240.2	87,083.0	1.19	16,157
R67C21	119,930.9	97,046.0	1.24	22,885
R67C5	61,711.6	69,545.0	.89	-7,833

[a] Accumulated annual irrigation benefits.

[b] Accumulated annual flood benefits to the reservoir.

[c] Accumulated annual fish benefit.

[d] Accumulated annual drainage benefits.

[e] Accumulated annual flood benefits to the channel.

[f] The total of the individual use benefits may not add to the accumulated total annual benefits due to rounding.

Table 16. Summary of results, rule curve modified by six percent

Project No.	Capacity: Reservoir	Capacity: Channel	AANIB[a]	AAFDB[b]	AAFIB[c]	AADRB[d]	AADCB[e]
	Acre-ft.	c.f.s.	Thousand dollars				
R97C21	97,000	21,000	49,588	4,027.6	52,470	50,000	8,195.4
R97C11	97,000	11,000	49,588	4,027.6	52,470	15,716	5,039.8
R97C5	97,000	5,000	49,590	4,027.6	49,631	0	0
R67C21	67,000	21,000	48,078	3,983.7	10,367	50,000	8,222.3
R67C5	67,000	5,000	48,085	3,983.7	10,367	0	0

	Accumulated total[f] annual benefits	Accumulated total annual costs	Benefit-cost ratio	Accumulated net benefits
	Thousand dollars	Thousand dollars		Thousand dollars
R97C21	164,281.0	114,584.0	1.43	49,697
R97C11	126,841.4	91,115.0	1.38	35,727
R97C5	103,248.6	87,083.0	1.16	16,165
R67C21	120,651.0	97,046.0	1.24	23,605
R67C5	62,435.7	69,545.0	.87	-7,109.5

[a] Accumulated annual irrigation benefits.

[b] Accumulated annual flood benefits to the reservoir.

[c] Accumulated annual fish benefit.

[d] Accumulated annual drainage benefits.

[e] Accumulated annual flood benefits to the channel.

[f] The total of the individual use benefits may not add to the accumulated total annual benefits due to rounding.

Accumulated net benefits remained nearly constant when compared to the three precent rule curve modification for projects R97C21, R97C11, and R97C5. However, net benefits did increase in projects R67C21 and R67C5. The increased benefits occurred for the same reason that the benefits increased in the first rule curve modification; i.e., increased irrigation and fish life benefits. Thus, the benefits from this modification of the rule curve is dependent upon the size of structure to be built. It improves the small reservoir benefits regardless of the channel size. The improvement of the large reservoir projects, however, is negligent.

Release modification

One of the problems in making releases from the reservoir is predicting what the uncontrolled inflow will be below the dam in subsequent days. Without taking uncontrolled inflow into account, releases may be made which aggravate the flood situation below the dam. In an attempt to ameliorate the problem, a routine was built into the model which compares desired releases with known inflow on the same day. Thus, should downstream inflow on a given day be greater than the channel capacity, no reservoir releases are made on that day.

An example will help to show the procedure. The actual channel level today is composed of what was in the channel yesterday, what came into the channel yesterday, both from the downstream and the lagged upstream flows, and what went out into the Willamette River at Albany. Or in mathematical terms,

CLVA.K = CLVA.J + (DT) (LROUT.JK + CIN.JK - COUT.JK).

Where

CLVA = actual channel level

ROUT = reservoir release

LROUT = lagged reservoir releases (one day)

CIN = downstream inflow

COUT = flows into the Willamette River

DT = time period, which in our case is one day

J, K, JK = subscripts denoting time. J and JK denote the previous time period and K denotes today.

Case I

Now let us suppose on day 2 CLVA.K equals 3,500 c.f.s. (see Table 17). It is composed of what was in the channel on day 1 (CLVA.J, 4,000 c.f.s.), what came into the channel during day 1 (CIN.JK, 2,500 c.f.s.), what was released from the reservoir on day 0 (LROUT.JK, 1,000 c.f.s.), and what went out into the Willamette River (COUT.JK, 4,000 c.f.s.). The reservoir manager seeing that the channel level is only 3,500 c.f.s. and desiring to lower the level in the reservoir releases 1,000 c.f.s. (It is assumed that the channel capacity is 5,000 c.f.s. and that the manager desires to keep emergency space for 500 c.f.s. in the channel.) On day 2, let us assume, CIN.KL increases to 5,500 c.f.s. This amount of water is 500 c.f.s. more than the 5,000 c.f.s. channel capacity and hence will cause a flood. CIN.KL from day 2 coupled with 800 c.f.s., the reservoir release of day 1, and the 3,500 c.f.s. outflow during day 2 means that CLVA.K on day 3 is 6,300 c.f.s. On day 3 CIN.KL drops to

Table 17. Actual channel level as affected by different reservoir release routines, Calapooia River

Day	CLVA.K	CLVA.J	LROUT.JK	CIN.JK	COUT.JK	ROUT.K
	c.f.s.	c.f.s.	c.f.s.	c.f.s.	c.f.s.	c.f.s.
Case I						
0	*	*	*	*	*	1,000
1	4,000	*	*	*	*	800
2	3,500	4,000	1,000	2,500	4,000	1,000
3	6,300	3,500	800	5,500	3,500	0
4	6,400	6,300	1,000	5,400	6,300	0
Case II						
0	*	*	*	*	*	1,000
1	4,000	*	*	*	*	800
2	3,500	4,000	1,000	2,500	4,000	0
3	6,300	3,500	800	5,500	3,500	0
4	5,400	6,300	0	5,400	6,300	0

* Information unimportant for example development.

5,400 c.f.s. and COUT.KL increases to 6,300 c.f.s. Therefore, on day 4 CLV.K climbs to 6,400 c.f.s. with the addition of the 1,000 c.f.s. which was released back on day 2. This means that a flood of 1,400 c.f.s. occurs on day 4.

Case II

But now suppose that on day 2 the manager had seen that CIN.KL for day 2 was to be 5,500 c.f.s. The manager, knowing that this amount of water would cause a 500 c.f.s. flood and assuming room existed in the reservoir for additional water, would not have released the 1,000 c.f.s. that he did. With all other things remaining the same, on day 4 CLVA.K would have been 5,400 c.f.s. and caused a flood of 400 c.f.s. However, on day 3 CLVA.K was 6,300 c.f.s. It is the highest flow in the series under consideration and is considered the flood for the period. The difference between it and the largest flow in Case I is 100 c.f.s. (6,400 c.f.s. - 6,300 c.f.s.). Thus it would appear that forecasting would improve flood benefits to the reservoir.

In the model forecasting means that the manager knows what the downstream flow will be for the entire day as he makes the daily release. The release decision in the model is made once each day. In the real system the manager would have access to weather forecasts as well as a constant flow of information about downstream conditions and, hence, could modify the quantity of water released several times during the day.[40] In getting

[40] This pinpoints the need for considerable care in the selection of DT. If DT is too short, computations require too much time and money. Also, data needs are more comprehensive. If DT is too long, important changes may occur in the real world which are missed or glossed over in the model. As a rule of thumb, DT should be selected and made just long enough so that no significant change occurs during the time interval.

In getting the model to simulate this aspect of the real system, it was assumed that the manager knew the downstream inflow for the entire day. This was necessary since we had chosen DT to be equal to one day.

As shown in Table 18, flood benefits increase for projects R97C11, R97C5, and R67C5, while all other benefits remain the same (compare Table 14). However, flood benefits do not increase for projects R97C21 and R67C21. Thus, the increase in flood benefits due to the release modification is dependent upon size of channel rather than size of reservoir. On the one hand, larger channels provide for greater certainty of a smaller flood in the event that mistakes were made in reservoir releases. However, on the other hand, with the less costly smaller channels, forecasting might be a cheaper means to increase flood benefits than constructing a larger channel.

Table 18. Summary of results, releases modified by downstream inflows, unmodified rule curve

Project No.	Capacity: Reservoir	Capacity: Channel	AANIB[a]	AAFDB[b]	AAFIB[c]	AADRB[d]	AAFCB[e]
	Acre-ft.	c.f.s.	Thousand dollars				
R97C21	97,000	21,000	49,568	4,031.9	41,070	50,000	8,191.5
R97C11	97,000	11,000	49,568	4,031.9	41,070	15,716	5,067.6
R97C5	97,000	5,000	49,570	4,031.9	42,635	0	0
R67C21	67,000	21,000	47,538	4,016.6	7,542.7	50,000	8,189.6
R67C5	67,000	5,000	47,547	4,016.6	7,606.7	0	0

	Accumulated total annual benefits[f]	Accumulated total annual costs	Benefit-cost ratio	Accumulated net benefits
	Thousand dollars	Thousand dollars		Thousand dollars
R97C21	152,861	114,584.0	1.33	38,277.0
R97C11	115,453.5	91,115.0	1.26	24,338.0
R97C5	96,236.9	87,083.0	1.09	9,154.1
R67C21	117,286.7	97,046.0	1.21	20,241.0
R67C5	59,170.3	69,545.0	.83	-10,375.0

[a] Accumulated annual irrigation benefits.

[b] Accumulated annual flood benefits to the reservoir.

[c] Accumulated annual fish benefit.

[d] Accumulated annual drainage benefits.

[e] Accumulated annual flood benefits to the channel.

[f] The total of the individual use benefits may not add to the accumulated total annual benefits due to rounding.

SUMMARY AND CONCLUSIONS

The general objective of this study was to test the applicability of the technique of computer simulation in the planning, development and evaluation of river basin projects. The public is vitally concerned with the proper management and development of water resources. Over time many government agencies, both federal and state, have been established to develop and manage the nation's water resources. For the most part, the agencies have been single purpose or function oriented; i.e., Corps of Engineers, flood control; Bureau of Reclamation, irrigation, etc. Historically each agency has been concerned with the development of its own function. However, with the passage of Senate Bill 97, comprehensive river basin planning has been emphasized. This has tended to stress agency cooperation in developing general comprehensive plans for entire river basins.

River basins that cover large geographical areas are complex systems. Generally, the water of a river basin system is used for several alternative purposes. These purposes are generally not complementary.

In the situation where insufficient natural flows exist to satisfy beneficial water uses, problems of conflict arise. There are many alternative ways, of course, to supply and control water. But, generally, water can only be supplied and controlled at a cost. Thus, proper planning and evaluation are necessary to decide if the beneficial uses can be met and if they are, which of the many projects or combinations of projects can best supply the users.

Evaluation of alternatives has been difficult in the past because no way other than the time-consuming and expensive budgeting approach was available to test the consequences of alternative projects. Indeed, without the computer, alternatives were generally limited to two or three modifications of one general plan. Truly comprehensive planning was impossible.

With the advent of the computer, previous time-consuming methods have been made obsolete in terms of time needed to perform the calculations. Furthermore, many alternatives can be compared. Also, with the advent of the computer, techniques were developed to make better use of computer capabilities. One of these techniques was simulation; that is, modeling on the computer the real world as it exists. Once the computer model is developed for a river basin system, alternative projects and management policies can be tested and the consequences traced. Physical construction of the contemplated structures or actual implementation of management decisions need not take place in order to find out the consequences over time. The authors believe that the complexities of river basin analysis, with its many possible water users and controlling agencies, lends itself well to the simulation approach.

A secondary objective of this study was to test the applicability of a specific simulation language, DYNAMO, to river basin analysis. Batelle Memorial Institute had previously used the language in an attempt to ascertain the important variables which influence economic growth of a river basin. The approach of this study is oriented to the planning and development of a river basin.

A study can be broken into two major divisions: hydrology and economics. A stochastic approach to hydrology generation was used where the flows on any particular day follow a determined frequency function. Considerable time was spent in developing a hydrologic sequence from the existing historical record and bench-mark floods. The economic section consists of a series of functions and equations which relate fulfillment of different water needs to dollars.

Tentative Conclusions Concerning the Calapooia River Basin Projects[41]

(1) Of the five projects considered, the 97,000 acre-feet reservoir in conjunction with the 21,000 cubic feet per second channel capacity provides the maximum net benefits.

(2) It appears that channel improvement is as important, or more so, than increasing reservoir size. For example, when the channel was increased from 5,000 to 21,000 cubic feet per second capacity, with the largest reservoir, the accumulated net benefits from the uses included in this study increased 344 percent. However, increasing the reservoir size from 67,000 to 97,000 acre-feet capacity with the largest channel (21,000 c.f.s.) only increased accumulated net benefits 89 percent.

(3) Through proper management it was shown that greater net benefits can be obtained. In fact, some modification in management policies can substitute for expensive construction of larger structures. For example, increasing the flood control rule by 3 percent, increased the net benefits

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[41] It should be recognized that these conclusions are based upon ballpark estimates of benefits and costs.

from the 97,000 acre-feet reservoir and 11,000 c.f.s. channel capacity to within 93 percent of the net benefits received from the 97,000 acre-feet reservoir and 21,000 c.f.s. channel capacity using the unmodified rule.

(4) Forecasting of downstream hydrologic flows increases the accumulated net benefits to some of the projects. In fact, the additional net benefits provides an estimate of the value of forecasting information. For example, in the case of the 97,000 acre-feet reservoir with the 5,000 c.f.s. channel capacity, the value of forecasting information could be estimated to be in excess of half a million dollars over the 100-year life of the project.

Conclusions Concerning Simulation

(1) Simulation as an approach to river basin planning and development appears to be promising. It appears to be the only way truly comprehensive planning can occur, if in fact comprehensive planning can be done. Simulation provides a visible integration of the hydrology and the technology with the economics of a planning problem and illustrates how all three aspects affect the operation of a system and *vice versa*.

(2) Simulation is a practical approach to the piecemeal planning philosophy wherein trial and error leads to improvement in the system's operation.

(3) Simulation appears to be a method of encouraging the assembling of all relevant information and data that may impinge upon the development of a comprehensive plan.

(4) Simulation encourages the planner to be explicit about his assumptions.

(5) DYNAMO simulation language appears to be a promising computer programming language for river basin studies. A number of modifications would improve its usefulness in this type of study. These modifications are: (a) variable time unit (DT), (b) greater capacity to use data exogenous to the model, (c) a more flexible print capability, (d) possibility of processing on other than IBM 7094 computers.

APPENDIX A

CALAPOOIA RIVER BASIN DYNAMO MODEL

Identification of Variables

This section is concerned with identifying the variables in the model and providing a listing of the computer program. The variables will be identified as they appear in the model. As part of the identification, the units of each variable will be specified as follows:

% = percentage

$ = dollars

a.f. = acre-feet

c.f.d. - cubic feet per day

c.f. = cubic feet

c.f.s. = cubic feet per second

AV = auxiliary variable

<u>DAYS COUNTER</u>--This subsection specifies the day of the year.

DAYS = Day number

DAYIN = First day of the year

DAYC = 1

DAYOT = 360 on 360th day, sets day counter back to zero at the beginning of each year. (AV)

<u>YEARS COUNTER</u>--This subsection specifies the year.

YEARS = Year number

YRSIN = Increases year number by one every 360 days. (AV)

<u>RESERVOIR AND CHANNEL LEVEL</u>

RLVA = Actual reservoir level (a.f.)

CLVA = Actual channel level (c.f.d.)

EVAPO = Evaporation rate per day (c.f.d.)

EVAPT = Table specifying monthly evaporation rate (c.f.)

<u>RESERVOIR RELEASES</u>--This subsection specifies releases as dictated by the rule curve and the water need for fish life.

LROUT = ROUT lagged one day (c.f.d.)

ROUT = Reservoir releases as dictated by the rule curve and the water need for fish life (c.f.d.).

ROUT1: Verifies that the spillway capacity is large enough to handle desired release (c.f.d.)(AV).

ROUT2: Verifies that the channel capacity is large enough to handle desired release (c.f.d.)(AV).

RWOPC: Is either the release specified by the rule curve or the maximum amount of water that can be released into the channel, depending on whether the release capacity is greater than the specified release (c.f.d.).

CCPLA = Channel capacity minus the safety factor minus the actual channel level (c.f.d.).

SPICP = Spillway capacity (c.f.d.).

DCHLV = Channel capacity minus safety factor (c.f.d.).

RWOPP = Desired reservoir level (%).

RWOPL = Desired reservoir level (a.f.).

RWOPA = Actual reservoir level minus desired reservoir level (c.f.).

RWOP = Desired reservoir release (c.f.).

ROUT3: Maximum between the desired release for fish and desired releases as specified by RWOPC (AV).

RLVA1 = Actual reservoir level (c.f.s.).

RLVA2 = Actual reservoir level minus desired release for fish minus fish safety factor (c.f.s.).

FSF = Fish safety factor (c.f.s.).

MRLV2 = Minimum reservoir level during the year (c.f.s.)

RMFF1 = Desired release for fish plus the fish safety factor (c.f.s.)

MINX: If RLVA2 is greater than zero, it is RMFF1;

if RLVA2 is less than zero, it is RLVA1(AV).

RMFF = Table showing daily desired release for fish.

UPSTREAM HYDROLOGY--In this subsection the upstream hydrology is generated.

RIN = Inflow into the reservoir (c.f.d.)

DIST1 = Normal distribution

MEAN1 = Mean of normal distribution (c.f.s.)

STD1 = Standard deviation for normal distribution (c.f.s.)

RINU: Makes any negative number from the uniform distribution (UDT1), positive (c.f.s.)(AV).

UDT1 = Uniform distribution number one

RAN1 = Uniform distribution parameter (c.f.s.)

ERIN1: If inflow is in critical zone, it becomes a number from a second uniform distribution (c.f.s.)(AV).

STD24 = Lower limit of critical zone (c.f.s.)

DIST2 = Normal distribution number 2.

STD2 = Parameter for second uniform distribution (c.f.s.)

ERIN3: Smooths out the descent of the hydrograph (c.f.s.)(AV).

ERIN2: Makes certain the simulated hydrologic flow never goes below the historical minimum daily flow (c.f.s.)(AV).

MRMF = Table showing the observed minimum daily flow (c.f.s.)

N6ER1 = Negative first difference equation

REDF = Negative first difference parameter

DOWNSTREAM HYDROLOGY--In this subsection the downstream hydrology is developed.

CIN = Channel inflow (c.f.d.)

ECIN2: Makes certain the downstream hydrologic flow never goes below the observed minimum daily flow (c.f.s.)(AV)

ECIN3 = Downstream hydrologic generating equation (c.f.s.)

CEDF1 = Equation parameter for ECIN3

STAE = Equation standard error for ECIN3

CEC1 = Equation constant for ECIN3

DMF = Downstream minimum flow (c.f.s.)

FLOWS INTO WILLAMETTE RIVER--The flows into the Willamette River are calculated in this subsection.

COUTS = Flow into Willamette River (c.f.s.)

COUT = Flow into Willamette River (c.f.d.)

IRRIGATION RELEASES--Releases made for irrigation are calculated in this subsection.

IRRIN = Irrigation return flow (c.f.d.)

PERRF = Percentage return flow (%)

IROUT = Irrigation releases (c.f.d.)

IROT: Releases for irrigation, either the water available in the reservoir or the desired irrigation release. (c.f.d.)(AV).

IRRN1 = Irrigation monthly need times irrigation growth factor (a.f.)

IRRN2 = Actual reservoir level times irrigation growth factor (a.f.)

IRRIGATION NEED--The need for the irrigation acreage is specified in this subsection.

IRRG = Irrigation growth factor

IRRNA = Irrigation monthly need (a.f.)

IRRNT = Irrigation total need (a.f.)

IRRIGATION BENEFIT CALCULATION--In this subsection the calculation of the benefits from irrigation occurs.

ANIB = Maximum irrigation benefit if entire need is met ($)

ANIBH = Actual annual irrigation benefit ($)

PERTM = Percentage of target irrigation need met (%)

TIROT = Accumulated annual releases for irrigation (c.f.d.)

FLOOD BENEFIT CALCULATION--Benefits from flood control are calculated in this subsection.

AFDB = Annual flood benefits ($)

AAFDB = Accumulated flood benefits ($)

DRAINAGE BENEFIT CALCULATION--Benefits arising from drainage enhancement are calculated in this subsection.

ATDRB = Table relating maximum possible drainage benefits to channel capacity ($)

PRCLV = Average percentage full of the channel during drainage period (%)

PRDTM = Table relating average percentage full of channel to percentage of maximum possible drainage benefits.

ACLVA = Average channel level during drainage period (c.f.d.)

DDR = Drainage period

ANDBR = Annual drainage benefits ($)

AADRB = Accumulation annual drainage benefits ($)

<u>FISH BENEFIT CALCULATION</u>--In this subsection the benefits arising as a result of improvement in fish habitat are included.

MICLS = Minimum PMICL (c.f.s.)

PMICL = Ratio of the actual channel level to the fish need (c.f.s.)

FIB = Relates the ratio of need met to maximum possible fish benefits

ACFIB = Annual fish benefits ($)

AFIB = Accumulated annual net fish benefits ($)

MINC = Table specifying fish requirements (c.f.s.)

<u>COSTS</u>--Operation, maintenance and repair, and amortized capital costs for the alternative projects are calculated in this subsection

RIRC = Table which relates irrigation costs to reservoir capacity ($)

RDAMC = Table which relates reservoir costs to reservoir capacity ($)

RTOCC = Annual reservoir costs ($)

RTOC = Accumulated annual reservoir costs ($)

CDRCC = Table relating drainage costs to channel capacity ($)

CDR = Annual channel costs ($)

CDRC = Accumulated annual channel costs ($)

AFC = Annual fish cost ($)

AAFCC = Accumulated annual fish costs ($)

<u>NET BENEFITS</u>--In this subsection the net benefits for the entire system are computed.

ARIFB = Accumulated reservoir benefits ($)

DRBC = Accumulated reservoir net benefits ($)

DCBC = Accumulated channel net benefits ($)

TNB = Accumulated total net benefits ($)

<u>STRUCTURE SIZES</u>--Specification of structure sizes for each proposed project occurs in this subsection

CCAPD = Channel capacity (c.f.d.)

DHCAP = Channel capacity (c.f.s.)

RECAP = Reservoir capacity (a.f.)

Table A-1. DYNAMO equations of Calapooia River Basin Model

```
2061-2,DYN,Test, 10,15,0,0     Approx. 50 pages

RUN     CM1965
NOTE
NOTE    Calapooia River Model   Continuous Hydrology
NOTE
NOTE    DAYS COUNTER
NOTE
1L      DAYS.K=DAYS.J+(DT)(DAYIN.JK-DAYOT.JK)                              DC1
6R      DAYIN.KL=DAYC                                                      DC2
C       DAYC=1                                                             DC3
41R     DAYOT.KL=PULSE(360,360,360)                                        DC4
NOTE
NOTE    YEARS COUNTER
NOTE
1L      YEARS.K=YEARS.J+(DT)(YRSIN.JK+O)                                   YC1
41R     YRSIN.KL=PULSE(1,360,360)                                          YC2
NOTE
NOTE    RESERVOIR AND CHANNEL LEVEL
NOTE
4L      RLVA.K=RLVA.J+(DT)(1/43560)(RIN.JK-ROUT.JK-IROUT.JK-EVAPO.JK+0+0)  RCL1
52L     CLVA.K=CLVA.J+(DT)(LROUT.JK+CIN.JK+IRRIN.JK-COUT.JK)               RCL2
20R     EVAPO.KL=EVAPT*12.K/30                                             RCL3
35B     EVAPT=BOXCYC(12,30)                                                RCL4
C       EVAPT*=13068/26136/34848/30492/26136/13068/0/0/0/0/0/0             RCL5
NOTE
NOTE    RESERVOIR RELEASES
NOTE
6R      LROUT.KL=ROUT.JK                                                   RR1
51A     ROUT1.K=CLIP(SPICP.K,RWOPC.K,RWOPC.K,SPICP.K)                      RR2
51A     ROUT2.K=CLIP(O,ROUT3.K,CLVAS.K,CCAP.K)                             RR3
51R     ROUT.KL=CLIP(RIN.K,ROUT2.K,RLVA.K,RCAP.K)                          RR4
51A     RWOP.K=CLIP(RWOP.K,CDLC.K,CDLC.K,RWOP.K)                           RR5
51A     CDLC.K=CLIP(CCPLA.K,O,DCHLV.K,CLVA.K)                              RR6
7A      CCPLA.K=DCHLV.K-CLVA.K                                             RR7
6A      SAFNO.K=SAFNU                   SAFETY FACTOR                      RR8
C       SAFNU=1728E+05                                                     RR9
6A      SPICP.K=SPICA                   SPILLWAY CAPACITY                  RR10
C       SPICA=2592E+05                                                     RR11
7A      DCHLV.K=CCAPD.K-SAFNO.K                                            RR12
58A     RWOPP.K=TABHL(WOPT,DAYS.K,1,361,15)                                RR13
C       WOPT*=.178/.01/.0/.0/.0/.0/.0/.183/.367/.478/.589/.696/.800.       RR14
X1      911/.911/.911/.911/.911/.911/.911/.911/.738/.550/.367/.178         RR14
12A     RWOPL.K=(RCAP.K)(RWOPP.K)                                          RR15
15A     RWOPA.K=(RLVA.K)(43560)+(-RWOPL.K)(43560)                          RR16
51A     RWOP.K=CLIP(RWOPA.K,O,RLVA.K,RWOPL.K)                              RR17
56A     ROUT3.K=MAX(ROUT1.K,MINXX.K)                                       RR18
```

```
51A     MINX.K=CLIP(RMFF1.K,RLVA1.K,RLVA2.K,0)                                RR19
44A     RLVA1.K=(RLVA.K)(43560)/86400                                          RR20
8A      RLVA2.K-RLVA1.K-RMFF.K-FSF*24.K                                        RR21
35B     FSF=BOXCYC(24,15)                                                      RR22
C       FSF*=0/0/16/8/0/0/0/0/0/0/0/0/5/0/4/0/35/0/50/0/0/0/0                  RR23
54A     MRLV2.K=MIN(MMRLV.JK,RLVA2.K)                                          RR24
51R     MMRLV.KL-CLIP(99999,MRLV2.K,DASY.K,360)                                RR25
7A      RMFF1.K=RMFF.K+FSF*24.K                                                RR26
12A     MINXX.K=(MINX.K)(86400)                                                RR27
58A     RMFF.K=TABHL(RMFT,DAYS.K,1,361,15)                                     RR28
C       RMFT*=140/140/140/140/85/85/50/50/80/80/115/115/140/140/
             90/70/50/5                                                        RR29
X1      0/50/50/50/160/160/160/140                                             RR49


NOTE
NOTE    UPSTREAM HYDROLOGY
NOTE
12R     RIN.KL=(ERIN2.K)(86400)                                                UH1
51A     RIND.K=CLIP(DIST1.K,RINU.K,DIST1.K,0)                                  UH2
43A     DIST1.K=SAMPLE(NDST1.K,1)                                              UH3
34A     NDST1.K=(1)NORMRN(MEAN1*12.K,STD1*12.K)                                UH4
35B     MEAN1=BOXCYC(12,30)                                                    UH5
35B     STD1=BOXCYC(12,30)                                                     UH6
C       MEAN1*=34/38/69/165/300/500/585/700/570/600/317/80                     UH7
C       STD1*=39/15/41/172/272/406/532/756/930/1085/798/387                    UH8
51A     RINU.K=CLIP(UDT1.K,UDTP1.K,UDT1.K,0)                                   UH9
6A      UDTP1.K=-UDT1.K                                                        UH10
43A     UDT1.K=SAMPLE(UDTN1.K,1)                                               UH11
33A     UDTN1.K=(RAN1*12.K)NOISE                                               UH12
35B     RAN1=BOXCYC(12,30)                                                     UH13
C       RAN1*=68/76/138/330/600/1000/1170/1400/1140/1200/634/160               UH14
51A     ERIN1.K=CLIP(MDST1.K,RIND.K,STD24*12.K)                                UH15
35B     STD24=BOXCYC(12,30)                                                    UH16
C       STD24*=88/59/126/405/679/1066/1327/2370/2625/2997/1430/863             UH17
6A      MDST1.K=IDST1.K                                                        UH18
51A     IDST1.K=CLIP(DIST2.K,PDST1.K,DIST2.K,0)                                UH19
6A      PDST1.K=-DIST2.K                                                       UH20
43A     DIST2.K=SAMPLE(NDST2.K,1)                                              UH21
33A     NDST2.K=(STD2*12.K)NOISE                                               UH22
35B     STD2=BOXCYC(12,30)                                                     UH23
        STD2*=754/848/240/2180/6520/5100/6540/26000/26000/26000/5440/10560     UH24
X1                                                                             UH24
51A     ERIN3.K=CLIP(N6ER1.K,ERIN1.K,N6ER1.K,RMF*12.K)                         UH25
51A     ERIN2.K=CLIP(ERIN3.K,MRMF*12.K,ERIN3.K,MRMF*12.K)                      UH26
6A      ERIN4.K=ERIN5.JK                                                       UH27
6R      ERIN5.KL=ERIN2.K                                                       UH28
35B     RMF=BOXCYC(12,30)                                                      UH29
C       RMF*=30/20/77/203/160/468/586/720/680/570/334/174                      UH30
35B     MRMF=BOXCYC(12,30)                                                     UH31
C       MRMF*=20/0/33/56/86/112/112/154/160/35/22/20                           UH32
7A      N6ER1.K=ERIN4.K-REDF.K                                                 UH33
12A     REDF.K=(REDF1*12.K)(ERIN4.K)                                           UH34
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35B     REDF1=BOXCYC(12,30)                                          UH35
C       REDF1*=1/1/1/1/.44/.31/.23/.47/.44/.41/.37/.43               UH36

NOTE
NOTE    DOWNSTREAM HYDROLOGY
NOTE
12R     CIN.KL=ECIN2.K)(86400)                                       DH1
51A     ECIN2.K=CLIP(ECIN3.K,DMF*12.K,ECIN3.K,DMF*12.K)              DH2
8A      ECIN3.K=STAE.K+CEC1*12.K+CEDF.K                              DH3
12A     CEDF.K=(CEDF1*12.K)(ERIN2.K)                                 DH4
43A     STAE1.K=SAMPLE(STAD.K,1)                                     DH5
51A     STAE.K=CLIP(STAE2.K,STAE1.K,ERIN2.K,5300)                    DH6
43A     STAE3.K=SAMPLE(STAED.K,1)                                    DH7
33A     STAED.K=(23852)NOISE                                         DH8
51A     STAE2.K=CLIP(STAE3.K,STAE4.K,STAE3.K,0)                      DH9
6A      STAE4.K=-STAE3.K                                             DH11
35B     STSD=BOXCYC(12,30)                                           DH12
34A     STAD.K=(1)NORMRN(0,STSD*12.K)                                DH13
C       STSD*=10/4/6/10/92/124/233/361/501/493/306/191               DH14
35B     CEC1=BOXCYC(12,30)                                           DH15
C       CEC1*=1/-10/0/4/-34/-213/-131/141/143/245/-29/-53            DH16
35B     CEDF1=BOXCYC(12,30)                                          DH17
C       CEDF1*=.02/.20/.13/.54/.52/.97/1.23/1.11/1.39/.92/.90/.69    DH18
35B     DMF=BOXCYC(12,30)                                            DH19
C       DMF*=0/0/0/0/0/25/60/90/55/0/0/0                             DH20
NOTE
NOTE    FLOWS INTO WILLAMETTE RIVER
NOTE
51A     COUTS.K=CLVAS.K                                              FW1
12R     COUT.KL=(COUTS.K)(86400)                                     FW2
NOTE
NOTE    IRRIGATION RELEASES
NOTE
12R     IRRIN.KL=(PERRF((IROUT.JK)         IRR RETURN TO CHANNEL     IR1
C       PERRF=.15                                                    IR2
51R     IROUT.KL=CLIP(IRRND.K,IROT.K,RLVA.K,IRRNA.K)                 IR3
51A     IROT.K=CLIP(IRRA.K,IRRND.K,IRRND.K,IRRA.K)                   IR4
13A     IRRN1.K=(IRNM*12.K)(IRRNT)(IRRG.K)                           IR5
13A     IRRN2.K=(IRNM*12.K)(RLVA.K)(IRRG.K)                          IR6
44A     IRRND.K=(IRRN1.K)(43560)/30                                  IR7
44A     IRRA.K=(IRRN2.K)(43560)/NDYCT*30.K                           IR8
```

```
NOTE
NOTE     IRRIGATION NEED
NOTE
58A      IRRG.K=TABHL(IRG,YEARS.K,1,40,20)  IRRIGATION GROWTH                 IN1
C        IRG*=.05/1.0/1.0                                                     IN2
12A      IRRNA.K=(IRRNB.K)(IRRG.K)                                            IN3
58A      IRRNB.K=TABHL(IRNT, DAYS.K,151,331,30)                               IN4
35B      IRNM=BOXCYC(12,30)                                                   IN5
C        IRNM*=.03/.305/.355/.20/.077/.03/0/0/0/0/0/0                         IN6
35B      NDYCT=BOXCYC(30,1)                                                   IN7
C        NDYCT*=2/3/4/5/6/7/8/9/10/11/12/13/14/15/16/17/18/19/20/21/22/23/2   IN8
X1       4/25/26/27/28/29/30/1                                                IN8
C        IRNT*=69900/67800/62400/48400/23600/2300/0                           IN9
C        IRRNT=69900                           IRRIGATION NEED A F            IN10
NOTE
NOTE     IRRIGATION BENEFIT CALCULATION
NOTE
C        ANIB=552690                           ANNUAL IRR BENEFIT             IBC1
12A      ANIBH.K=(PERTM.K)(ANIB)                                              IBC2
20A      PERTM.K=TIRO.K/IRRNT                  PERCENT IRR NEED MET           IBC3
1L       TIROT.K=TIROT.J+(DT)(IROUT.JK-ACIRO.JK)                              IBC4
51R      ACIRO.KL=CLIP(TIROT.K,0,DAYS.K,360)                                  IBC5
20A      TIRO.K=TIROT.K/43560                                                 IBC6
51R      ANIRB.KL=CLIP(ANIBH.K,0,DAYS.K,360)     ANN IRR BENEFIT OBTAIN       IBC7
1L       AANIB.K=AANIB.J+(DT)(ANIRB.JK+0)     ACC AN IRR BENEFITS             IBC8
NOTE
NOTE     FLOOD LOSS
NOTE
7A       TRCIN.K=ERIN2.K+ECIN2.K                                              FL1
58A      TRCIS.K=TABHL(CODIT,TRCIN.K,0,45000,5000)                            FL2
C        CODIT*=19/5700/11400/17100/22800/28500/34200/39900/45600/51300       FL3
         MTIN.K=MAX(Min.JK,TRCIS.K)                                           FL4
51R      MIN.KL=CLIP(0,MTIN.K,DAYS.K,360)                                     FL5
56A      MRIN.K=MAX(UMIN.JK,ERIN2.K)                                          F16
51R      UMIN.KL=CLIP(0,MRIN.K,DAYS.K,360)                                    FL7
58A      FLDLP.K=TABHL(FLDLT,FLDSH.K,10,20,2)     FLOOD LOSS POTENTIAL        FL8
C        FLDLT*=0/2200/16000/135000/550000/1000000                            FL9
51A      FDLPR.K=CLIP(FLDLP.K,0,DAYS.K,360)  FLOOD LOSS POTEN (ANN)           FL10
58A      FLDSH.K=TABHL(FLDST,MTIN.K,0,90000,10000)  FLOOD STAGE AT SHEDD      FL11
C        FLDST*=10/15.75/16.6/16.9/17.15/17.3/17.5/17.65/17.82/18             FL12
20A      CLVAS.K=CLVA.K/86400                                                 FL13
58A      CLVAI.K=TABHL(CODIT,CLVAS.K,0,45000,5000)                            FL14
56A      MCLVA.K=MAX(MCLV.JK,CLVAI.K)                                         FL15
51R      MCLV.KL=CLIP(0,MCLVA.K,DAYS.K,360)                                   FL16
C        FDST1*=10/11/14/15.1/15.75/16.15/16.35/16.5/16.6/16.7                FL17
58A      AFLDS.K=TABHL(FDST1,MCLVA.K,0,90000,10000)                           FL18
58A      FDLAR.K=TABHL(FLDLT,AFLDS.K,10,20,2)   FLOOD LOSS ACTUAL ANN         FL19
51A      FDLR.K=CLIP(FDLAR.K,0,DAYS.K,360)                                    FL20
NOTE
NOTE     FLOOD BENEFIT CALCULATION
NOTE
7A       AFDB.K=FDLPR.K-FDLR.L                FLOOD BENEFITS                  FBC1
1L       AAFDB.K=AAFDB.J+(DT)(AFDB.JK+0)   ACC ANN FLD BEN TO RES             FBC2
NOTE
```

```
NOTE   DRAINAGE BENEFIT CALCULATION
NOTE
58A    ATDRB.K=TABHL(ATDBT,CCAP.K,5000,25000,5000)  DRAINAGE BENEFITS      DB1
C      ATDBT*=0/71450/500000/500000/500000                                 DB2
42A    PRCLV.K=ACLVA.K/((CCAP.K)(86400))    PROP CHANNEL FULL              DB3
58A    PRDTIM.K=RABHL(PDTMT,PRCLV.K,0,L,.L)  PROP DRAIN BEN OBTAIN         DB4
C      PDTMT*=1/1/1/1/.8/.6/.4/.3/.2/.1/0                                  DB5
3L     ACLVA.K=ACLVA.J+(DT)(1/DDR.J)(RCLVA.JK-CLVAO.JK)  AVE CHAN LEV      DB6
51R    RCLVA.KL=CLIP(CLVAA.K,0,DAYS.K,121)                                 DB7
51A    CLVAA.K=CLIP(0,CLVA.K,DAYS.K,241)                                   DB8
51R    CLVAO.KL=CLIP(ACLVA.K,0,DAYS.K,360)                                 DB9
51A    DDR.K=CLIP(1,120,DAYS.K,241)  DRAIN PERIOD DAYS                     DB10
12A    ANDRB.K=(PRDTM.K)(ATDRB.K)                                          DB11
51R    ANDBR.KL=CLIP(ANDRB.K,0,DAYS.K,360)                                 DB12
1L     AADRB.K=AADRB.J+(DT)(ANDBR.JK+0)    ANN DRAIN BEN OBT               DB13
NOTE
NOTE   FISH BENEFIT CALCULATION
NOTE
54A    MICLS.K=MIN(MICL1.JK,PMICL.K)                                       F1
51R    MICL1.KL=CLIP(20,MICLS.K,DAYS.K,360)                                F2
20A    PMICL.K=CLVAS.K/MINC.K                                              F3
51A    MIPCF.K=CLIP(MICLS.K,0,DAYS.K,360)                                  F4
51R    FIB1.KL=CLIP(530000,FIB.K,MIPCF.K,1)                                F5
51A    FIB.K=CLIP(FIB2.K,0,MIPCF.K,.4)                                     F6
12A    FIB2.K=(MIPCF.K)(530000)                                            F7
7A     AFIB.K=ACFIB.K-AAFCC.K   AN. FISH BEN.                              F8
1L     ACFIB.K=ACFIB.J+(DT)(FIB1.JK+0)                                     F9
58A    MINC.K=TABHL(MINCT,DAYS.K,1,361,15)                                 F10
C      MINCT*=140/140/140/140/140/140/140/140/140/140/140/140/140/140/90/  F11
X1     70/50/50/50/50/50/160/160/160/140                                   F11
NOTE
NOTE   COSTS
NOTE
58A    RIRC.K=TABHL(RIRCT,RCAP.K,60000,90000,15000)  IRR COSTS             CC1
C      RIRCT*=31682/31682/31682                                            CC2
       RDAMC.K=TABHL(RDMCT,RCAP.K,60000,90000,15000)   RES COST            CC3
C      RDMCT*=554972/648276/730345                                         CC4
7A     RTOCC.K=RDAMC.K+RIRC.K   ANN COST RESER                             CC5
1L     RTOC.K=RTOC.J+(DT)(RTO.JK+O)                                        CC6
51R    RTO.KL=CLIP(RTOCC.K,0,DAYS.K,360)                                   CC7
59A    CDRCC.K=TABLE(CDRCT,CCAP.K,5000,25000,5000)  DRAIN COSTS            CC8
C      CDRCT*=3480/18000/147000/278490/278490                              CC9
1L     CDRD.K=CDRC.J+(DT)(CDR.JK+O)                                        CC10
51R    CDR.KL=CLIP(CDRCC.K,0,DAYS.K,360)                                   CC11
C      AFC=105317       FISH COST                                          CC12
51R    AFCC.KL=CLIP(AFC,0,DAYS.K,360)                                      CC13
1L     AAFCC.K=AAFCC.J+(DT)(AFCC.JK+0)                                     CC14
NOTE
NOTE   NET BENEFITS
NOTE
7A     ARIFB.K=AAFDB.K+AANIB.K   ACC AN IRR FLD   BRN TO RES               NB1
7A     DRBC.K=ARIFB.K-RTOC.K   ACC AN BEN-  AN COST RESER                  NB2
7A     DCBC.K=AADRB.K-CDRC.K   ACC AN BEN-AN COST CHANNEL                  NB3
```

```
8A      TNB.K=DRBC.K+DCBC.K+AFIB.K TOTAL AN NET BENEFIT                  NB4
NOTE
NOTE    STRUCTURE SIZES
NOTE
6A      CCAP.K=CHCAP                   CAP OF CHAN CFS                   SS1
12A     CCAPD.K=(CCAP.K)(86400)                                          SS2
C       CHCAP=21000                                                      SS3
6A      RCAP.K=RECAP                   CAP OF RES AF                     SS4
C       RECAP=90000                                                      SS5

        INITIAL CONDITIONS
6N      MCLV=1
6N      DAYS=0
6N      RLVA=0
6N      CLVA=0
6N      TIROT=0
6N      AANIB=0
6N      AAFDB=0
6N      YEARS=1
6N      ACLVA=0
6N      AADRB=0
6N      MIN=1
6N      CDRC=0
6N      ERIN2=1
6N      RTOC=0
6N      UMIN=1
6N      ECIN2=1
6N      ACFIB=0
6N      AAFCC=0
6N      ERIN5=1
6N      MICL1=20
6N      LROUT=0
6N      ROUT1=0
6N      MMRLV=99999
PRINT   1)DAYS,YEARS/2)PERTM,AANIB/3)MTIN,MCLVA/4)FDLPR,FDLAR/5)FLDSH,AFLD
        S/6)AFDB/7)ARIFB/8)DRBC/9)PRCLV,ANDRB/10)AADRB,AFIB/11)DCBC,MRLV2/
        12)MIPCF,TNB/13)AAFDB,AAFCC/14)CDRC,RTOC,ACFIB
SPEC    DT=1/LENGTH=36360/PRTPER=360/PLTPER=0
```

```
NOTE
RUN       CM 1965
NOTE      CALAPOOIA MODEL, RESERVOIR 90000, CHANNEL 11000
C         CHCAP=11000
C         FDST1* = 10/14/15.75/16.35/16.6/16.75/16.9/17.05/17.15/17.25
NOTE
RUN       CM 1965
NOTE      CALAPOOIA MODEL, RESERVOIR 90000, CHANNEL 5000
C         CHCAP = 5000
C         FDST1* = 10/15.75/16.6/16.9/17.15/17.3/17.5/17.65/17.82/18
NOTE
RUN       CM 1965
NOTE      CALAPOOIA MODEL, RESERVOIR 600000, CHANNEL 21000
C         CHCAP = 21000
C         RECAP = 60000
NOTE
RUN       CM 1965
NOTE      CALAPOOIA MODEL, RESERVOIR 60000, CHANNEL 5000
C         RECAP = 60000
C         CHCAP = 5000
C         FDST1* = 10/15.75/16.6/16.9/17.15/17.3/17.5/17.65/17.82/18
```

The release modification constituted changing 1 card and adding 1 card. ROUT became

```
51R       ROUT.KL=CLIP(RIN.K,ROUT4.K,RLVA.K,RCAP.K).                RR4
51A       ROUT4.K=CLIP(0,ROUT2.K,ECIN2.K,CCAP.K)   was the equation added.
```

Reservoir Releases: An Example of the Model Logic

In this section, part of the logic of the computer model will be described. To help in the explanation a block diagram of reservoir release for flood control and fish life is given in Figure A-2. Figure A-1 provides the conventions used in the block diagram. The reader will find it helpful to follow the flows in Figure A-2 as he reads the following explanation.

ROUT (Reservoir OUTflow) is the end result of the reservoir releases. Its final use is in determining the level in the channel, but before doing so it is lagged 1 day (DT) becoming LROUT (Lagged Reservoir OUTflow). ROUT is determined from the following equation:

ROUT=RIN if RLVA $\geq$ to RCAP

ROUT=ROUT2 if RLVA < RCAP

Where:

RIN=Reservoir INflow in cubic feet (c.f.)

ROUT2=Auxiliary No. 2 ROUT equation (c.f.)

RLVA=Reservoir LeVel Actual in acre-feet (a.f.)

RCAP=Reservoir CAPacity (a.f.)

The release is therefore dependent upon whether the actual reservoir level is less than, equal to, or greater than the reservoir capacity. But the release is also dependent upon whether the channel level is greater than, equal to, or less than the channel capacity. Therefore,

Figure A-1. Block diagram conventions

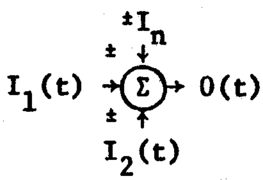
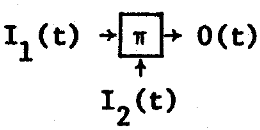
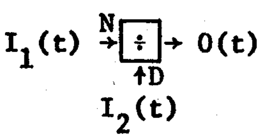
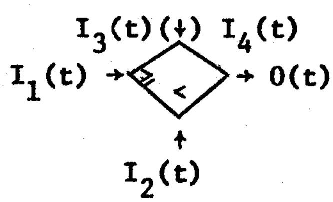
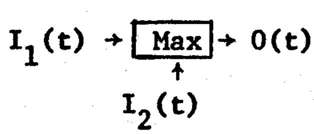
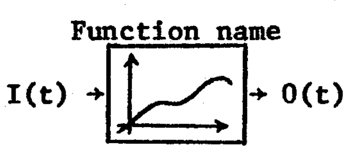
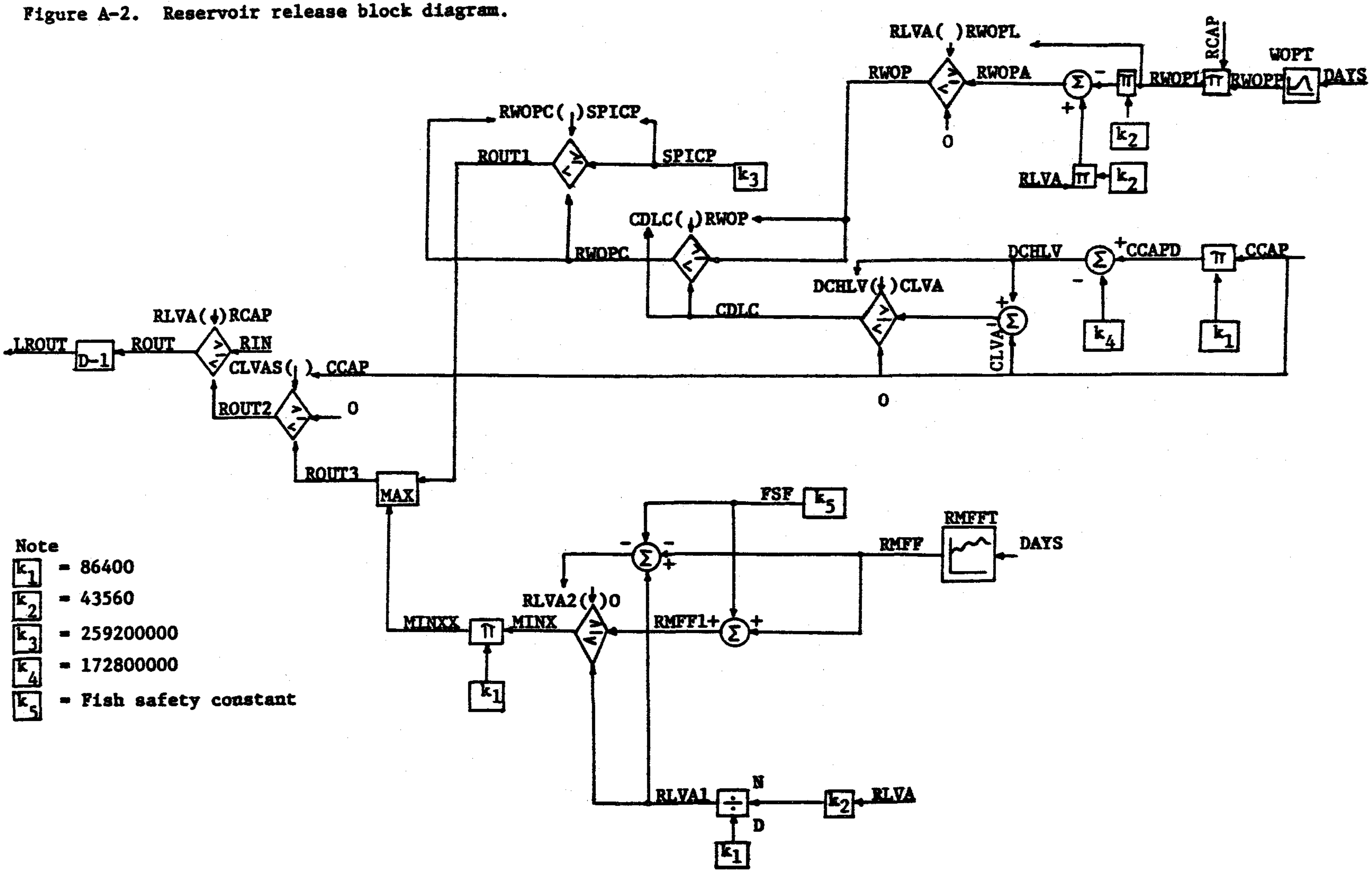
	Symbol	Definition
1.		$O(t) = \pm I_1(t) \pm I_2(t) \pm \ldots \pm I_n(t)$
2.		$O(t) = I_1(t) \times I_2(t)$
3.		$O(t) = I_1(t)/I_2(t)$ (N denotes numerator and D the denominator
4.		$O(t) = K \times I(t)$ (K a constant)
5.		$O(t) = I_1(t)$ if $I_3(t) \geq I_4(t)$ $O(t) = I_2(t)$ if $I_3(t) < I_4(t)$
6.		$O(t)$ = Maximum of $I_1(t)$, $I_2(t)$
7.		$O(t)$ is the named function of $I(t)$

Figure A-2. Reservoir release block diagram.



Note

$\boxed{k_1}$ = 86400

$\boxed{k_2}$ = 43560

$\boxed{k_3}$ = 259200000

$\boxed{k_4}$ = 172800000

$\boxed{k_5}$ = Fish safety constant

ROUT2 is:

ROUT2=0 if CLVAS - CCAP

ROUT2=ROUT3 if CLVAS   CCAP

Where:

CLVAS=Channel LeVel Actual in cubic feet per Second (c.f.)

CCAP=Channel CAPacity (c.f.s.)

ROUT3=Auxiliary No. 3 ROUT equation (c.f.s.)

ROUT3 is the maximum between the releases to adhere to the rule curve and releases for fish life.

ROUT3=MAX (ROUT1, MINXX)

Where:

ROUT1=Auxiliary No. 1 ROUT equation (c.f.), A variable that is concerned with the rule curve.

MINXX=MINimum fish need (c.f.)

The discussion will now follow ROUT1 and then return to MINXX.

ROUT1 is determined by whether the amount of water desired to be released in the channel is greater than, equal to, or less than the spillway capacity.

ROUT1=SPICP if RWOPC is $\geq$ SPICP

ROUT1=RWOPC if RWOPC < SPICP

Where:

RWOPC is either the desired release from reservoir or the difference between channel level and capacity. SPICP=SPIllway CaPacity (c.f.)

The spillway capacity is a design constant.

RWOPC is controlled by the channel capacity minus a safety factor minus the actual channel level and by the desired release from the reservoir.

RWOPC=RWOP if CDLC $\geq$ RWOP

RWOPC=CDLC if CDLC is < RWOP

Where:

RWOP=Desired releases from the Reservoir Water Operation Procedure

CDLC=Channel Difference between Level and Capacity.

Now starting from the end of the CDLC chain.

CCAPD=(CCAP)(86400)

DCHLV=CCAPD-K_4

CCPLA=DCHLV-CLVA

Where:

CCAPD=Channel CAPacity per Day (c.f.d.),

it converts channel capacity in c.f.s. to cubic feet per day.

K_4 = 2592 (10^5) channel safety factor in c.f.d.

CLVA=Channel LeVel Actual (c.f.d.)

Now,

CDLC=CCPLA if DCHLU is greater than or equal to CLVA

CDLA=0 if DCHLV is less than 0

Thus, if channel capacity minus the safety factor is greater to or equal to the actual channel level, CCPLA is released; if not, nothing is released.

Following the RWOP chain and starting from the end, we find RWOPP coming out of a table WOPT with DAYS as the independent variable. RWOPP is the Reservoir Water Operating Procedure in Percentage.
When multiplied by the Reservoir CAPacity (RCAP), the Reservoir Water Operating Procedure Level (RWOPL) is obtained.

That is, the number of acre-feet of water specified by the rule curve.

Now,

RWOPA=RLVA(K_2)-RWOPL(K_2)

Where:

K_2=Conversion factor that changes acre-feet into c.f.

Then,

RWOP=RWOPA if RLVA RWOPL

RWOP=0 if RLVA RWOPL

which is the point from which we started.

Return now to MINXX, the basic mechanism for determinining water releases for fish life. It has been shown that ROUT3 is the maximum between the water to be released by the rule curve procedure and the water necessary to maintain fish life. MINXX, then, is MINX multiplied by a constant. This puts MINXX in terms of cubic feet per day.

MINX=RMFF1 if RLVA2 $\geq$ 0

MINX=RLVA1 if RLVA2 < 0

Where:

RLVA1=RLVA in c.f.s.

RLVA2=RLVA1-FSF-RMFF

RMFF1=RMFF+FSF

FSF=Fish Safety Factor

Due to the lag between water release and effect on channel level, at the end of certain months a safety factor must be included in the release to insure adequate water for fish needs.

RMFF=Reservoir Minimum Flow for Fish

Fish needs minus the observed minimum flow for each month.

APPENDIX B

Basic Data and Simulation Results

Table B-1. Coefficient of the Regression Upstream Peak Flow Above Mean

Month[a]	Constant	Slope	R^2
November	143	-.37	.72
December	273	-.41	.83
January	331	-.44	.79
February	400	-.47	.83
March	106	-.23	.50
April	162	-.31	.74
May	184	-.44	.77

[a]June, July, August, September, and October were not calculated. These months have consistent low flows; therefore, a built-in correlation between days was not needed.

Table B-2. Downstream Flow Developed as a Function of Upstream Flow

Month	Constant	Slope	R
November	-29	.90	.66
December	245	.92	.66
January	143	1.39	.72
February	141	1.11	.70
March	-131	1.23	.73
April	-213	.97	.73
May	-34	.52	.53
June	4	.54	.55
July	0	.13	.32
August	-10	.20	.25
September	1	.02	.04

Table B-3. Flood Stage at Shedd, Oregon, Regulated and Unregulated Flows

Year	Unregulated	Regulated R97C21	R97C11	R97C5	R67C21	R67C5
	feet	feet	feet	feet	feet	feet
1	14.167	10.535	12.138	13.742	10.535	13.742
2	13.507	10.527	12.110	13.217	10.527	13.217
3	15.878	10.741	12.966	14.512	10.741	14.512
4	16.655	11.699	14.408	15.969	11.762	15.969
5	14.409	10.519	12.078	13.299	10.532	13.299
6	17.022	14.479	16.011	16.722	14.572	16.720
7	14.606	10.645	12.579	12.863	10.657	12.936
8	13.668	10.590	12.360	13.644	10.590	13.644
9	16.905	13.505	15.461	16.372	13.505	16.372
10	13.739	10.464	11.857	13.000	10.464	13.020
11	13.239	10.433	11.732	12.809	10.433	12.809
12	14.238	10.479	11.914	13.388	10.510	13.388
13	15.464	10.594	12.375	13.415	10.594	13.415
14	13.948	10.532	12.128	13.258	10.532	13.258
15	13.909	10.508	12.033	12.454	10.508	12.454
16	13.765	10.648	12.591	12.639	10.648	12.689
17	17.014	14.331	15.930	16.690	14.354	16.691
18	16.962	14.337	15.934	16.657	14.337	16.657
19	16.918	13.782	15.623	16.538	13.782	16.538
20	16.967	14.216	15.868	16.695	14.239	16.710
21	13.882	10.559	12.237	13.152	10.559	13.152
22	13.650	10.540	12.160	12.739	10.540	12.739
23	16.859	13.177	15.270	16.435	13.241	16.434
24	16.141	10.776	13.105	14.463	10.776	14.463
25	16.913	14.128	15.820	16.672	14.128	16.672
26	14.967	10.647	12.587	13.362	10.647	13.362
27	17.066	14.654	16.107	16.773	14.654	16.773
28	16.061	10.890	13.560	15.327	10.890	15.327
29	13.972	10.468	11.871	13.761	10.468	13.761
30	14.154	10.625	12.499	13.590	10.625	13.590
31	17.196	14.968	16.278	16.864	14.968	16.864
32	16.811	13.226	15.298	16.400	13.226	16.400
33	13.919	10.521	12.082	13.270	10.533	13.270
34	14.305	10.702	12.808	13.134	10.702	13.134
35	14.039	10.643	12.571	13.385	10.643	13.385
36	13.851	10.471	11.883	12.921	10.471	12.921
37	14.778	10.544	12.177	13.385	10.544	13.385
38	14.749	10.629	12.516	13.964	10.629	13.964
39	14.284	10.512	12.049	12.821	10.570	12.821
40	16.090	10.882	13.527	15.575	10.882	15.575
41	17.224	15.233	16.401	16.958	15.233	16.958
42	13.706	10.542	12.170	13.434	10.542	13.434
43	16.887	14.287	15.906	16.665	14.287	16.665
44	13.670	10.526	12.106	12.965	10.526	12.965
45	15.984	10.772	13.088	15.032	10.772	15.032
46	14.074	10.501	12.004	13.087	10.501	13.087
47	14.461	10.495	11.979	13.271	10.516	13.378
48	13.786	10.494	11.977	13.801	10.494	13.801
49	16.967	14.119	15.815	16.633	14.144	16.622

Continued

Table B-3. Flood Stage at Shedd, Oregon, Regulated and Unregulated Flows (continued)

Year	Unregu-lated	Regulated				
		R97C21	R97C11	R97C5	R67C21	R67C5
	feet	feet	feet	feet	feet	feet
50	15.826	10.622	12.488	13.964	10.622	13.964
51	14.246	10.649	12.595	13.304	10.649	13.304
52	14.271	10.567	12.268	13.427	10.567	13.427
53	16.979	14.193	15.856	16.653	14.193	16.723
54	14.299	10.614	12.455	13.034	10.614	13.034
55	16.689	11.910	14.531	16.008	11.910	16.008
56	16.728	13.308	15.346	16.404	13.436	16.440
57	15.122	10.533	12.131	13.261	10.533	13.261
58	13.315	10.452	11.808	12.265	10.452	12.265
59	14.310	10.570	12.279	13.425	10.570	13.425
60	13.611	10.435	11.741	13.190	10.435	13.190
61	17.006	14.332	15.931	16.697	14.355	16.716
62	16.900	14.122	15.816	16.633	14.146	16.640
63	14.583	10.539	12.159	13.805	10.560	13.805
64	13.436	10.484	11.936	12.858	10.484	12.858
65	15.033	10.602	12.410	13.319	10.602	13.319
66	16.358	10.993	13.970	15.863	10.993	15.863
67	13.537	10.526	12.105	12.938	10.526	12.938
68	13.715	10.468	11.870	13.153	10.468	13.153
69	17.116	14.608	16.081	16.810	14.608	16.810
70	16.937	13.551	15.488	16.618	13.551	16.618
71	16.782	13.651	15.546	16.411	13.651	16.411
72	17.163	14.817	16.196	16.823	14.820	16.824
73	16.713	13.232	15.302	16.384	13.301	16.473
74	16.810	13.818	15.644	16.548	13.945	16.526
75	14.102	10.567	12.269	13.297	10.567	13.297
76	14.472	10.664	12.656	13.297	10.664	13.297
77	14.468	10.513	12.051	13.686	10.513	13.686
78	13.141	10.384	11.535	12.679	10.384	12.679
79	13.532	10.579	12.318	12.996	10.579	12.996
80	17.036	14.158	15.836	16.643	14.182	16.650
81	15.367	10.658	12.630	14.475	10.658	14.475
82	13.750	10.438	11.751	13.664	10.438	13.664
83	14.629	10.586	12.344	13.551	10.586	13.551
84	13.582	10.450	11.798	13.312	10.450	13.312
85	14.299	10.501	12.003	12.879	10.501	12.879
86	13.144	10.401	11.606	12.827	10.401	12.827
87	13.475	10.553	12.213	13.162	10.553	13.162
88	13.706	10.532	12.127	12.938	10.532	12.938
89	15.762	10.785	13.139	13.662	10.785	13.662
90	14.281	10.561	12.245	13.195	10.561	13.195
91	13.347	10.427	11.710	13.171	10.431	13.171
92	16.872	13.567	15.498	16.561	13.567	16.561
93	13.398	10.467	11.867	12.783	10.467	12.783
94	13.882	10.541	12.162	12.789	10.541	12.789
95	14.160	10.599	12.396	13.912	10.599	13.912
96	14.361	10.541	12.163	12.763	10.565	12.763
97	16.677	13.129	15.242	16.347	13.129	16.347
98	17.005	14.376	15.955	16.702	14.376	16.702
99	14.468	10,580	12.319	12.933	10.580	12.933
100	16.958	14.009	15.755	16.603	14.009	16.603

Table B-4. Flood damage at Shedd, Oregon, Regulated and Unregulated Flows

Year	Unregu-lated	Regulated				
		R97C21	R97C11	R97C5	R67C21	R67C5
	dollars	dollars	dollars	dollars	dollars	dollars
1	25,930	588	3,150	14,220	588	14,220
2	12,600	580	2,960	10,590	580	10,590
3	127,720	816	8,860	46,490	816	46,490
4	270,870	1,869	40,260	133,160	1,938	133,160
5	40,320	571	2,740	11,160	585	11,160
6	347,070	44,523	137,380	284,740	50,030	284,390
7	52,060	709	6,200	8,160	723	8,660
8	13,710	649	4,680	13,550	649	13,550
9	322,820	12,583	102,940	212,110	12,583	212,110
10	14,200	511	2,040	9,100	511	9,240
11	10,750	476	1,910	7,780	476	7,780
12	30,140	526	2,110	11,780	561	11,780
13	103,080	653	4,790	11,960	653	11,960
14	15,640	585	3,080	10,880	585	10,880
15	15,370	559	2,430	5,340	559	5,340
16	14,380	713	6,280	6,610	713	6,950
17	345,430	35,677	130,860	278,220	37,061	278,470
18	334,660	36,045	131,060	271,310	36,045	271,310
19	325,460	14,494	112,550	246,660	14,494	246,660
20	335,720	28,840	127,130	179,300	30,216	282,310
21	15,190	615	3,830	10,150	615	10,150
22	13,530	594	3,300	7,300	594	7,300
23	313,310	10,323	91,570	225,250	10,761	225,130
24	164,350	854	9,820	43,560	854	43,560
25	324,380	23,592	124,270	274,390	23,592	274,390
26	73,550	711	6,250	11,590	711	11,590
27	356,120	54,926	157,170	295,370	54,926	295,370
28	147,660	979	12,970	94,940	979	94,940
29	15,810	514	2,060	14,350	514	14,350
30	25,160	687	5,640	13,170	687	13,170
31	383,120	73,623	192,740	314,310	73,623	314,310
32	303,280	10,657	93,250	218,100	10,657	218,100
33	15,430	573	2,770	10,960	587	10,960
34	34,130	772	7,770	10,030	772	10,030
35	18,340	707	6,140	11,760	707	11,760
36	14,970	518	2,070	8,560	518	8,560
37	62,290	599	3,420	11,760	599	11,760
38	60,570	692	5,760	15,750	692	15,750
39	32,920	563	2,540	7,870	627	7,870
40	153,740	970	12,740	109,710	970	109,710

Continued

Table B-4. Flood Damage at Shedd, Oregon, Regulated and Unregulated Flows--continued

Year	Unregu-lated	Regulated				
		R97C21	R97C11	R97C5	R67C21	R67C5
	dollars	dollars	dollars	dollars	dollars	dollars
41	388,900	89,368	218,240	333,870	89,368	333,870
42	13,970	597	3,370	12,100	597	12,100
43	319,030	33,066	129,430	273,000	33,066	273,000
44	13,720	579	2,930	8,860	579	8,860
45	134,030	849	9,710	77,390	849	77,390
46	20,400	551	2,230	9,700	551	9,700
47	43,400	544	2,180	10,970	567	11,711
48	14,520	544	2,170	14,630	544	14,630
49	335,680	23,085	123,990	226,340	24,580	264,110
50	124,650	684	5,570	15,750	684	15,750
51	30,650	713	6,300	11,200	713	11,200
52	32,150	624	4,050	12,040	624	12,040
53	338,140	27,512	126,400	270,450	27,512	285,000
54	33,780	675	5,340	9,340	675	9,340
55	278,000	2,101	47,580	136,620	2,101	136,620
56	286,060	11,227	96,110	218,830	12,111	226,360
57	82,750	586	3,110	10,900	586	10,900
58	11,270	497	1,990	4,030	497	4,030
59	34,470	627	4,130	12,030	627	12,030
60	13,320	479	1,920	10,410	479	10,410
61	343,780	35,767	130,910	279,590	37,144	283,490
62	321,780	23,250	124,080	266,400	24,697	267,770
63	50,660	593	3,280	14,650	616	14,650
64	12,110	532	2,130	8,120	532	8,120
65	77,440	663	5,030	11,300	663	11,300
66	209,210	1,092	15,790	126,850	1,092	126,850
67	12,810	579	2,930	8,670	579	8,670
68	14,030	514	2,060	10,160	514	10,160
69	366,560	52,146	151,880	302,990	52,146	302,990
70	329,400	12,900	104,530	263,300	12,900	263,300
71	297,280	13,590	108,000	220,200	13,590	220,200
72	376,410	64,606	175,580	305,730	64,764	305,880
73	282,990	10,700	93,460	214,670	11,176	233,170
74	302,970	14,746	113,820	248,810	15,621	244,070
75	22,080	624	4,060	11,150	624	11,150
76	44,100	730	6,730	11,150	730	11,150
77	43,830	564	2,550	13,830	564	13,830
78	10,070	422	1,690	6,890	422	6,890
79	12,770	637	4,390	9,070	637	9,070

Continued

Table B-4. Flood Damage at Shedd, Oregon, Regulated and Unregulated Flows--continued

Year	Unregulated	Regulated				
		R97C21	R97C11	R97C5	R67C21	R67C5
	dollars	dollars	dollars	dollars	dollars	dollars
80	350,070	25,430	125,270	268,570	26,815	269,790
81	97,350	723	6,550	44,240	723	44,240
82	14,280	481	1,930	13,680	481	13,680
83	53,420	645	4,570	12,900	645	12,900
84	13,120	495	1,980	11,250	495	11,250
85	33,810	551	2,220	8,260	551	8,260
86	10,100	442	1,770	7,910	442	7,910
87	12,380	608	3,670	10,220	608	10,220
88	13,970	585	3,080	8,680	585	8,680
89	120,820	863	10,060	13,670	863	13,670
90	32,740	617	3,890	10,450	617	10,450
91	11,490	470	1,880	10,280	474	10,280
92	316,000	13,013	105,100	251,440	13,013	251,440
93	11,850	513	2,050	7,600	513	7,600
94	15,190	595	3,320	7,650	595	7,650
95	25,520	659	4,930	15,390	659	15,390
96	37,470	595	3,330	7,460	622	7,460
97	275,480	9,987	89,880	206,930	9,987	206,930
98	343,590	38,344	132,310	280,750	38,344	280,750
99	43,870	638	4,400	8,640	638	8,640
100	333,740	16,559	120,430	260,030	16,559	260,030

APPENDIX C

Historical and Simulated Frequency Functions

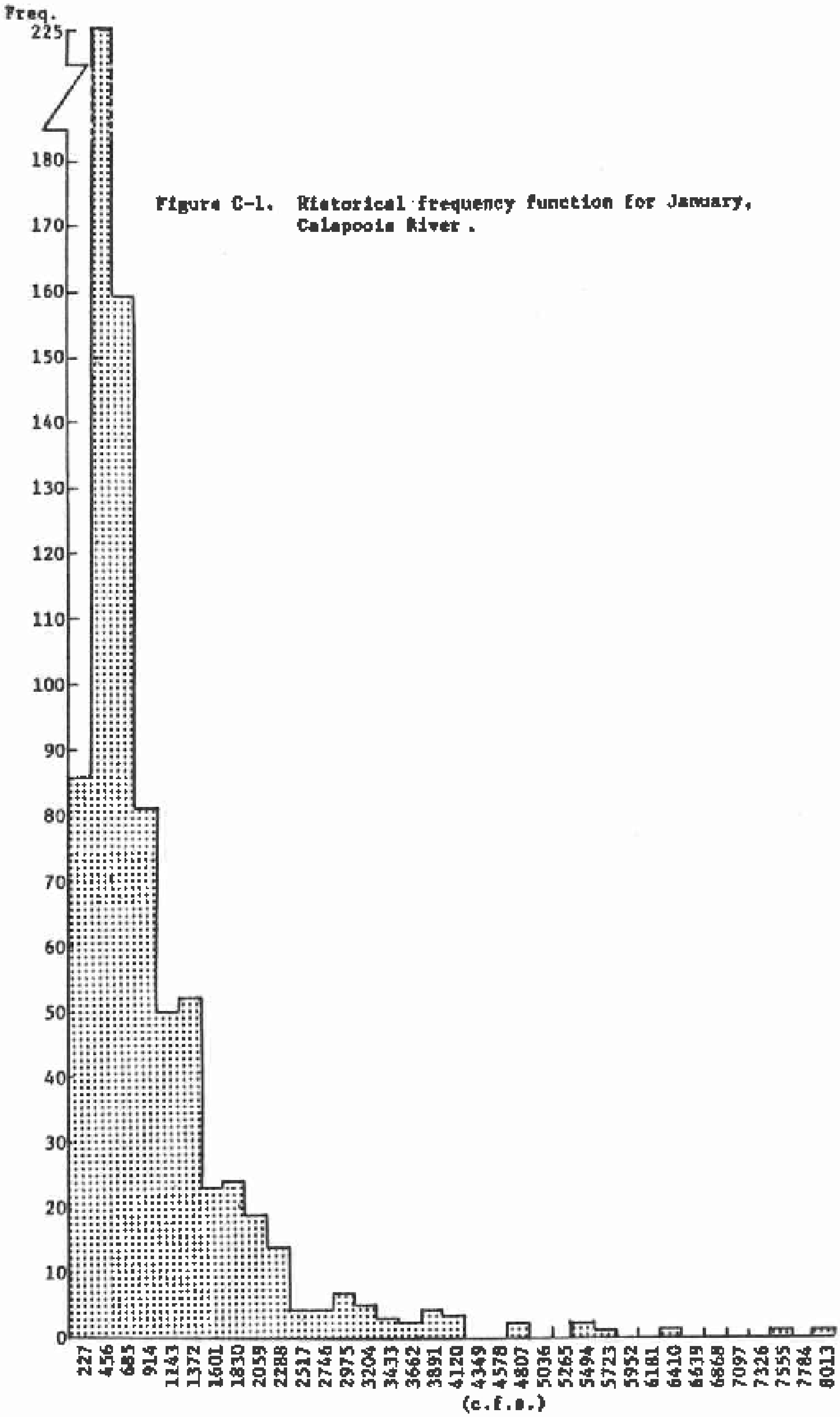


Figure C-1. Historical frequency function for January, Calapooia River .

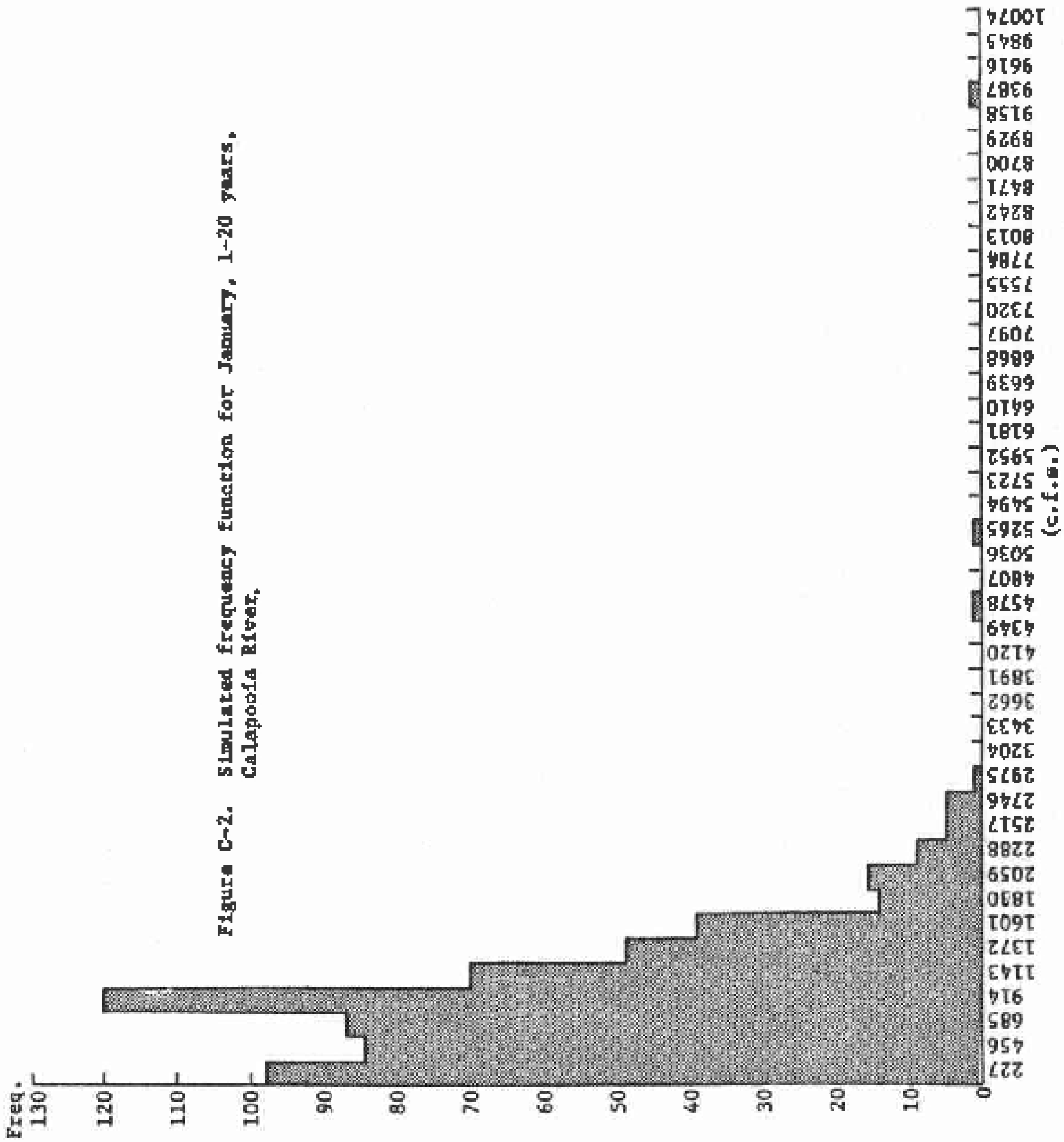


Figure C-2. Simulated frequency function for January, 1-20 years, Calapooia River.

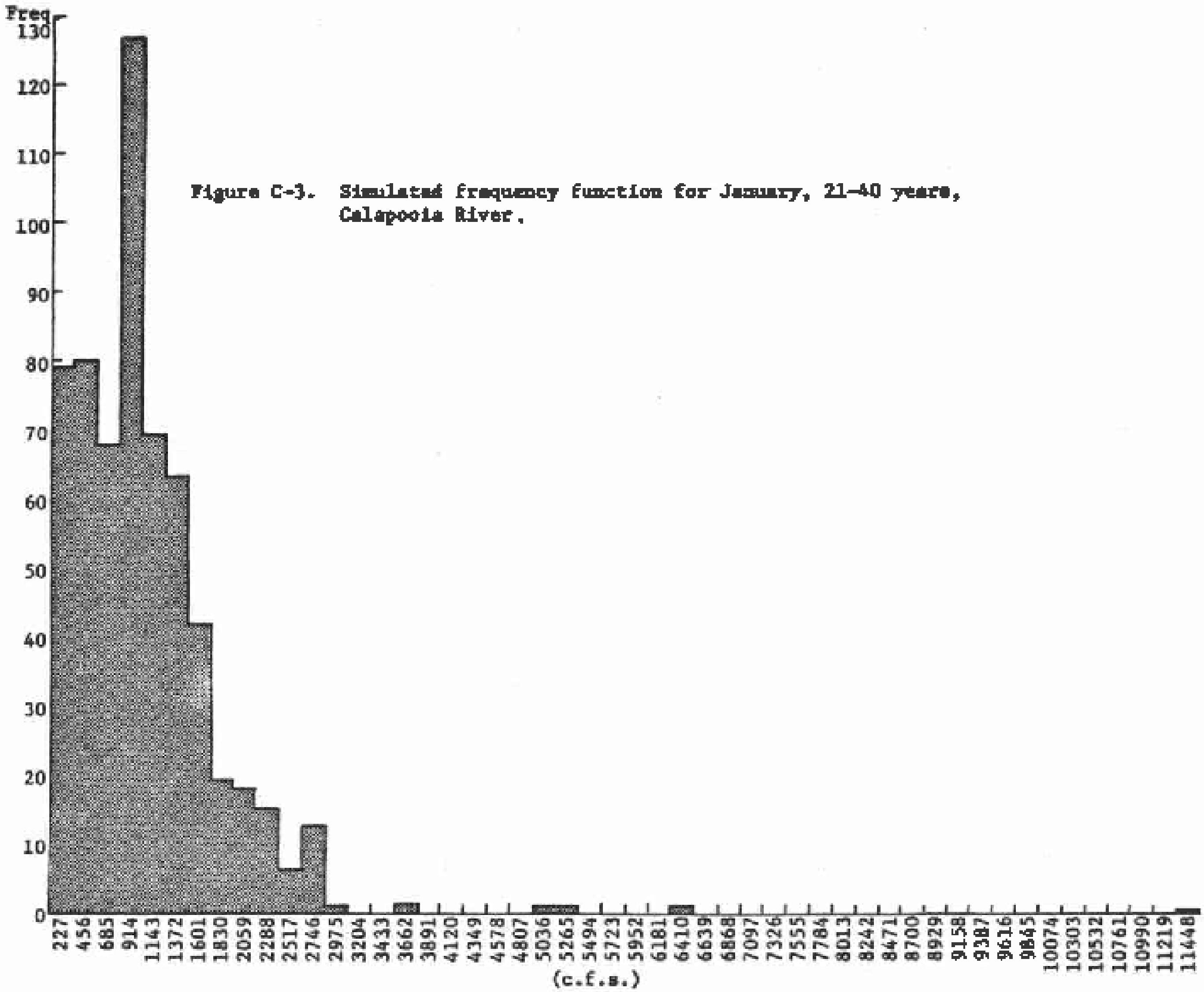


Figure C-3. Simulated frequency function for January, 21–40 years, Calapooia River.

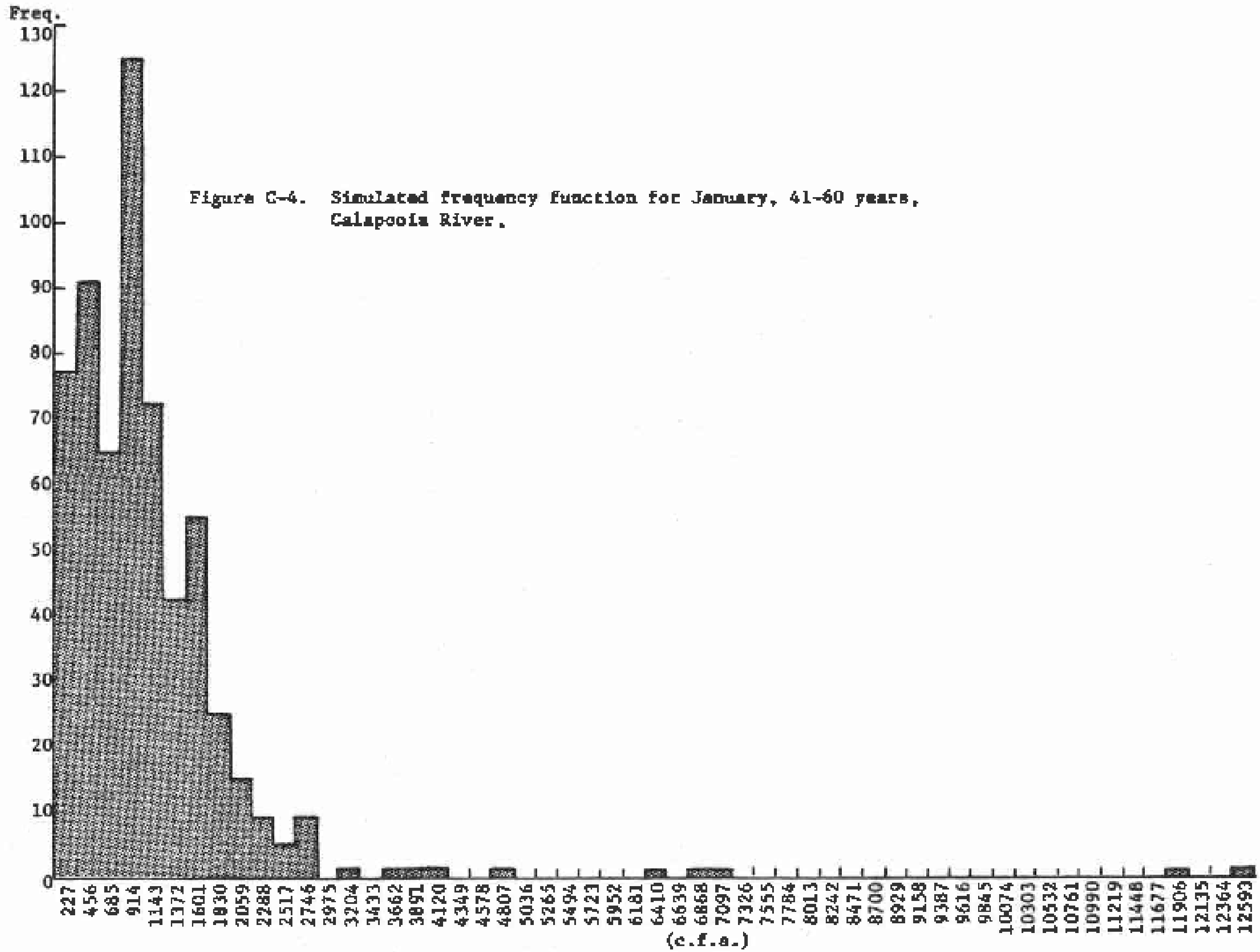


Figure C-4. Simulated frequency function for January, 41–60 years, Calapooia River.

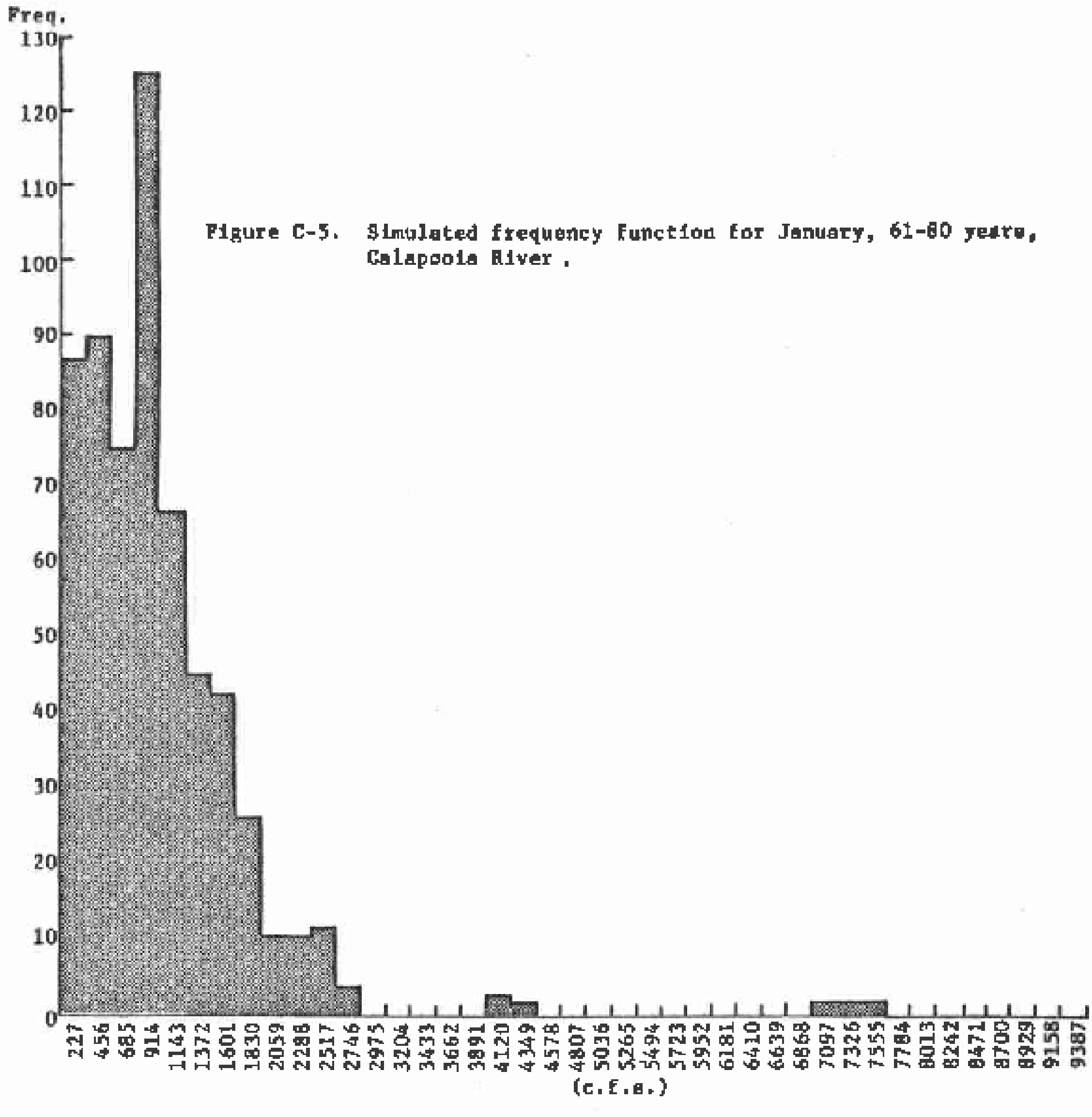


Figure C-5. Simulated frequency function for January, 61-80 years, Calapooia River .

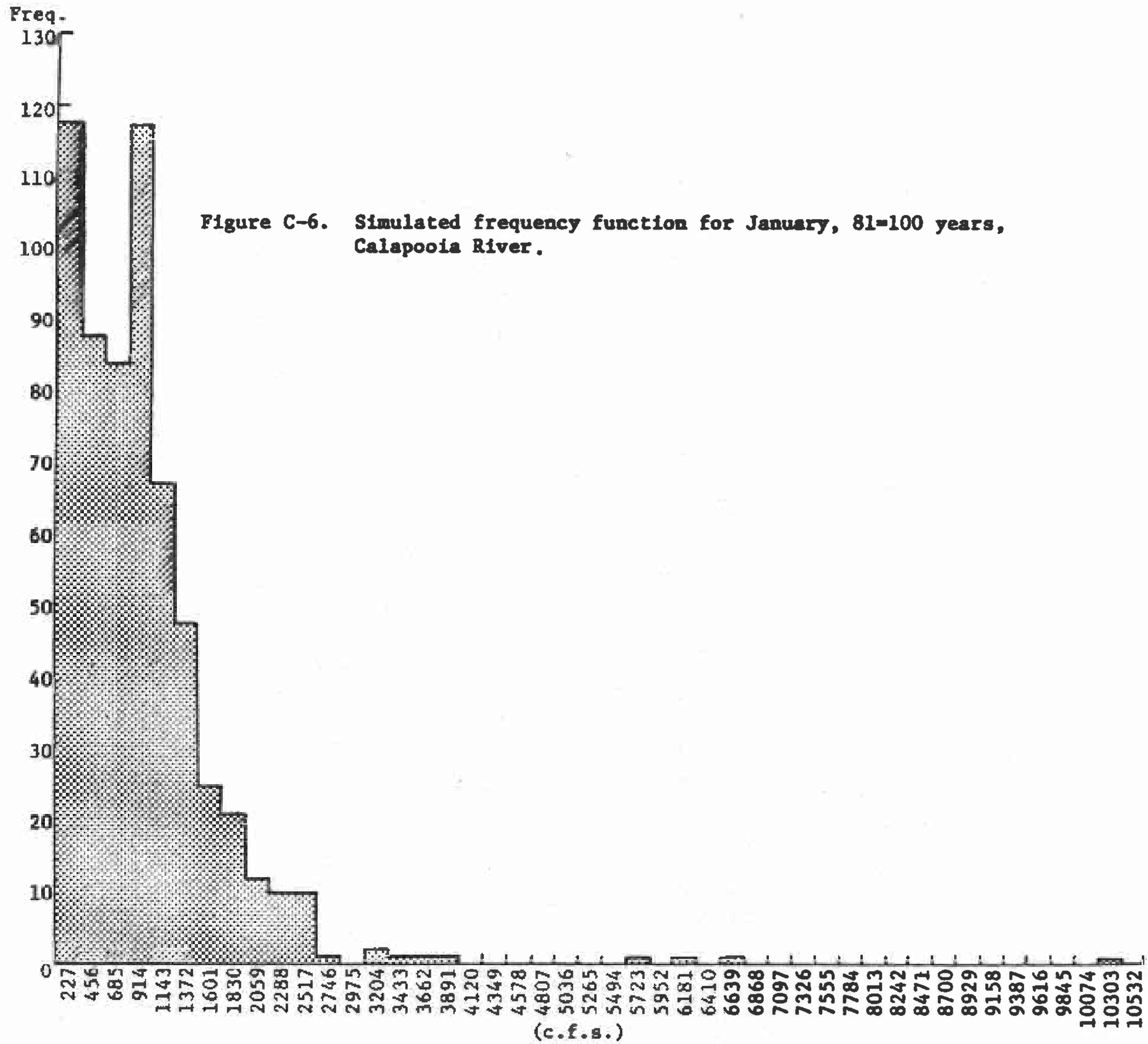


Figure C-6. Simulated frequency function for January, 81=100 years, Calapooia River.